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FINITE ELEMENT IMPLEMENTATION, VALIDATION, AND
DEEP FOUNDATION APPLICATION OF
A BOUNDING-SURFACE PLASTICITY MODEL

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A Thesis

Submitted under the supervision of

Drs. Bengt H. Fellenius and Erman Evgin

in partial fulfillment
of the requirements for the degree of
Doctor of Philosophy
in Civil Engineering

Department of Civil Engineering
Faculty of Engineering
University of Ottawa
Ottawa, Canada



Abdul Ameir Altaee, Ottawa, Canada, 1992



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To Suhaila Abbas

SUMMARY

Several constitutive soil models have been developed using different approaches. Among the most recent approaches is the bounding-surface plasticity concept used by Bardet (1986; 1987) in developing the bounding surface plasticity model for cohesionless soils. The Bardet constitutive model is based on observations made on laboratory tests and well identified aspects of soil behavior by other theories or concepts. It accounts for strain-softening and hardening, stress dilatancy, and accumulation of irreversible strain during cyclic loading.

The Bardet model requires nine parameters in order to describe the behavior of a soil. This thesis presents methods of determination of these parameters and the model sensitivity with respect to variation in values of its parameters. This sensitivity analysis illustrates the relative importance of each one of the model parameters in the overall performance of the model. The model parameters are grouped into three categories based on their relative effect on the model performance.

Details of the model formulation are presented. Numerical problems are observed as associated with extreme softening of the soil (very low values of bulk modulus) in cases of low mean pressure. The problems are due to the use of a linear relation between the bulk modulus and the mean pressure for all levels of the mean pressure. To overcome this difficulty, modification is made in the expression determining the elastic bulk modulus. This is achieved by using one of three alternatives. The first is to modify the pressure dependency of the bulk modulus to incorporate the effect of a transitional pressure. The second is to express the dependency of the Young modulus on the mean pressure in a non-linear form. The third is to use a constant value of the Young modulus representative to the range of mean pressure under consideration. The first and third alternatives are more attractive due to that no additional model parameters are required.

The constitutive model is implemented numerically. The numerical implementation is carried out in two different forms. In the first, the constitutive equations are reduced to the formulation in the triaxial plane of stresses and written explicitly in matrix form. Conventional and non-conventional triaxial tests are simulated under drained and undrained loading conditions. Subsequently, in the second form, the constitutive

relations in matrix form are developed for generalized, three-dimensional stress states. These equations in matrix form are used to incorporate the bounding surface plasticity model into the finite element program, SAC-2 developed at the University of California, Davis. This program is a displacement based, two dimensional, quasi-static program. The global nonlinear solution scheme adopted is incremental iterative with the tangent stiffness and the successive substitution methods are both incorporated as options.

To use the model for the simulation of soil behavior, the constitutive equations are integrated, using a step by step integration, following the desired stress or strain path. It is observed in this study, also by other researchers, that the size of strain increment is not the only factor controlling the accuracy of a numerical integration over a solution step. States in the vicinity of the hydrostatic state of stress and those approaching the region of the critical state require much smaller strain increments to achieve numerical integration with acceptable accuracy. The multistep integration technique is used to overcome the difficulties faced in performing accurate numerical integration. For predicted stress states lying outside the bounding surface, the classical return procedures are used to bring back the stress state to the surface.

The calculation of pore water pressure is achieved by considering an extremely small compressibility for the pore water and soil particles. The pore water pressure is expressed as the product of the volume change and the combined bulk modulus of soil particles and pore water. For drained condition, the combined bulk modulus takes the value of zero. For undrained conditions, it approaches infinity and the soil becomes incompressible.

Verification of implementation of the model in the finite element program is carried out. Finite element calculations are made for soil behavior in different laboratory tests and compared with results of calculations made by integrating the constitutive equations numerically. Four test conditions are considered in the verification process. These are, monotonic drained, monotonic undrained, cyclic drained, and cyclic undrained cases. These comparisons demonstrate that the constitutive model is correctly implemented in the finite element program.

A two-stage validation analysis is carried out to demonstrate the capabilities of the constitutive model in simulating soil behavior in simple stress analysis and boundary value problems.

In the first stage of validation, the performance is investigated when the constitutive model is used to simulate stress-strain and strength of cohesionless soils tested under variety of test conditions. Comparisons are made between laboratory test results and the corresponding model simulations. The behavior of Sacramento River sand, Fuji River sand, a crushed quartz sand, and a sandy silt, are considered in a number of conventional and non-conventional stress-path and strain-path tests. Comparisons between the model simulations and the measured responses of these soils demonstrate a very good performance of the model. The calculations, using the model, reproduce several important aspects of soil behavior.

In the second stage of validation, analyses of three boundary value problems are performed. The first, is the behavior of a model-scale footing resting on the surface of a sandy silt. The second, is the behavior of 14/25 Leighton Buzzard sand in the Cambridge Simple Shear Device. The third, is the analysis of a six metre wide foundation resting on the ground surface for three different conditions of the supporting soil. These are, a clean loose sand, oil contaminated loose sand, and locally oil contaminated loose sand.

The results of the finite element analysis compare very well with the experimentally observed load-displacement relation and the localized nature of soil deformation of the model-scale footing.

The finite element analysis of the behavior of 14/25 Leighton Buzzard sand in the Cambridge Simple Shear Device confirms the experimental observation that the difference between the measurements of shear stresses on the sample core and those normally made in routine test along the full top boundary is smaller than 5%. Further, the agreement between the finite element analysis and the measurements is very good, proving the ability of the model in simulating the behavior of sand.

The finite element analyses of a six metre wide foundation provide results useful for comparison. These include, load-displacement relation, rotation of foundation,

horizontal displacement of foundation, and vertical stresses in the soil close to the foundation face. Vertical displacement of the foundation on oil contaminated sand, for instance, is about 40% larger than that of foundation on clean sand at 150 kPa applied pressure. When a small section of the foundation soil is contaminated with oil, the foundation displaces about 20 mm horizontally toward the contaminated sand side.

A case history of pile loading tests is also included: an instrumented, 285 mm square, precast concrete pile driven 11.0 m into a sand deposit and subjected to three axial compression and one tension static loading tests. The axial loads imposed in the pile during the tests were determined by means of strain gage instrumentation

The pile was equipped with eight load cells that were calibrated in the laboratory under static, repeated, and sustained loading. Cell 1, the uppermost cell, was used as benchmark for determining the relation between the measured strain and the load in the pile.

The first compression loading test consisted of applying load in increments up to a total load of 1,000 kN and unloading from this level. The second compression loading test consisted of reloading the pile to 1,000 kN and unloading. In the third compression loading test, the pile was again loaded to 1,100 kN, which load was reached at a pile head movement of 120 mm and could not be maintained. Five months after the third compression test, a tension test was conducted with the pile loaded by tension to 580 kN. At this load, the upward pile head movement was 65 mm and the load could not be maintained. No attempt was made to measure the residual load resulting from the installation of the test pile, and all gages were zeroed before commencing each loading test.

A preliminary analysis of the field test data using a bearing capacity equation, gave an apparent toe bearing capacity factor of about 10. This value is much smaller than any value reported in the literature for piles driven in sand. This discrepancy raised questions about the validity of the assumption that the residual load due to pile installation is negligible. On the other hand, the unit shaft resistance obtained from the measured test data was a function of the overburden stress only at shallow depth. Deeper down, it appeared to be about constant suggesting the presence of a critical

depth. Further, the total shaft resistance in compression is much larger than the total shaft resistance in tension.

Two different approaches for analyzing the field tests data are undertaken as follows:

In the first approach, the procedure recommended by Fellenius (1989b) for using an effective stress analysis to determine the distribution of residual load was successfully applied to the test data proving that the pile was subjected to residual load. The data were corrected for residual load and the beta-values and the toe bearing capacity coefficient in the soil at the site were determined. Without the correction for residual load, considerable errors resulted in the evaluation of the test results with regard to shaft and toe resistances. As to the toe resistance, the error is a 50 percent too low value --190 kN instead of 360 kN-- which error is due to the neglect of a residual toe load of 170 kN. The analysis following this approach indicated no difference between the shaft resistance in tension and the shaft resistance in compression.

In the second approach, the load transfer behavior of the test pile was modelled using the finite element program developed herein. The analysis was performed in a continuous manner for all stages of the testing programme: the pile installation, the three compression tests in sequence, and the tension test. The numerical results from each event provided the initial state of stresses and strains in the modelling of the subsequent event.

One set of model parameters is used for each soil layer for all stages of the numerical analysis. These are based on the results of the friction-jacket cone penetrometer tests, and the experience with the sensitivity of the model simulations to each one of its parameters.

In order to model correctly the behavior of the test pile, it was necessary to include the effect of pile installation. A new procedure is developed and applied to determine the magnitude and distribution of residual load in the test pile. The procedure differentiates between residual loads developed in piles in sand as a result of the installation method (whether pushed-in, buried, or driven).

Residual load in a pile affects its load-movement response. This is demonstrated by the present finite element analysis. A pile with no residual load requires larger movements to mobilize the same load as apposed to a pile with residual load. This is in agreement with the laboratory observations on small-scale piles, results from analytical analysis, and results of analyses of some case histories reported in the literature.

Each test showed an increase in the load carried by the pile toe and reduction in the load carried by the pile shaft for 1,000 kN load at the pile head. The computed toe resistances for each of the three compression tests are 360 kN, 400 kN, and 430 kN and the shaft resistances are 640 kN, 600 kN, and 570 kN, respectively. The shear stress along the pile shaft decreased with each loading event. The reduction in the shear stress is higher in the vicinity of the pile toe compared to the reductions at other locations. The gradual degradation of shaft resistance is attributed to a potential progressive decrease in the horizontal (lateral) stress in the soil. Indeed, the present study indicates that the horizontal stress decreases from test to test.

Before the tension test started, the pile was under compression due to the residual load that remained after the third compression test. With increasing tension load at the pile head, the soil along the lower portion of the pile mobilizes more and more shaft resistance and the residual load at the toe unloads. At the complete unloading, the distribution of the axial load in the pile represents the load remaining in the pile after the tension test. The residual toe load diminishes to zero at the end of the test and a significant amount of residual tension load is produced as the result of the tension test.

For the third compression test and the tension test, the ultimate shear stresses on the vertical planes passing through the soil elements adjacent to the pile shaft are similar along the whole pile length. This means that the ultimate shaft resistance in compression is equal to that in tension for this test pile.

The agreement between the finite element calculations and load values measured in the pile during the three compression tests is excellent only when the residual load is considered in the analysis. The residual loads used in the analysis of all tests differ from each other. This is due to a gradual increase in the residual toe load that develops

with each repeated compression loading due to the accumulation of the downward toe movement at the end of each load repetition.

Using the residual load computed as an integral part of the analysis eliminates the need for the Critical Depth concept in order to explain the measured data. Further, the test results and the results of the finite element analyses demonstrate the importance of loading history on the development of residual load and load transfer.

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CHAPTER ONE

INTRODUCTION

1.1 Statement of the Problem

Soil differs from many other building materials. It exhibits a non-linear stress-strain response at an early stage of loading, it undergoes volume change in response to changes in mean pressure, compressing or dilating during shearing, and, sometimes after reaching a peak strength, it exhibits strain softening. Added to the above, the presence of pore water is one of the unique characteristics of soils which increases the difficulty in quantifying soil behavior.

The analysis of most geotechnical problems is conventionally carried out in a two-stage process: first, deformation analysis which provides information about deformations "at working stress levels", and second, a stability analysis which provides information on a limiting load but no load-deformation information. In fact, the two-stage process of analysis is classic because of its simplicity and that, in the past, there was no alternative analysis method that could provide more accurate solutions.

The conventional analysis has several shortcomings and there is a growing need for comprehensive constitutive soil models that more adequately describe the stress-strain behavior of soil. However, developing a general constitutive model to describe the stress-strain behavior of all soils and suitable for all loading conditions is difficult. Until now, most attempts have been restricted to developing constitutive models able to

describe the behavior of certain soils subjected to limited loading types and stress levels.

Several constitutive soil models have been proposed using different approaches. Among the most recent approaches is the bounding-surface plasticity concept put forward by Dafalias and Popov (1974; 1975; 1976) and Krieg (1975). Its attraction is that it generalizes the conventional plasticity theory. This approach has been used in the Bardet (1986; 1987) bounding surface plasticity model, which is one of the most recently developed models for cohesionless soils. The Bardet model is based on observations made on laboratory test results combined with well identified aspects of soil behavior by other theories or concepts.

Before a constitutive model can become a useful engineering tool, a critical evaluation of the model is necessary and, most importantly, the constitutive model must be incorporated into a numerical method of analysis, such as a finite element program. The incorporation of a constitutive soil model with a proven ability to reproduce observed soil behavior is vital for the better understanding of the behavior of soil masses and soil-structure interaction problems.

An example of a geotechnical application requiring an elaborate soil constitutive model is the load transfer of axially loaded piles. In its most simplistic approach, the capacity of axially loaded piles is calculated as the sum of two individual components, shaft resistance and toe resistance. Conventionally, the theoretical model used to calculate shaft resistance is similar to that used to analyze resistance to sliding of a rigid body in contact with soil. The conventional model for the determination of toe resistance is based on idealizing the soil as a rigid plastic material or, more recently, as a linear or non-linear elastic-plastic material.

Pile load transfer is a complex soil-structure interaction problem. It is governed by several factors, such as how the pile was installed, presence of residual stress in the pile and the surrounding soil, the three-dimensional nature of the interaction between the pile and the soil, loading sequence, the non-linear stress-dependent behavior of the soil, and pile material properties, size, length, and stiffness.

At present time, there is no method available that addresses all interaction aspects for piles. Developing a computer program which incorporates a constitutive soil model able to reproduce soil behavior as measured in laboratory testing is an important step toward having such a method.

1.2 Research Objectives

The main objectives of the research work are:

1. To develop and verify a non-linear finite element program incorporating the bounding-surface plasticity model of Bardet to model the stress, strain, and strength behavior of cohesionless soil.
2. To use the program to study the load-transfer of axially loaded piles, specifically the Critical Depth Concept and residual loads.
3. To use the program in the analysis of full-scale loading tests of an instrumented pile.

1.3 Scope of Work

The work has proceeded as follows:

1. Developing a computer program for simulating soil behavior using the Bardet (1986; 1987) bounding-surface plasticity formulation.
2. Simulating stress, strain, and strength behavior of cohesionless soils by means of the above program. Comparisons are made between simulated results and published results of laboratory tests as well as results from tests carried out as part of the research work. The laboratory tests include a variety of conditions, such as initial density of the sample, initial state of stress, drainage condition, stress or strain path followed during the test, and loading type (monotonic or random cyclic).
3. Numerical implementation of the Bardet (1986; 1987) bounding-surface plasticity model into a non-linear finite element program called Soil Analysis Code, SAC,

developed at the University of California, Davis (Herrmann et al., 1986).

4. Verifying the numerical implementation of the model into the computer program by comparing results obtained using the finite element program, Point 3, and the program developed in Points 1 and 2. The comparisons are made for variety of test conditions, such as monotonic and cyclic triaxial tests with drained and undrained test conditions.
5. Validating the bounding-surface plasticity model by comparing calculated behavior with test results of different, well documented, boundary value problems.
6. Simulating numerically the development of residual stress (locked-in stress).
7. Analyzing a case history of pile loading tests using the developed finite element computer program. To verify agreement between simulation and observations, comparisons are made between the results of the numerical simulations and those obtained from the loading tests.

1.4 Outline of Thesis

Chapter 2 details the modelling of soil behavior using the bounding-surface plasticity concept. The basic formulation of the Bardet (1986; 1987) bounding-surface plasticity model is presented and discussed.

Chapter 3 describes the numerical implementation of the bounding surface plasticity model into the finite element program. This chapter also presents four different cases of verification of the correctness of the numerical implementation of this stage.

Chapter 4 describes the parameters of the bounding surface plasticity model, explains the method of determining each parameter, and shows the sensitivity of the model to the variation of these parameters.

Chapter 5 elaborates on the performance of the bounding-surface plasticity model when used to simulate stress-strain and strength of cohesionless soils tested under a variety of test conditions.

Chapter 6 demonstrates the performance of the bounding-surface plasticity model when applied to the analysis of two well documented boundary value problems: a footing placed on the surface of a sandy silt and the behavior of Leighton Buzzard sand in the Cambridge Simple Shear Device.

Chapter 7 presents the state-of-the-art of the performance of single piles installed in sand and subjected to axial loading.

Chapter 8 presents and discusses the results of full-scale loading tests of an instrumented precast concrete pile in sand.

Chapter 9 presents the finite element analyses to model the pile tests presented in Chapter 8. The pile performance during different stages of the loading test is analyzed and compared with the actual measurements.

Finally, Chapter 10 presents the conclusions, and provides some suggestions for further research and development.

CHAPTER TWO

BOUNDING SURFACE PLASTICITY MODEL

2.1 Introduction

Experimental observations on stress strain behavior of metals, soils, and other materials show that stress-strain curves converge at bounds in the stress-strain space. These bounds cannot be crossed by any stress increment, however, they may change in size during the process of cyclic loading. Such a bound has been called bounding surface.

The bounding surface concept was introduced in plasticity by Dafalias and Popov (1974; 1975; 1976) and Krieg (1975) for the description of the cyclic and monotonic behavior of metals. It has been used increasingly in the development of models for the mechanical behavior of clays, sands, concrete, and rocks. Few examples of these work are Aboim and Roth (1982), Bardet (1986; 1987; 1988), Dafalias and Herrmann (1980; 1982), Hashiguchi and Ueno (1977), and McVay and Taesiri (1985).

The plastic modulus, the most important parameter in the simulation of the material behavior, is expressed in terms of the proximity of the current state of stress to the bounds. This allows the simulation of hysteretic stress-strain response using a continuous function of the distance between the stress state and the bounds.

A comprehensive description of the mathematical foundation of the bounding surface plasticity is presented by Dafalias (1986). The application of the theory to isotropic cohesive soils was treated by Dafalias and Herrmann (1986) and to anisotropic cohesive soils by Anandarajah and Dafalias (1986).

Using the bounding surface plasticity concept, Bardet (1983; 1985; 1986; 1987) developed constitutive relations to simulate the behavior of sands under cyclic and monotonic loading conditions. The critical state was considered as the ultimate state which controls the evolution of the bounding surface.

The attractive features of this model are its account for several experimental observations such as strain-softening, stress dilatancy, and accumulation of irreversible strain during cyclic loading. The model has nine material parameters which can be determined, with relative ease, from conventional laboratory tests.

In the following, the bounding surface plasticity model developed by Bardet (1986; 1987) is presented.

2.2 Stress Invariants

There exist quantities of stress or strain that are independent of orientation of the coordinate system. These quantities are called stress or strain invariants. These invariants are used in the present formulation as follows.

$$I_1 = \sigma_{ii} \quad (2.1)$$

$$J_2 = \frac{1}{2} s_{ij} s_{ij} \quad (2.2)$$

and

$$J_3 = \frac{1}{3} s_{ij} s_{jk} s_{ki} \quad (2.3)$$

where,

I_1 = first stress invariant

J_2 = second deviator stress invariant

J_3 = third deviator stress invariant

i and j are equal to 1,2, and 3.

In the following, the first stress invariant, I_1 , is replaced by I for convenience

$$I = I_1 \quad (2.4)$$

the square root of the second deviator stress invariant is termed J,

$$J = \sqrt{J_2} \quad (2.5)$$

the third root of the third deviator stress invariant is termed S,

$$S = J_3^{1/3} \quad (2.6)$$

The Lode angle, α , (Zienkiewicz and Pande, 1977) is used to represent the third deviator stress invariant such that

$$\alpha = \frac{1}{3} \sin^{-1} \left[\frac{3\sqrt{3}}{2} \frac{S^3}{J^3} \right] \quad (2.7)$$

where,

s_{ij} = deviator stress, which can be expressed as

$$s_{ij} = \sigma_{ij} - \frac{1}{3} \sigma_{kk} \delta_{ij} \quad (2.8)$$

in which,

σ_{ij} = effective stress tensor

δ_{ij} = Kronecker delta such that,

$$\delta_{ij} = 0 \text{ if } i \neq j, \text{ and } \delta_{ij} = 1 \text{ if } i = j \quad (2.9)$$

The value of the Lode angle, α , is varied between two extremes, $-\pi/6$ and $+\pi/6$. The minimum value corresponds to the conventional triaxial extension, whereas the maximum value corresponds to the conventional triaxial compression.

2.3 Strain Decomposition

As in the classical theory of plasticity (Desai and Sirwardene, 1984), bounding surface plasticity models assume that the total strain increment tensor, $d\epsilon_{ij}$, can be decomposed into elastic, $d\epsilon_{ij}^e$, and plastic, $d\epsilon_{ij}^p$, parts in the following form.

$$d\epsilon_{ij} = d\epsilon_{ij}^e + d\epsilon_{ij}^p \quad (2.10)$$

Different constitutive relations are used to compute the components of elastic and plastic strain.

2.4 Elastic Constitutive Relations

The elastic constitutive relations are given in the generalized form

$$d\epsilon_{ij}^e = C_{ijkl} d\sigma_{kl} \quad (2.11)$$

where,

C_{ijkl} = tensor of elastic compliance

For the present model, the elastic constitutive relations are expressed as follows

$$d\epsilon_{ij}^e = \frac{1}{9B} \delta_{ij} d\sigma_{kk} + \frac{1}{2G} ds_{ij} \quad (2.12)$$

where,

B = bulk modulus

G = shear modulus

The bulk modulus is assumed to be proportional to the first stress invariant and the shear modulus is related to the bulk modulus using Poisson's ratio which is taken as a constant. The bulk modulus, B, is expressed as

$$B = \frac{1+e_o}{\kappa} I/3 \quad (2.13)$$

where,

e_o = initial void ratio

κ = a model parameter which represents the slope of unloading and reloading line of isotropic consolidation test presented as a plot of e versus $\ln(p)$, where e is the void ratio and p is the mean pressure ($I/3$).

and the shear modulus, G , is related to the bulk modulus as

$$G = \frac{3(1-2\nu)}{2(1+\nu)} B \quad (2.14)$$

in which

ν = Poisson's ratio, used as a model parameter

Therefore, nonlinear elastic constitutive relations are used to specify the elastic portion of the strain. Furthermore, only two model parameters, κ and ν , are used in the formulation.

2. 5 Plastic Constitutive Relations

The plastic constitutive relations are given by

$$d\varepsilon_{ij}^p = \left\langle \frac{1}{H} \eta_{kl} d\sigma_{kl} \right\rangle \eta_{ij} \quad (2.15)$$

The term between the symbol $\langle \rangle$ is the loading function,

and

H = plastic modulus

η_{ij} = unit vector normal to the bounding surface at the image stress point.

The symbol $\langle \rangle$ represents a function such that

$$\langle x \rangle = x \text{ if } x > 0, \text{ and } \langle x \rangle = 0 \text{ if } x \leq 0 \quad (2.16)$$

where x is a scalar quantity.

The constitutive relations represented by Equation (2.15) signify that plastic strains are assumed to occur when the stress state is on or inside the bounding surface and the loading function is positive. Furthermore, the material is assumed to behave elastically if the loading function is negative (unloading) or the loading function is equal to zero (neutral loading).

The calculation of plastic strains requires an equation for the bounding surface in the stress space, a mapping rule defining the location of the image stress, an evolution rule of the bounding surface during plastic flow, and a relation between plastic moduli at the image point and the current stress state.

2.6 Definition of Bounding Surface

The selection of a bounding surface is an important step in establishing a bounding surface plasticity model requiring experimental observations and well established behavior of soils. An ellipsoidal bounding surface in the first stress invariant, I , and the square root of the second deviator stress invariant, J , space is selected as shown in Fig. 2.1.

The ellipse has a vertical normal at the critical and characteristic states to satisfy the requirement of zero plastic volumetric strain increments at these states. The following equation is used for the ellipse bounding surface

$$f = \left[\frac{I/3 - A}{\rho - 1} \right]^2 + 3 \left[\frac{J}{M_{(\alpha)}} \right]^2 - A^2 = 0 \quad (2.17)$$

where

$3A$ = the coordinate of the ellipse summit along the I -axis

ρ = a model parameter that corresponds to the bounding surface (ellipse) aspect ratio

$M_{(\alpha)}$ = a parameter that depends on the Lode angle, α , and the slopes of the critical state lines in compression and extension sides of the mean pressure versus deviator stress plane as

$$M_{(\alpha)} = \frac{2 M_e M_c}{M_c + M_e - (M_c - M_e) \sin 3\alpha} \quad (2.18)$$

For a value of α of $-\pi/6$ (minimum value), $M_{(\alpha)}$ is equal to M_e , and for a value of α of $\pi/6$ (maximum value), $M_{(\alpha)}$ equals to M_c . The two model parameters, M_c and M_e , are function of the post-peak (ultimate) friction angles corresponding to the conventional triaxial compression, ϕ_c and extension, ϕ_e , as follows:

$$M_c = \frac{6 \sin \phi_c}{3 - \sin \phi_c} \quad (2.19)$$

and,

$$M_e = \frac{6 \sin \phi_e}{3 + \sin \phi_e} \quad (2.20)$$

The cross section of the bounding surface in the deviatoric plane for various values of M_e/M_c is shown in Fig. 2.2.

2.7 Mapping Rule

The mapping rule is the procedure by which a stress state on the bounding surface (image stress) is determined as corresponding to a stress state (current stress) within the bounding surface. Different mapping rules have been used in formulating different bounding surface models (Dafalias and Herrmann, 1980; Bardet, 1988). The radial mapping rule (Dafalias and Herrmann, 1980) is used to define an image point on the

bounding surface in the present model. The image stress is obtained by intersecting the bounding surface with a radial line connecting the projection center and the actual stress state represented by σ_{ij} . The present model uses the coordinate center of the principal stress space as the projection center.

At the image point, the first stress invariant is

$$\bar{I} = \gamma (3A) \quad (2.21)$$

where γ is a scalar quantity obtained from

$$\gamma = \frac{1 + (\rho - 1)\sqrt{1 + x^2\rho(\rho - 2)}}{1 + (\rho - 1)^2x^2} \quad (2.22)$$

and

$$x = \frac{\sqrt{3} J}{M_{(\alpha)} \frac{\bar{I}}{3}} \quad (2.23)$$

2.8 Plastic Flow Direction

In this bounding surface plasticity model, the direction of the increment of plastic strain is assumed co-linear with the normal to the bounding surface at the image point. This is equivalent to the normality condition in classical theory of plasticity. The plastic flow direction depends on the present state of stress and it is not related to the direction of the stress increment. The plastic flow direction is expressed in terms of the components of the unit vector normal to the bounding surface at the image stress, η_{ij} .

The equation for the unit normal vector is

$$\eta_{ij} = \frac{1}{g} \frac{\partial f}{\partial \sigma_{ij}} \quad (2.24)$$

where,

f = the function of the bounding surface

g = the gradient amplitude:

$$g = \sqrt{\frac{\partial f}{\partial \sigma_{ij}} \frac{\partial f}{\partial \sigma_{kl}}} \quad (2.25)$$

or

$$g = \sqrt{3 (f_{,I})^2 + (f_{,J})^2 (1 + 9 \xi^2 \cos^2 3\alpha)/2} \quad (2.26)$$

$$\begin{aligned} \frac{\partial f}{\partial \sigma_{ij}} = & \delta_{ij} ((f_{,I}) - \sqrt{3} \xi (f_{,J})) + (f_{,J}) s_{ij} / 2J (1 - 3 \xi \sin 3\alpha) \\ & + 3\sqrt{3} \xi (f_{,J}) s_{ik} s_{kj} / 2J^2 \end{aligned} \quad (2.27)$$

and

$$\xi = \frac{M_c - M_c}{2 M_c M_c} M_{(\alpha)} \quad (2.28)$$

$$f_{,I} = (\gamma - 1) M_{(\alpha)} / 3 \quad (2.29)$$

$$f_{,J} = \sqrt{3} \gamma \times (\rho-1)^2 \quad (2.30)$$

$f_{,I}$ and $f_{,J}$ are the partial derivatives of the function f of the bounding surface (Equation 2.17) with respect to the first stress invariant, I , and the square root of the second deviator stress invariant, J , respectively.

2.9 Evolution of Bounding Surface

The bounding surface moves during plastic flow. The plastic volumetric strain, ε_{pv} , is assumed as the only internal variable which governs the evolution of the bounding surface. This assumption is based on the experimental observations on Sacramento River sand. Triaxial tests of this sand under a wide range of consolidation pressures (30 kPa-4,000 kPa) and different initial void ratios ($e_o=0.87$ and $e_o=0.61$) show that the critical state is an asymptotic state toward which stresses and void ratios are forced to converge as shown in Fig. 2.3.

During loading, the bounding surface expands or contracts. It always envelops the current stress state because a stress state outside the bounding surface has no physical meaning.

The projection of the ellipse summit, which is the point of intersection of the bounding surface with the critical state line, along the mean pressure axis was used as a measure of the bounding surface evolution. For a given stress state, the location of the projection of the ellipse summit is determined from the intercept of the critical state line with an unloading re-loading line passing through the stress state ($e, I/3$) under consideration. The value of A is calculated from:

$$A = A_o \exp \left[\frac{\Gamma - e - \kappa \ln(I/3)}{\lambda - \kappa} \right] \quad (2.31)$$

where

A_0 = a unit pressure

Γ = a model parameter corresponding to the critical void ratio at the unit mean pressure,

λ = a model parameter corresponding to the slope of virgin compression line in isotropic consolidation test represented as a plot of e versus $\ln(p)$, where p is the mean pressure ($I/3$),

e = current void ratio.

According to Eq. 2.31, the bounding surface expands ("A" increases) if the plastic volumetric strain increases (compression states), and shrinks ("A" decreases) if the plastic volumetric strain decreases (dilation state). Furthermore, the bounding surface stops its evolution for stress states along the critical state.

Fig. 2.4 shows a graphical representation for the change in the bounding surface size as expressed in terms of "A". In this figure, a current stress state is represented by the current void ratio, e_a , and the current mean pressure, $I_a/3$. For this state of stress, the value of the "A" is found by plotting a straight line parallel to the unloading reloading line (with slope = κ) and passing through the current stress state. This line intersects the critical state line at a mean pressure equal to " A_a ". If the material undergoes a compressive volumetric plastic strain due to a change in its state from $e_a, I_a/3$ to $e_b, I_b/3$, then, the new "A" is found in a similar way to that of " A_a ". The new value of "A" will be " A_b " as shown in Fig. 2.4. On the other hand, if the material undergoes a dilative plastic volumetric strain due to a change in its state from $e_a, I_a/3$ to $e_c, I_c/3$, then, the new value of "A" will be " A_c ". Notice that " A_b " is greater than " A_a " and " A_c " is smaller than " A_a ".

2.10 Plastic Modulus at Image Stress

The plastic strain change results from the current stress change and the change of the corresponding image stress are assumed equal in bounding surface plasticity formulation. This is equivalent to

$$\frac{1}{H} \eta_{ij} d\sigma_{ij} = \frac{1}{H_b} \eta_{ij} d\bar{\sigma}_{ij} \quad (2.32)$$

where,

H = plastic modulus at stress point

H_b = plastic modulus at corresponding image point

$d\sigma_{ij}$ = change of stress at stress point

$d\bar{\sigma}_{ij}$ = change of stress at corresponding image point

The plastic modulus at the image point is expressed as

$$H_b = - \frac{\partial f}{\partial \varepsilon_v} \frac{1}{g^2} \frac{\partial f}{\partial \sigma_{ii}} \quad (2.33)$$

Substituting the suitable terms into Eq. (2.33), the following expression is obtained for the plastic modulus at the image point.

$$H_b = \frac{1+e_o}{\lambda-\kappa} \frac{A}{g^2} M_{(\omega)}^2 (\gamma-1)(\gamma+\rho(\rho-2)) \quad (2.34)$$

The plastic modulus at the image point (on the bounding surface) can have positive, negative or zero value. When the stress state is below the critical state line, H_b is

positive and the material is undergoing stress hardening. On the other hand, if the stress state of the material is above the critical state, then H_b is negative which corresponds to strain softening behavior of the material. When the stress state reaches the critical state, then H_b will be zero and there will be no further hardening or softening associated with this state.

2. 11 Plastic Modulus at Stress State

The plastic modulus at a stress point is expressed in terms of the plastic modulus at the image point and a positive function which depends on the relative distance between the stress point and its corresponding image stress. The plastic modulus at the stress point is given as:

$$H = H_b + \frac{1+e_o}{\lambda-\kappa} \frac{\delta}{\delta_{\max}-\delta} h_o I < \frac{M_p}{M_c} - x > \quad (2.35)$$

and

$$\delta = \sqrt{(\sigma_{ij}-\sigma_{ij}^*)(\sigma_{ij}-\sigma_{ij}^*)} \quad (2.36)$$

where

δ = distance between current stress state and its image point

$\delta_{\max}$ = maximum possible distance between current stress state and its image point

h_o = a model parameter, plastic modulus constant

M_p = slope of the peak failure line defined as

$$M_p = \frac{6 \sin \phi_p}{3 - \sin \phi_p} \quad (2.37)$$

where ϕ_p is the peak friction angle measured in conventional triaxial compression test.

Since a stress state outside the bounding surface is not permitted, the maximum possible distance, $\delta_{\max}$, is the distance between the stress origin and the image point on the bounding surface.

The term $\frac{\delta}{\delta_{\max} - \delta}$ which reflects the relative distance between the stress point and its image on the bounding surface can be replaced by the following term.

$$\frac{\delta}{\delta_{\max} - \delta} = \frac{\gamma(3A)}{I} - 1 \quad (2.38)$$

The relation presented in Eq. (2.35) for the plastic modulus at the stress point satisfies the following

$$H = +\infty \text{ if } \delta \geq \delta_{\max} \quad (2.39)$$

and

$$H = H_b \text{ if } \delta = 0 \quad (2.40)$$

The first condition which is represented by Eq. (2.39) ensures that the current stress point will move with the bounding surface when it reaches it. And the second condition expressed in Eq. (2.40) ensures that the strains will be elastic for stress states very far from the bounding surface.

As mentioned previously in this chapter, the expression of plastic modulus is an important part in the formulation of any bounding surface plasticity model. The form presented in Eq. (2.35) is composed of two parts. The first part is the plastic modulus at the image point, H_b , and this was described above. The second part of the plastic modulus equation is a positive function which depends on the relative distance between the stress point and its image on the bounding surface.

2.12 Formulation of Model in Triaxial Plane

In the process of calibrating the model to a specific soil in order to determine the model parameters, only simple and conventional stress path tests are used. There is a need to simplify the formulation presented in the preceding paragraphs to a set of equations suitable for the triaxial plane.

The stresses acting in such a plane can be represented by

$$p = \frac{\sigma_1 + 2\sigma_3}{3} \quad (2.41)$$

$$q = \sigma_1 - \sigma_3 \quad (2.42)$$

and the strains by

$$\varepsilon_v = \varepsilon_1 + 2\varepsilon_3 \quad (2.43)$$

$$\varepsilon_q = \frac{2(\varepsilon_1 - \varepsilon_3)}{3} \quad (2.44)$$

where

$$p = \text{mean stress } (= I/3)$$

q = deviator stress ($= J/\sqrt{3}$)

σ_1 = axial stress

σ_3 = radial stress

ε_v = volumetric strain

ε_q = deviatoric strain

ε_1 = axial strain

ε_3 = radial strain

Total strain increment ($d\varepsilon_v, d\varepsilon_q$) can be decomposed to its elastic and plastic components in the following form.

$$d\varepsilon_v = d\varepsilon_v^e + d\varepsilon_v^p \quad (2.45)$$

$$d\varepsilon_q = d\varepsilon_q^e + d\varepsilon_q^p \quad (2.46)$$

where

$d\varepsilon_v^e$ = increment of elastic volumetric strain

$d\varepsilon_v^p$ = increment of plastic volumetric strain

$d\varepsilon_q^e$ = increment of elastic deviatoric strain

$d\varepsilon_q^p$ = increment of plastic deviatoric strain

The elastic components of strain increment are determined from the following.

$$d\epsilon_v^e = \frac{dp}{B} \quad (2.47)$$

$$d\epsilon_q^e = \frac{dq}{3G} \quad (2.48)$$

The plastic components of the strain increments are determined as

$$d\epsilon_v^p = < \frac{1}{H} (\eta_p dp + \eta_q dq) > \eta_p \quad (2.49)$$

$$d\epsilon_q^p = < \frac{1}{H} (\eta_p dp + \eta_q dq) > \eta_q \quad (2.50)$$

where

$$H = H_b + \frac{1+e_o}{\lambda-\kappa} \frac{\delta}{\delta_{\max}-\delta} h_o p < \frac{M_p}{M_c} - |x| > \quad (2.51)$$

$$H_b = \frac{1+e_o}{\lambda-\kappa} \frac{A}{g^2} M^2(\gamma-1)[(\gamma + \rho(\rho-2))] \quad (2.52)$$

$$\eta_p = \frac{M(\gamma-1)}{g} \quad (2.53)$$

$$\eta_q = \frac{(\rho-1)^2 \gamma x}{g} \quad (2.54)$$

$$g = \sqrt{M^2(\gamma-1)^2 + (\rho-1)^4 \gamma^2 x^2} \quad (2.55)$$

$$\delta = (\gamma A - p) \sqrt{1+n^2} \quad (2.56)$$

$$\delta_{\max} = \gamma A \sqrt{1+n^2} \quad (2.57)$$

$$n = \frac{q}{p} \quad (2.58)$$

$$\gamma = \frac{1+(\rho-1) \sqrt{1+x^2 \rho(\rho-2)}}{1+(\rho-1)^2 x^2} \quad (2.59)$$

$$x = \frac{n}{M} \quad (2.60)$$

where M takes the value M_c for triaxial compression paths and M_e for triaxial extension paths.

CHAPTER THREE

MODEL IMPLEMENTATION AND VERIFICATION

3.1 Introduction

The model formulation in differential form is presented in Chapter 2 for a simplified triaxial state of stress as well as for the generalized state of stress conditions. In this chapter, the constitutive equations are written explicitly in matrix form. First, triaxial drained and undrained tests are considered with the independent variable being either stress or strain. Subsequently, the constitutive relations in matrix form are developed for generalized, three-dimensional stress states. The equations in matrix form are used to incorporate the bounding surface plasticity model into a finite element program, SAC (Soil Analysis Code).

Verification of implementation of the model in the finite element program is carried out. Finite element calculations are made for soil behavior in different laboratory tests and compared with results of calculations made by integrating the constitutive equations numerically. Four test conditions are considered in the verification process. These are, monotonic drained, monotonic undrained, cyclic drained, and cyclic undrained cases.

Numerical problems associated with extreme softening of soil (very low values of bulk modulus) in cases of low mean pressure are observed. To overcome this difficulty, modification is made in the expression determining the elastic bulk modulus. Three alternatives are presented. These are achieved by either introducing a transitional pressure in the formulation, using a non-linear relation between the bulk modulus and

the mean pressure, or by using a constant value of the Young modulus representative to the range of mean pressure under consideration.

3.2 Constitutive Matrix for Stress States in Triaxial Plane

In the following, the constitutive equations are reduced to the formulation in the triaxial plane of stresses. By doing so, conventional and non-conventional triaxial tests can be simulated under drained as well as undrained loading conditions.

The constitutive matrices are developed for four different testing conditions. These are stress-controlled drained test, strain-controlled drained test, stress-controlled undrained test, and strain-controlled undrained test. In the stress-controlled tests, the deviator stress is considered to represent the independent variable. In the strain controlled tests, the axial strain is taken to be the independent variable. All the other variables are expressed in terms of the independent variable in both cases. In all of these cases, the notation and symbols of Chapter 2 are used.

3.2.1 Stress-Controlled Drained Test

The general relation for drained and undrained stress controlled path is

$$\begin{bmatrix} d\epsilon_q \\ d\epsilon_v \end{bmatrix} = \begin{bmatrix} C_1 & C_2 \\ C_2 & C_3 \end{bmatrix} \begin{bmatrix} dq \\ dp \end{bmatrix} \quad (3.1)$$

where,

$$C_1 = \frac{1}{3G} + \frac{1}{H} \eta_q^2 \quad (3.2)$$

$$C_2 = \frac{1}{H} \eta_p \eta_q \quad (3.3)$$

$$C_3 = \frac{1}{B} + \frac{1}{H} \eta_p^2 \quad (3.4)$$

where H , G , B , η_p , and η_q are as defined in Chapter 2.

In this case the independent variable is the deviator stress, dq . The increments of mean pressure, dp , deviatoric strain, $d\epsilon_q$, and volumetric strain, $d\epsilon_v$, are to be expressed in terms of the increments of independent variable.

For a constant confining pressure ($d\sigma_3=0$), the following can be written

$$d\sigma_1 = dq \quad (3.5)$$

and

$$dp = \frac{dq}{3} \quad (3.6)$$

substituting Eq. (3.6) into Eq. (3.1) gives

$$d\epsilon_q = \left[C_1 + \frac{C_2}{3} \right] dq \quad (3.7)$$

and

$$d\varepsilon_v = \left[C_2 + \frac{C_3}{3} \right] dq \quad (3.8)$$

and increments of the radial strain, $d\varepsilon_3$, and axial strain, $d\varepsilon_1$, can be found from

$$d\varepsilon_3 = \left[\frac{3 C_3 + 2 C_2 - 9 C_1}{18} \right] dq \quad (3.9)$$

and

$$d\varepsilon_1 = \left[\frac{C_3 + 6 C_2 + 9 C_1}{9} \right] dq \quad (3.10)$$

3.2.2 Strain-Controlled Drained Test

The general relation for drained and undrained strain controlled test is

$$\begin{bmatrix} dq \\ dp \end{bmatrix} = \begin{bmatrix} D_1 & D_2 \\ D_2 & D_3 \end{bmatrix} \begin{bmatrix} d\varepsilon_q \\ d\varepsilon_v \end{bmatrix} \quad (3.11)$$

where,

$$D_1 = 3G \left[1 - \frac{3G}{H} \frac{\eta_q^2}{\eta_p^2 \left[1 + \frac{B}{H} \right] + \eta_q^2 \left[1 + \frac{3G}{H} \right]} \right] \quad (3.12)$$

$$D_2 = -3G \frac{B}{H} \left(\frac{\eta_q \eta_p}{\eta_p^2 \left[1 + \frac{B}{H} \right] + \eta_q^2 \left[1 + \frac{3G}{H} \right]} \right) \quad (3.13)$$

$$D_3 = B \left[1 - \frac{B}{H} \frac{\eta_p^2}{\eta_p^2 \left[1 + \frac{B}{H} \right] + \eta_q^2 \left[1 + \frac{3G}{H} \right]} \right] \quad (3.14)$$

In this case the independent variable is the axial strain, ε_1 . The increments of mean pressure, dp , deviatoric stress, dq , radial strain, $d\varepsilon_3$, and volumetric strain, $d\varepsilon_v$, are to be expressed in terms of the increments of independent variable.

From Eq. (3.11) we can write

$$d\sigma_1 = \left[\frac{2D_1}{3} + D_2 \right] d\varepsilon_1 + \left[2D_2 - \frac{2D_1}{3} \right] d\varepsilon_3 \quad (3.15)$$

and

$$d\sigma_1 = (2D_2 + 3D_3) d\varepsilon_1 + (6D_3 - 2D_2) d\varepsilon_3 \quad (3.16)$$

Combining Eqs. (3.15) and (3.16) gives

$$d\varepsilon_3 = \left[\frac{6D_3 - 4D_2 + \frac{2D_1}{3}}{\frac{2D_1}{3} - D_2 - 3D_3} \right] d\varepsilon_1 \quad (3.17)$$

and

$$d\sigma_1 = \left[2D_2 + 3D_3 + (6D_3 - 2D_2) \left[\frac{6D_3 - 4D_2 + \frac{2D_1}{3}}{\frac{2D_1}{3} - D_2 - 3D_3} \right] \right] d\varepsilon_1 \quad (3.18)$$

The volumetric strain, $d\varepsilon_v$, is

$$d\varepsilon_v = \left[1 + \left[\frac{12D_3 - 8D_2 + \frac{4D_1}{3}}{\frac{2D_1}{3} - D_2 - 3D_3} \right] \right] d\varepsilon_1 \quad (3.19)$$

3.2.3 Stress-Controlled Undrained Test

The independent variable in this test is the deviator stress. The axial strain, radial strain, and the excess pore water pressure are to be written in terms of the deviator stress.

Since the volumetric strain is equal to zero, $d\varepsilon_v=0$, then the relations of Eq. (3.1) become

$$d\varepsilon_q = C_1 dq + C_2 dp \quad (3.20)$$

and

$$0 = C_2 dq + C_3 dp \quad (3.21)$$

From Eq. (3.21)

$$dp = -\frac{C_2}{C_3} dq \quad (3.22)$$

substitute (3.22) into (3.20)

$$d\varepsilon_q = \left[C_1 - \frac{C_2^2}{C_3} \right] dq \quad (3.23)$$

To determine $d\varepsilon_1$, and $d\varepsilon_3$

$$d\varepsilon_v = 0 = d\varepsilon_1 + 2 d\varepsilon_3 \quad (3.24)$$

and

$$d\varepsilon_q = \frac{2}{3} (d\varepsilon_1 - d\varepsilon_3) \quad (3.25)$$

then

$$d\varepsilon_1 = d\varepsilon_q \quad (3.26)$$

and

$$d\varepsilon_3 = -\frac{d\varepsilon_q}{2} \quad (3.27)$$

At this stage, $d\varepsilon_q$, $d\varepsilon_1$, $d\varepsilon_3$, and dp are known quantities in terms of the increment of independent variable, dq . The increment of the excess pore water pressure is determined as follows:

Using the effective stress concept, we can write

$$dq = (d\sigma_1' - du) - (d\sigma_3' - du) \quad (3.28)$$

or

$$dq = d\sigma_1' - d\sigma_3' \quad (3.29)$$

and

$$dp = \frac{(d\sigma_1' - du) + 2(d\sigma_3' - du)}{3} \quad (3.30)$$

or

$$dp = \left[\frac{d\sigma_1' + 2 d\sigma_3'}{3} \right] - du \quad (3.31)$$

where

$d\sigma_1^t$ = total axial stress

$d\sigma_3^t$ = total radial stress

If the total confining pressure is held constant throughout the test, as in the conventional triaxial undrained test, then:

$$d\sigma_3^t = 0 \quad (3.32)$$

and Eqs. (3.29) and (3.31) can be rewritten as

$$d\sigma_1^t = dq \quad (3.33)$$

and

$$dp = \frac{d\sigma_1^t}{3} - du \quad (3.34)$$

Joining Eqs. (3.33) and (3.34) yields to

$$dp = \frac{dq}{3} - du \quad (3.35)$$

or

$$du = \left[\frac{1}{3} + \frac{C_2}{C_3} \right] dq \quad (3.36)$$

3.2.4 Strain-Controlled Undrained Test

The independent variable in this test is the axial strain. The radial strain, deviatoric stress, mean pressure, and the excess pore water pressure are to be written in terms of the axial strain.

Since the volumetric strain is equal to zero, $d\varepsilon_v=0$, the relations in Eq. (3.11) become

$$dq = D_1 d\varepsilon_q \quad (3.37)$$

and

$$dp = D_2 d\varepsilon_q \quad (3.38)$$

also

$$d\varepsilon_3 = -\frac{d\varepsilon_1}{2} \quad (3.39)$$

and

$$d\varepsilon_q = d\varepsilon_1 \quad (3.40)$$

By substituting Eq. (3.40) into Eqs. (3.37) and (3.38), the following equations are obtained.

$$dq = D_1 d\varepsilon_1 \quad (3.41)$$

and

$$dp = D_2 d\varepsilon_1 \quad (3.42)$$

At this stage, dp , dq , and $d\varepsilon_3$ are expressed in terms of the incremental variable $d\varepsilon_1$.

Finally the values of dp , and dq are substituted into Eq. (3.35) to obtain the increment of the excess pore water pressure as follows

$$du = \left[\frac{D_1}{3} - D_2 \right] d\varepsilon_1 \quad (3.43)$$

3.2.5 Step by Step Integration of Constitutive Equations

In order to use the model for simulation of material behavior, the above constitutive equations should be integrated following the desired stress or strain path. Such an integration is referred to as the step by step integration. Several numerical methods are available to achieve this purpose.

The accuracy of the numerical integration is of prime concern. However, the relative computer time required to perform this integration (integration efficiency) with sufficient accuracy is not a major concern at this stage. In performing the numerical integration, the independent variable, deviatoric stress or axial strain, was applied in very small increments such that further reduction in the increment size produced no changes in the calculated responses.

The numerical procedure for the case of strain controlled test, for instance, required that the axial strain should be applied in increments in the range of 0.0005 and 0.001. Similar values of strain increments were used in previous studies. Bardet (1983) used

increments of 0.001 in the numerical integration process he performed using a bounding surface plasticity model. Faruque and Desai (1985) used strain increments of 0.0005 in their numerical integration of a modified cap model. They called the process of dividing the strain increments into very small subincrements during the numerical integration as subincrementation technique. This technique was also used in the finite element program by Faruque and Desai (1985).

The above technique for performing numerical integration is quite simple and useful as a bench mark to test the correctness of implementation of more advanced numerical integration methods. However, the technique is not the best available to be incorporated into finite element analysis. For instance, it was observed in this study and by other researchers (Herrmann et al., 1986), that the size of strain increment is not the only factor controlling the accuracy of a numerical integration over a solution step. States in the vicinity of the hydrostatic state of stress and those approaching the region of the critical state required much smaller strain increments to achieve numerical integration with acceptable accuracy. For these reasons, a more elaborate and efficient numerical integration method was applied to the generalized states as shown in section 3.3.

3.3 Constitutive Matrix for Generalized, Three Dimensional Stress States

The relation between the effective stress and strain in differential form can be written using tensor notation (Dafalias, 1986) as follows:

$$d\sigma_{ij} = D_{ijkl} d\epsilon_{kl} \quad (3.44)$$

where D_{ijkl} is a fourth order tensor defined as follows

$$D_{ijkl} = P_1 - P_2 * P_3 * \langle P_5 \rangle / P_4 \quad (3.45)$$

The symbol $\langle \rangle$ represents a function defined in Equation 2.16. P_1 , P_2 , P_3 , P_4 , and P_5 appearing in Eq. (3.45) can be calculated using the following equations.

$$P_1 = G (\delta_{ki} \delta_{lj} + \delta_{kj} \delta_{li}) + (B - \frac{2}{3} G) \delta_{ij} \delta_{kl} \quad (3.46)$$

$$P_2 = 3B(f_{,l})\delta_{ij} + \frac{G}{J} (f_{,l})s_{ij} + \frac{\sqrt{3}G}{J\cos 3\alpha} (3\xi(f_{,l})) \left[\frac{s_{in}s_{nj}}{J^2} - \frac{3S^3s_{ij}}{2J^4} - \frac{2\delta_{ij}}{3} \right] \quad (3.47)$$

$$P_3 = 3B(f_{,l})\delta_{kl} + \frac{G}{J} (f_{,l})s_{kl} + \frac{\sqrt{3}G}{J\cos 3\alpha} (3\xi(f_{,l})) \left[\frac{s_{kn}s_{nl}}{J^2} - \frac{3S^3s_{kl}}{J^4} - \frac{2\delta_{kl}}{3} \right] \quad (3.48)$$

$$P_4 = H + 9B(f_{,l})^2 + (G + 9\xi^2 \frac{G}{J^2})(f_{,l})^2 \quad (3.49)$$

$$P_5 = \frac{1}{P_4} \left\{ 3B(f_{,l})d\varepsilon_{kk} + \frac{G}{J}(f_{,l})s_{ij}d\varepsilon_{ij} + \frac{\sqrt{3}G}{J\cos 3\alpha} (3\xi(f_{,l})) \left[\left(\frac{s_{ik}s_{kj}}{J^2} - \frac{3S^3s_{ij}}{2J^4} \right) d\varepsilon_{ij} - \frac{2\varepsilon_{kk}}{3} \right] \right\} \quad (3.50)$$

Definitions of the terms appearing in these equations are already given in Chapter 2.

In a matrix form the relations given in Eq. (3.44) can be expressed as

$$\{d\sigma\} = [D] \{d\varepsilon\} \quad (3.51)$$

where

$$d\sigma = \begin{pmatrix} d\sigma_x \\ d\sigma_y \\ d\sigma_z \\ d\tau_{xy} \\ d\tau_{xz} \\ d\tau_{yz} \end{pmatrix} \quad (3.52)$$

and

$$d\varepsilon = \begin{pmatrix} d\varepsilon_x \\ d\varepsilon_y \\ d\varepsilon_z \\ d\gamma_{xy} \\ d\gamma_{xz} \\ d\gamma_{yz} \end{pmatrix} \quad (3.53)$$

in which

$$d\gamma_{ij} = 2 d\varepsilon_{ij} \quad (3.54)$$

The elements of $[D]_{6 \times 6}$ matrix are related to the elements of the $3 \times 3 \times 3 \times 3$ D_{ijkl} tensor by the following relations (Herrmann et al. 1982):

$$D_{11} = D_{1111} \quad (3.55)$$

$$D_{12} = D_{1122} \quad (3.56)$$

$$D_{13} = D_{1133} \quad (3.57)$$

$$D_{14} = D_{1112} \quad (3.58)$$

$$D_{15} = D_{1113} \quad (3.59)$$

$$D_{16} = D_{1123} \quad (3.60)$$

$$D_{22} = D_{2222} \quad (3.61)$$

$$D_{23} = D_{2233} \quad (3.62)$$

$$D_{24} = D_{2212} \quad (3.63)$$

$$D_{25} = D_{2213} \quad (3.64)$$

$$D_{26} = D_{2223} \quad (3.65)$$

$$D_{33} = D_{3333} \quad (3.66)$$

$$D_{34} = D_{3312} \quad (3.67)$$

$$D_{35} = D_{3313} \quad (3.68)$$

$$D_{36} = D_{3323} \quad (3.69)$$

$$D_{44} = D_{1212} \quad (3.70)$$

$$D_{45} = D_{1213} \quad (3.71)$$

$$D_{46} = D_{1223} \quad (3.72)$$

$$D_{55} = D_{1313} \quad (3.73)$$

$$D_{56} = D_{1323} \quad (3.74)$$

$$D_{66} = D_{2323} \quad (3.75)$$

The differential form of the constitutive relations expressed in Eq. (3.51) can be written in an incremental form suitable for use in the finite element program as below

$$\{\Delta\sigma\} = [D_{avg}] \{\Delta\varepsilon\} \quad (3.76)$$

where

$[D_{avg}]$ is the average value of $[D]$ over the solution increment Δt_N , as in the following relation (Herrmann et al., 1986)

$$[D_{avg}] = \frac{1}{\Delta t_N} \int_{t_{N-1}}^{t_N} [D] dt \quad (3.77)$$

The time in the above equation, t , is introduced in the formulation even though the model is rate independent. The time scale is divided into a number of finite (solution) steps. For time independent situations, as the case in the present study, the time has no units.

Several possibilities are available to evaluate the average value of $[D]$ over the solution increment. The trapezoidal rule is one of these methods. If the solution increment treated as a one step, then $[D_{avg}]$ can be determined from

$$[D_{avg}] = \frac{1}{2\Delta t} \{[D]_{N-1} + [D]_N\} \quad (3.78)$$

The one step trapezoidal rule was used by Herrmann et al. (1982). It is also used in the early stages of the present work. The results, however, were not encouraging and confirmed the observation made by Herrmann et al. (1986) that the one step trapezoidal rule is not adequate, for some states, to provide accurate numerical integration even for very small solution steps.

On the other hand, if the solution increment is subdivided into M equal substeps (subincrementation or multistep integration), then $[D_{avg}]$ can be determined from the following

$$[D_{avg}] = \frac{1}{M} \sum_{m=1}^M [D_{avg}]_m \quad (3.79)$$

in which

$$[D_{avg}]_m \approx \frac{1}{2\Delta t_m} \{[D]_{m-1} + [D]_m\} \quad (3.80)$$

where

$[D]_{m-1}$ = the value of $[D]$ corresponding to the stress and strain states at the beginning of a substep.

$[D]_m$ = the value of $[D]$ corresponding to the stress and strain states at the end of a substep.

For increment N , the strain at time t_m for substeps of equal length is

$$\{\varepsilon\}_m = \{\varepsilon\}_{N-1} + \frac{m}{M} \{\Delta\varepsilon_N\} \quad (3.81)$$

and stress estimate at the corresponding time is initially taken as

$$\{\sigma\}_m \approx \{\sigma\}_{m-1} + \{\Delta\sigma\}_{m-1} = \{\sigma\}_{N-1} + \sum_{i=1}^{m-1} \{\Delta\sigma\}_i + \{\Delta\sigma\}_{m-1} \quad (3.82)$$

where

$$\Delta t_m = \frac{\Delta t_N}{M} \quad (3.83)$$

$$t_{m-1} = t_{N-1} + (m-1)\Delta t_m \quad (3.84)$$

$$t_m = t_{m-1} + \Delta t_m \quad (3.85)$$

The multistep integration technique was used here to overcome the difficulties faced in performing numerical integration as mentioned in Section 3.2.5 and this section, and the demonstrated success of this technique when applied for other constitutive models (Herrmann et al., 1986).

For predicted stress states laying outside the bounding surface the classical return procedures (Hughes, 1983; Herrmann et al., 1986) were used to return the stress state to the surface. In this procedure, a stress state outside the bounding surface is scaled back along the line connecting the stress origin to the state. The scaled invariant of stress are

$$I_{\text{scaled}} = b I \quad (3.86)$$

$$J_{\text{scaled}} = b J \quad (3.87)$$

and

$$S_{\text{scaled}} = b S \quad (3.88)$$

where b is a scalar value. It represents the relative location of a stress state with respect to its corresponding image point.

3.4 Simulation of Pore Water Pressure Generation

The stress-strain relations presented so far were expressed in terms of effective stresses and strains. The incorporation of the total stresses in the analysis requires the simulation of the generation of pore water pressure. This was treated as follows (Herrmann, 1965; Sangrey, et al. 1969; Zienkiewicz, 1977; Zienkiewicz et al., 1981; and Herrmann et al., 1986):

The change in total stress $\Delta\sigma'_{ij}$ is the sum of the changes in effective stress and pore water pressure

$$\{\Delta\sigma\}' = \{\Delta\sigma\} + \Delta u \{1\} \quad (3.89)$$

where $\{1\}$ is the unit vector, its transpose is

$$\{1\}^T = \{1, 1, 1, 0, 0, 0\} \quad (3.90)$$

For drained conditions, excess pore pressure is equal to zero, $\Delta u = 0$, and total stress equals to effective stress, $\{\sigma\}' = \{\sigma\}$, and all the stress-strain relations presented above can be taken to represent total stress-strain relations. On the other hand, for the undrained loading conditions, excess pore water pressure should be evaluated in order to determine total stresses using Eq. (3.89).

The method used in Section 2 of this chapter to determine the pore pressure required separate formulations for drained and undrained conditions. For undrained conditions, the determination of pore water pressure was achieved by imposing a zero volume change condition on the stress-strain equations. This is equivalent to assume that both soil particles and pore water are completely incompressible (have a combined bulk modulus equal to infinity). This approach is simple and useful for the kind of cases considered in Section 2, however, a more generalized method is needed for the finite element analysis in order to have a unified formulation for drained and undrained

conditions. This was achieved by considering an extremely small compressibility for the pore water and soil particles (Sangrey et al. 1969; Herrmann et al. 1986). Using Γ as the combined bulk modulus for the soil particles and pore water, the pore water pressure, u , resulted from a volume change, ε_{kk} , can be written as

$$\Delta u = \Gamma \Delta \varepsilon_{kk} \quad (3.91)$$

Equation 3.91 is valid for drained and undrained conditions. For drained condition, Γ takes the value of zero and the resulted pore water pressure becomes zero for any value of volume change. For undrained conditions, Γ approaches infinity and the soil becomes incompressible and the pore water pressure is determined as a function of the very small volume change.

The magnitude of calculated pore water pressure depends on the value of Γ used in the analysis. Higher pore water pressure results by using higher values of Γ . However, for ideal undrained conditions, values of Γ beyond 10 GPa are observed not to cause further increase in the calculated pore water pressure. This means that a value of Γ of about 10 GPa can be used in practical analysis of ideal undrained situations. Finn (1988) warned from possible numerical problems due to the use of high values of Γ . No numerical problems are observed in this study when using values of Γ as high as 100 GPa in the analysis.

Using the reduced integration procedures (Zienkiewicz, 1977; Herrmann et al., 1986) for the element matrices, the change in total stress can be expressed as

$$\{\Delta \sigma\}^t = ([D_{avg}] + [d]) \{\Delta \varepsilon\} + \{\Delta \sigma_o\} \quad (3.92)$$

where

$$[d] = \begin{pmatrix} \Gamma & \Gamma & \Gamma & 0 & 0 & 0 \\ \Gamma & \Gamma & \Gamma & 0 & 0 & 0 \\ \Gamma & \Gamma & \Gamma & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{pmatrix} \quad (3.93)$$

3.5 Numerical Problem

In the elastic constitutive relation described in Chapter 2 (Eqs. 2.10 through 2.13), the bulk modulus was assumed to be proportional to the first stress invariant, Eq. 2.13, and the shear modulus is related to the bulk modulus using Poisson's ratio which is taken as a constant, Eq. 2.14.

Numerical problems associated with extreme softening of the soil (very low values of bulk modulus) in cases of low mean pressure ($I/3$) were observed due to the use of the bulk modulus relation as given in Eq. 2.13. According to this relation, which has been used in several models based on critical state soil mechanics, consolidation curves in $e-\ln(I/3)$ plane should have linear unloading-reloading portion for all mean stress ($I/3$) levels. However, experiments show that there exists a transitional stress below which the unloading-reloading portion of a consolidation curve changes from linear in $e-\ln(I/3)$ to linear in $e-(I/3)$ plane (Dafalias, 1986). Furthermore, other constitutive models that use the relation of Eq. 2.13 also suffer from the numerical problems observed in this study. One of these constitutive models is Dafalias model for clay soils (Dafalias, 1987).

To overcome this difficulty, modification was required in the expression determining the elastic bulk modulus. This was achieved by using one of the following alternatives.

The first alternative is to modify the pressure dependency of the bulk modulus to incorporate the effect of the transitional pressure (Dafalias and Herrmann, 1980). The equation for the bulk modulus (Eq. 2.13) was replaced by the following relation

$$B = \frac{1+e_0}{3} (<I - I_L> + I_L) \quad (3.94)$$

where I_L is the transitional stress. The value of I_L , however, is not always well defined from experimental data due to the use of relatively high stress with laboratory experiments. A value of I_L equal to the atmospheric pressure was suggested and used by Dafalias (1987). This alternative was incorporated in the Bardet model formulation as an option. The attractive aspect of this alternative is that it does not add additional model parameter to the nine parameters already required by the model. The selection of I_L equal to the atmospheric pressure was not based on sufficient experimental observations, however.

The second alternative is to express the dependency of the Young modulus on the mean pressure in a form similar to that developed by Duncan and Chang (1970) and used by Lade (1975) and others. The same approach was also used by Bardet (1988) in the formulation of a simple bounding surface plasticity model.

According to Duncan and Chang (1970) the unloading reloading modulus is given by

$$E_{ur} = K_{ur} P_a \left(\frac{\sigma_3}{P_a} \right)^n \quad (3.95)$$

where

E_{ur} = unloading-reloading modulus

K_{ur} = modulus number

n = exponent number

σ_3 = confining pressure

P_a = atmospheric pressure

The above expression insures that even for relatively small confining pressures, the unloading reloading modulus (bulk modulus and shear modulus also) does not approach zero.

The following similar expression is used herein to express Young modulus dependency on the mean pressure, $I/3$.

$$E = E_0 P_a \left(\frac{I/3}{P_a} \right)^n \quad (3.96)$$

In this expression, E_0 and n are two parameters to be determined from series of triaxial test carried out using different initial pressures by fitting a straight line to the test results plotted in terms of $\log (E/P_a)$ versus $\log((I/3)/P_a)$. The value of E at a mean pressure $(I/3)$ equal to the atmospheric pressure is equal to E_0 . The exponent n is equal to the slope of the straight line.

The bulk modulus and shear modulus are related to the Young modulus using the following expressions. Poison's ratio was assumed to remain constant as in the original formulation of the model,

$$B = \frac{E}{3(1-2\nu)} \quad (3.97)$$

$$G = \frac{E}{2(1+\nu)} \quad (3.98)$$

The use of this alternative to express the dependency of the bulk modulus and the shear modulus on the mean pressure adds two additional model parameters. However, the method of determination of these two parameters from experimental data is well established.

As a special case of the above alternative is to use a constant value of the Young modulus (Desai and Siriwardane, 1984). However, the value should be representative to the range of mean pressure encountered in the problem considered.

All the three above alternatives are used in this study. The first and third alternatives are found to be more attractive due to that no additional model parameters are required in using them.

3.6 Finite Element Program

The finite element program on which the Bardet bounding surface plasticity model was incorporated is called SAC (Soil Analysis Code), Herrmann and Kaliakin (1987). This program is a displacement based two dimensional (plane strain, plane stress, and axially symmetric) quasi-static finite element program. The program was written in a modular form by using the FORTRAN 77 language.

To account for the nonlinearity and path dependency of elasto-plastic constitutive models, the global nonlinear solution scheme adopted in the program is incremental. In order to avoid using extremely small increments, iterations are carried out within each increment. The resulted solution technique is therefore incremental iterative. The tangent stiffness method as well as the successive substitution method (Owen and Hinton, 1980; Herrmann et al., 1982) are both incorporated as solution options.

3.7 Verification of Model Implementation into Finite Element Program

3.7.1 General

Verification of the finite element implementation of the model was carried out in the following way: Finite element calculations were made for soil behavior in different laboratory tests. The results of finite element calculation were compared with results of calculations made by integrating the constitutive equations numerically as introduced in Section 3.2 using the same stress or strain paths.

The four test conditions considered in this verification process included monotonic drained case, monotonic undrained case, cyclic drained case, and cyclic undrained case.

The model parameters used in the verification process were obtained from the laboratory tests performed on 1) the Sacramento River sand by Lee and Seed (1967), and Seed and Lee (1967), and 2) the Fuji River sand by Ishihara et al. (1975), and Tatsuoka and Ishihara (1974).

All the finite element calculations were performed using axially symmetric idealization with a one element mesh. Fig. 3.1 shows this mesh with the boundary conditions used in the analysis.

3.7.2 Verification Case 1

Verification Case 1 used the results of a drained conventional triaxial compression test on a dense Sacramento River sand having an initial void ratio of 0.61 (density index = 100%). The test was reported by Lee and Seed (1967). The sample was first consolidated using a 300 kPa effective confining pressure and then sheared in drained condition by increasing the axial strain while the lateral stress was held constant throughout the test.

Fig. 3.2 shows the calculated deviator stress versus axial strain. The solid line represents the numerical integration calculations and the circular symbols represent the results of the finite element analysis. The calculations of the volume change versus the axial strain of the same test are presented in Fig. 3.3. In the numerical integration calculations, the incremental variable was the axial strain and the incremental relations of Section 3.2.2 were used. In the finite element analysis, ten equal increments of axial displacement (each increment represents 1% axial strain) were applied at Nodes 3 and 4 (Fig. 3.1). Both the numerical integration calculations and the finite element analysis gave identical results for the full range of axial strain used in this comparison.

3.7.3 Verification Case 2

The results of an undrained conventional triaxial compression test on a loose Sacramento River sand of an initial void ratio of 0.87 (density index = 38%) was used in this case. The test was carried out by Seed and Lee (1967). The sample was first consolidated using a 300 kPa cell pressure and then sheared in undrained condition by increasing the axial strain while the cell pressure was held constant throughout the test.

The numerical integration calculations of the excess pore water pressure versus axial strain using the relations of Section 3.2.4 and the finite element results are shown in Fig. 3.4. The calculated effective stress path in the deviator stress-mean pressure plane is shown along with the finite element results in Fig. 3.5. The rapid build-up of the pore water pressure, which was also observed in the actual test, required the use of very small axial strain increments (0.05%) specially at the initial stages of the calculation (about 1.5% axial strain) to ensure convergence in the numerical integration. In the finite element analysis, similar to the analysis in Verification Case 1, axial displacements were applied in increments. The loading history was divided into two parts. In the first part, four increments of axial displacement (each increment represents 0.5% axial strain) were applied. In the second part, eight increments of axial displacement (each increment represents 1% axial strain) were applied. The results of the numerical integration calculations and the finite element analysis were also identical for this case.

3.7.4 Verification Case 3

The data used in Verification Case 3 were from a two-way cyclic drained triaxial test on a loose Fuji River sand of an initial void ratio of 0.77. The test was carried out by Tatsuoka and Ishihara (1974). In this test, the sample was first consolidated using a 200 kPa cell pressure. The cell pressure was held constant while the axial stress was cycled two ways with increasing amplitude at each cycle.

The results of the finite element analysis and the numerical integration using the relations 3.2.1 of this test are shown in Fig. 3.6. The volumetric strain changes are

plotted against the stress ratio which is defined as the ratio between the deviator stress and the mean effective pressure. A total of 48 load increments were used in the finite element analysis. The magnitude of the incremental variable (the axial stress) was not constant during loading. A constant number of load increments was employed in each cycle. Identical results, as shown in Fig. 3.6, were obtained from the finite element analysis and the numerical integration process.

3.7.5 Verification Case 4

Verification Case 4 used the data from a two-way cyclic undrained triaxial test on a loose Fuji River sand of an initial void ratio of 0.77. The test was carried out by Ishihara et al. (1975). In this test, the sample was first consolidated using a 220 kPa cell pressure. The cell pressure was held constant while the axial stress was cycled between two limits (272 kPa and 128 kPa). Drainage was prevented in this test.

The results are presented in the deviator stress versus mean effective pressure plane as shown in Fig. 3.7. A total of 99 load increments were used in the finite element analysis. The magnitude of the incremental variable (the axial stress) was kept constant during load increments. This led to a constant number of increments per load cycle since the deviator stress limits were kept constant. The finite element results and the results obtained by the numerical integration process using the relations of Section 3.2.3 were identical.

The above four cases of comparisons between the results of numerical integration of the constitutive equations and the finite element analysis demonstrate that the constitutive model is correctly implemented in the finite element program.

CHAPTER FOUR

MODEL PARAMETERS AND SENSITIVITY ANALYSIS

4.1 Introduction

There are nine parameters required by the present constitutive model in order to describe the behavior of a soil. This chapter describes the nine model parameters and their methods of determination. Further, the model sensitivity with respect to variation in values of the parameters is presented. The study of the model sensitivity provided a feeling for the relative importance of each one of the model parameters in the overall performance of the model during the process of its calibration.

Before using this constitutive model in the simulation of the material behavior, the initial state of the soil should be defined. Initial effective stresses (σ_{xx} , σ_{yy} , σ_{zz} , τ_{xy} , τ_{xz} , and τ_{yz}), porewater pressure (u_0), and void ratio (e_0), are required to define the initial state of a soil element.

4.2 Unloading Reloading Modulus, κ

To represent the slope of the unloading reloading portion of isotropic compression test results an unloading-reloading modulus, κ , is used. The results of an isotropic compression test should be plotted on e - $\ln(p)$ plane, where the parameter e represents the current void ratio and p is the mean pressure. Fig. 4.1 presents a sample of such plot. One isotropic compression test with an unloading and reloading branch is sufficient to determine this parameter. If the swelling index, C_r , is available from the

analysis of one dimensional compression test results, then κ can be determined from C_r as follows:

$$\kappa = \frac{C_r}{2.303} \quad (4.1)$$

4.3 Slope of Critical State Line in e-ln(p) Plane, λ

The slope of the critical state line in e-ln(p) plane is assumed to be the same as the slope of the virgin compression portion of isotropic compression test results. As for κ , one isotropic compression test is required to determine λ . Fig. 4.1 shows a sample of test data evaluated to obtain λ . If the compression index C_c is available from one dimensional compression test results, then λ can be determined from C_c as follows:

$$\lambda = \frac{C_c}{2.303} \quad (4.2)$$

4.4 Critical Void Ratio at Unit Mean Effective Pressure, Γ

Critical void ratio is the void ratio corresponding to unit mean pressure along the critical state line in the e-ln(p) plane. The determination of this parameter requires the determination of the projection of the critical state line (slope and position) on the e-ln(p) plane. The slope of this line is equal to λ and the position can be determined from the results of drained conventional triaxial compression tests plotted in the e-ln(p) plane. Ordinarily, three tests are sufficient to establish the location of the critical state line. For loose sands, one such test "might" be sufficient to establish the position of the critical state line if the triaxial compression test is carried out to a large axial strain.

4.5 Ultimate Friction Angle in Compression, ϕ_c

The post peak (ultimate) angle of friction in degrees can be determined from conventional drained triaxial compression tests. The same test results used in determining the location of the critical state line in the e-ln(p) is usually used to

determine this angle of friction. In the model formulation presented in this chapter, the parameter M_c was used instead of ϕ_c . The correlation between M_c and ϕ_c was given in Eq. (2.19).

4.6 Ultimate Friction Angle in Extension, ϕ_e

The post peak (ultimate) angle of friction in degrees can be determined from conventional drained triaxial extension tests. Two to three triaxial extension tests are ordinarily sufficient to obtain this angle of friction. In the model formulation presented in Chapter 2, the parameter M_e was used instead of ϕ_e . The correlation between M_e and ϕ_e was given in Eq. (2.20). If test results are not available to determine ϕ_e , a value of 0.8-1.0 can be assumed for the ratio M_e/M_c as a reasonable estimate.

4.7 Peak Failure Friction Angle, ϕ_p

The peak failure friction angle corresponds the peak stress at failure. It is determined, conventionally, from the Mohr circle relations. Its value should be equal or greater than the ultimate friction angles in compression and in extension. A conventional triaxial drained compression test at relatively low confining pressure is sufficient to determine this friction angle. In the model formulation presented in this chapter, the model parameter, M_p , was used instead of using the peak failure friction angle. The correlation between the model parameter M_p and the peak failure friction angle was given in Eq. (2.37).

4.8 Poisson's ratio, ν

The Poisson's ratio, ν , is assumed to take a typical value in the range of 0.2 and 0.4. Its effect on the simulations of the overall material response is small.

4.9 Bounding Surface Aspect Ratio, ρ

The bounding surface (ellipse) aspect ratio affects the plastic flow direction. The determination of this parameter requires a plot of the test results in terms of the plastic flow direction represented by $d\varepsilon_p^q/d\varepsilon_p^p$ versus the stress level represented by the ratio q/p . Model simulations with different values of ρ should be carried out and the results of this simulations compared with experimental values to chose a reasonable value for the aspect ratio.

In plotting the experimental results, the measured volumetric and deviatoric total strains can be assumed equal to the plastic components of the strain. This is due to that the elastic strains are negligible compared with the plastic strains.

In the process of conducting model simulations to compare with experimental data, values of the aspect ratio should be assumed in the range of 1.5 to 2.0 for dense sands, and in the range of 2.0 to 3.0 for loose sands. However, the following limitation on the value of the aspect ratio should be satisfied in order to prevent possible numerical problems

$$\rho \geq 1 + \sqrt{1 - \frac{M_c^2}{M_p^2}} \quad \text{if } 1.5 < \rho < 2.0 \quad (4.3)$$

4.10 Plastic Modulus Parameter, h_0

There are no direct procedures to determine the plastic modulus parameter, h_0 , from the experimental results. Instead, this parameter is determined by trial and error by comparing model simulations with experimental results. Variation in the value of this parameter has a little effect on model simulations of material behavior. It effects the location of the peak failure in monotonic loading. This means that a higher value for the parameter h_0 results in that less strain is required to obtain the peak failure stress.

4.11 Model Sensitivity

4.11.1 General

Model sensitivity is the variation of results from simulations of material response due to variation in its parameters. The study of the model sensitivity was undertaken to obtain a feeling for the relative importance of each of the model parameters in the overall performance of the model when determining its parameters.

A loose to medium sand was selected for the purpose of this study. The model parameters representing this sand is listed in Table 4.1. Since triaxial drained tests are usually used to determine the model parameters, the quantitative effect of changes in each of the model parameters was studied on the deviator stress, q , versus axial strain, ϵ_1 , and volumetric strain, ϵ_v , versus axial strain relations.

Figs. 4.2 through 4.9 show the qualitative influence of varying one of the model parameters at a time while keeping the others constant on the behavior of the model. Initial effective confining pressure of 300 kPa was used. Series "a" of these figures shows the deviator stress versus axial strain relation, whereas, series "b" shows the volumetric strain versus axial strain relation. Each of these figures shows five curves labeled as base, -10%, -30%, +10%, and +30%. The curves labeled "base" correspond to the material behavior using the base values for all model parameters. The curves labeled -10% and -30% represent the material behavior when the subject model parameter value is reduced by 10% or 30% of its base value respectively. Similarly, for the curves labeled +10% and +30% correspond to the behavior when the subject parameter value is increased by 10% or 30% respectively. The selection of the 10% and 30% was made to create a base for the comparison between different model parameters. In all the cases below, the incremental relations of Section 3.2.2 were used with the increment of axial strain equal to 0.05%.

4.11.2 Effect of Unloading Reloading Modulus, κ

The qualitative effect of varying the unloading reloading modulus, κ , is shown in Figs. 4.2a and 4.2b. Variations in κ showed no effect on the deviator stress versus axial

strain as well as volumetric strain versus axial strain. An increase in κ means a more elastic behavior since with the increase in κ plastic deformation will decrease. So one should expect, for small strain, a softer response when κ increases and vice versa. However, the results obtained in Figs. 4.2a and 4.2b show negligible effect of varying this parameter. This can be attributed to that the elastic components of strain are very small as compared with the plastic components.

The stress path considered in this sensitivity study is related to conventional triaxial compression and extension tests only. This parameter is important when simulations are carried out for undrained tests as well as isotropic and one-dimensional consolidation tests.

4.11.3 Effect of Slope of Critical State Line in $e-\ln(p)$ Plane, λ

The variations in the slope of critical state line in $e-\ln(p)$ plane, λ , produced the results presented in Figs. 4.3a and 4.3b. As the value of this parameter increased, the material behaved in a softer manner. That is, a lower deviator stress and a higher volumetric strain are produced at the same axial strain as compared with the base response.

4.11.4 Effect of Critical Void Ratio at Unit Mean Effective Pressure, Γ

A more pronounced variation in the material response was observed when the critical void ratio at unit mean effective pressure, Γ , was varied as shown in Figs. 4.4a and 4.4b. An increase in the value of Γ resulted in much stiffer response as compared with that of the base behavior. The comparison is shown in Fig. 4.4a. Furthermore, the higher the value of the critical void ratio the higher was the dilatancy of the material.

4.11.5 Effect of Friction Angles, ϕ_c , ϕ_e , and ϕ_p

The effect of varying the ultimate friction angle in compression, ϕ_c , on the material behavior is presented in Figs. 4.5a and 4.5b. The higher the value of the ultimate friction in compression, the higher the strength (Fig. 4.5a) and lower the dilatancy

(Fig. 4.5b) of the material. The same kind of effect is expected to take place in studying the ultimate friction angle in extension, ϕ_e . An increase in the peak failure friction angle, ϕ_p caused the deviator stress to increase and the volumetric strain to decrease as compared with those of the base case as shown in Figs. 4.6a and 4.6b.

4.11.6 Effect of Poisson's Ratio, ν

The variation of the Poisson's ratio, ν , on the material behavior is shown in Figs. 4.7a and 4.7b. Similar to the effect of the model parameter κ , the variation in the Poisson's ratio, ν , produced a negligible effect on the material behavior.

4.11.7 Effect of Bounding-Surface Aspect Ratio, ρ

Higher values for the bounding surface aspect ratio, ρ , produced a softer response of the material behavior as shown in Fig. 4.8a. The lower values of the bounding-surface aspect ratio produced more dilatant behavior as shown in Fig. 4.8b. The effect of this model parameter is pronounced even for low axial strain level. All other model parameters, except for Γ , started to show their effect on the material behavior at relatively higher axial strain levels.

4.11.8 Effect of Plastic Modulus Parameter, h_0

Similar to the model parameters κ and ν , the plastic modulus parameter, h_0 , produced minimal effect on the material behavior as shown in Fig. 4.9a and 4.9b.

4.12 Model Parameters Categorization

The results presented in Figs 4.2 throughout 4.9 provided useful information regarding the model sensitivity to the variation in its parameters. However, this was for a loose to medium sand tested under medium confining pressure (300 kPa). To make reliable conclusions regarding which model parameter is the most governing and which are the least effective parameters, extensive parametric study was carried out. In this study, the

analysis presented earlier for the loose to medium sand (Figs. 4.2-4.9) was repeated for loose and dense sands. Furthermore, different initial confining pressures were considered in the analysis for each of the loose, loose to medium, and dense sands (Altaee et al., 1991) .

According to the results of the parametric study, the model parameters were divided into three categories as follows:

1. High influence parameters (includes one parameter)

Critical void ratio at unit pressure, Γ

2. Intermediate influence parameters (includes five parameters)

Bounding-surface aspect ratio, ρ

Ultimate friction angle in compression, ϕ_c

Ultimate friction angle in extension, ϕ_e

Peak friction angle in compression, ϕ_p

Slope of critical state line in e - $\ln(p)$ plane, λ

3. Low influence parameters (includes three parameters)

Plastic modulus, h_o

Unloading-reloading modulus, κ

Poisson's ratio, ν

In addition, the qualitative effect of each of the above parameters on the q - ε_1 and ε_v - ε_1 is shown in Table 4.2. A plus sign in this table means the effect is to increase, and the negative sign means that the effect is to decrease the subject value.

CHAPTER FIVE
MODEL VALIDATION:
SIMPLE STRESS ANALYSIS

5.1 Introduction

Every newly developed constitutive model should be validated before its adaptation for use in practical application. The advantage of such validation is to gain a required confidence with the constitutive model as well as to identify potential limitations.

In the work presented by Bardet (1986; 1987), comparisons were made between model simulations and measured stress-strain response of the Sacramento River sand and Fuji River sand. These comparisons illustrated the validity of Bardet's model for the cases considered. However, only the conventional triaxial compression and the conventional triaxial extension stress-path tests were considered. The validity of the model for other stress paths was not established. More importantly, the performance of the model in the solution of boundary value problems was not demonstrated. This later stage is considered in Chapter 6.

In this chapter, as the first part of the validity investigation, comparisons were made between laboratory test results and the corresponding model simulations. The behavior of four different soils were considered in a number of conventional and non-conventional stress-path tests. Comparisons were made between the model simulations and the measured response for the loose and dense Sacramento River sand, loose Fuji

River sand, a silty sand, and a crushed quartz sand. Tests on the Sacramento River sand were conducted by Lee and Seed (1967) and Seed and Lee (1967). Tests on the Fuji River sand were carried out by Ishihara et al. (1975) and Tatsuoka and Ishihara (1974). True triaxial test results published by Desai and Siriwardane (1984) were used in the comparisons made for the silty sand. The tests on the crushed quartz sand were carried out at the University of Ottawa.

5.2 Behavior of Sacramento River Sand

5.2.1 General

Sacramento River sand is a subangular to subrounded sand. It has a maximum and minimum void ratios of 1.03 and 0.61 respectively (Lee and Seed, 1967). The experimental observations from the behavior of this sand during different laboratory tests have contributed to the development of Bardet model. Tests results are available for this sand with void ratios varied between 0.61 and 0.87. The group of tests considered herein included drained and undrained triaxial compression tests carried out under a wide range of confining pressure on loose ($e_o=0.87$) and dense ($e_o=0.61$) samples of this sand.

The nine model parameters for the dense and loose sands were evaluated as given in Table 5.1. Only the triaxial drained tests were used in determining these parameters. The parameters listed in Table 5.1 were used without modification in performing model simulations for drained conventional triaxial compression, undrained conventional triaxial compression, proportional loading, and K_o -loading stress path tests. This means that the only one set of the model parameters listed in Table 5.1 are sufficient to describe the behavior of the this sand under the variety of test conditions considered herein.

5.2.2 Drained Triaxial Compression Tests

The comparison between measured and calculated stress-strain and volume change behavior for the dense and loose Sacramento River sand is shown in Figs. 5.1 and 5.2,

respectively. The symbols in these figures represent the measured soil behavior and the lines represent the calculations by the constitutive model. These comparisons are provided using deviator stress ($q = \sigma_1 - \sigma_3$) versus axial strain, ε_1 , and volumetric strain, ε_v , versus axial strain.

The calculation reproduced several important aspects of soil behavior as measured during the tests for both dense and loose sands. These aspects are the reduction of the deviator stress (softening) after reaching some peak value, the trend of volumetric strain variation with different confining pressures from expansive to contractive for small and high confining pressures, and the tendency for the increments of volumetric strains to become negligible at high axial strains.

For a very wide range of void ratio and confining pressure considered here, the calculations are in very good agreement with those measured for the full range of strains considered in the tests.

5.2.3 Undrained Triaxial Compression Tests

Results of undrained triaxial compression tests for the same dense and loose Sacramento River sand described in Section 5.2.2 were available (Seed and Lee, 1967) which facilitated its use for comparison with present calculation. As pointed out earlier, the results of these tests were not used in any way in the determination of the model parameters listed in Table 5.1.

The measured and calculated pore pressure versus axial strain, and effective stress path in p - q plane for the dense Sacramento River sand are shown in Fig. 5.3 for samples tested with initial effective confining pressures of 1.05, 1.51, 2.02, 2.99, and 4.01 MPa. Similarly, Fig. 5.4 compares the measured and calculated quantities for the loose Sacramento River sand tested with initial effective confining pressures of 0.3, 0.5, and 1.26 MPa.

The comparisons provided in Figs. 5.3 and 5.4 show a good agreement between the measured and calculated undrained response for the whole range of initial effective confining pressures as well as the initial void ratio.

5.2.4 Proportional Loading Stress Path

The loading condition in which the ratio between the principal stresses remains unchanged during the course of loading or unloading is termed as proportional loading. Test results indicated that proportional loadings with increasing stresses produce elastic as well as plastic deformations, while proportional loadings with decreasing stresses produce only elastic deformation. This experimental observation cannot be modeled by some elasto-plastic constitutive models such as Lade's work hardening model (Lade, 1972). In this model, a conical yield surface was employed with straight line traces in planes containing the hydrostatic axis. This deficiency in the model was later overcome in two independent developments. In the first, Lade developed a more elaborate constitutive model, named the work-softening model. In the second development, Evgin and Altaee (1990) attached a cap yield surface to the original Lade's work-hardening model. Both models correctly reproduced the material behavior during proportional loading.

In the present work, proportional loading tests are modeled by specifying increments of principal stresses such that the ratio between the principal stresses is always constant and equal to the value applied in the experiments. Proportional loading tests on the dense and loose Sacramento River sand are available (Lade, 1975) for ratios of major to minor principal stresses of 1.00, 1.77, 2.20, and 2.80. These ratios also represent the axial stress to the lateral stress. Using the same model parameters provided in Table 5.1, simulations were made for these tests. For all ratios, the simulation started from an isotropic stress condition of 10 kPa. Comparison between the observed and calculated strains for the proportional loading tests of the dense sand is shown in Figs. 5.5 and 5.6. The calculated axial strains (Fig. 5.5) as well as the volumetric strains (Fig. 5.6) are reasonably close to the values measured for the different-stress-ratio tests and for the wide range of stress level considered in the tests.

5.2.5 K_0 -Loading Stress Path

The no lateral strain test condition is referred to the K_0 loading condition. Calculation was performed to simulate the K_0 loading condition using the model parameters of Table 5.1. The calculation starts from an isotropic state of stress of 10 kPa. The calculated K_0 behavior of the dense Sacramento River sand is presented in Figs. 5.7 and 5.8. No test results are available to compare with. However, the calculated behavior is compared with K_0 values calculated from the Jaky's equation

$$K_0 = 1 - \sin\phi \quad (5.1)$$

The angle ϕ corresponds to the effective peak friction angle in the conventional triaxial compression test. The calculation proceeded by simulating test conditions usually maintained in conducting such tests. This was achieved by incrementally increasing the axial stress and finding the corresponding lateral stress which satisfy zero lateral strain for each of the applied loading increment. The axial stress versus lateral stress is shown in Fig. 5.7. It is of interest to note that this relation is not linear which suggests different K_0 values corresponding to different stress level to which the sample was subjected. This observation is more clear in Fig. 5.8 which shows the K_0 values corresponding to different lateral stresses. In this figure, K_0 increased from about 0.3 at low lateral pressure to about 0.4 at higher pressure.

The calculated K_0 values using the model and those calculated using the above equation compare very well. The values of ϕ used in the calculation using the Jaky's equation are those peak values measured during drained triaxial compression tests (Lee and Seed, 1967). The capability of the model in reproducing values of K_0 that depend on the stress level is attributed to the fact that the constitutive model accounts for the curvature of the failure envelop in the p-q plane.

5.3 Behavior of Fuji River Sand

5.3.1 General

Fuji River sand is subangular in grain shape. The effective particle size is 0.22 mm, and the maximum and minimum void ratios are 1.08 and 0.53, respectively. The uniformity coefficient is 2.21 (Tatsuka and Ishihara, 1974). Different monotonic and cyclic triaxial tests on loose Fuji River sand are available (Tatsuka and Ishihara, 1974; and Ishihara et al., 1975).

For the loose Fuji River sand, the model parameters were evaluated using drained triaxial compression and extension tests. The triaxial compression test was performed on a sample having initial void ratio of 0.772, whereas, the extension test was performed on a similar sample having initial void ratio of 0.725. The effective initial confining pressure used in both tests was 200 kPa. The calculated model parameters are listed in Table 5.2. The parameters listed in Table 5.2 were then used without modification in performing model simulations for monotonic triaxial compression and extension tests, and cyclic drained and undrained tests.

5.3.2 Monotonic Triaxial Compression and Extension Behavior

The parameters listed in Table 5.2 were determined from the triaxial drained compression and extension tests. These parameters were used in reproducing the drained behavior of the two tests used in the calibration of these parameters. The comparison between the test results and the simulations of these two tests is shown in Fig. 5.9. Fig. 5.9a shows the deviator stress versus the axial strain for the compression and extension tests. Fig. 5.9b shows the volumetric strain versus the axial strain of the same tests. Comparing these results does not provide indication of how good the model is since the model parameters were determined from -matched to- these tests. However, the use of these parameters, without any modification, in simulating the undrained behavior of this sand during triaxial compression and extension tests provides further validation of the model for this sand which is quite different in its properties than the Sacramento River sand considered in Article 5.2.

Fig. 5.10 compares the measured and simulated behavior of the loose Fuji River sand in triaxial undrained compression and extension tests. These tests were carried out on samples having initial void ratio of 0.756 for the compression test and 0.774 for the extension test. The deviator stress versus deviator strain is shown in Fig. 5.10a, whereas, the effective stress path represented in the deviator stress versus the mean effective pressure is shown in Fig. 5.10b. Confining pressure of 300 kPa was used in both tests. The simulations of both tests are very close to the measurements made for the full range of the tests.

5.3.3 Drained Cyclic Triaxial Behavior

Without adding further parameters to the model parameters listed in Table 5.2, simulations were carried out for the cyclic behavior of the Fuji River Sand. In the drained cyclic triaxial test considered here, the confining pressure was kept at 200 kPa throughout the test. The deviator stress, on the other hand, was increased at each loading cycle. The simulation carried out for the behavior of the sand during this test followed the stress controlled nature of this test. The stress ratio (q/p) versus the deviator strain is shown in Fig. 5.11a for the test results and in Fig. 5.11b for the simulation. The similarity between the measured and simulated stress-strain (expressed as stress ratio versus deviator strain) is clear in this figure. Further, the model was able to reproduce the volumetric strains measured throughout this test as shown in Fig. 5.12. Fig. 5.12a shows the measured volumetric strain versus the stress ratio, whereas, Fig. 5.12b shows the simulated volumetric strains versus the stress ratio.

5.3.4 Undrained Cyclic Triaxial Behavior

The experimental results and the model simulation of the undrained cyclic triaxial test is shown in Figs. 5.13 and 5.14. In this test, cycles of constant amplitude of the deviator stress were applied under undrained condition. Fig. 5.13a shows the measured effective stress path represented by the deviator stress versus mean pressure, whereas, Fig. 5.13b shows the simulation of this relation. Both, the experimental results and model simulation showed similar trends. The pore water pressure continued to build up (the effective mean pressure decreased as in Fig. 5.13) until the stress state reached the

critical state line in the extension side. By definition of the critical state, as explained in Chapter 2, the soil sample at this state showed infinite deviator strain. At the stress state where the sample is at the state of initial liquefaction, the present constitutive model is quite capable of reproducing this state of initial liquefaction. However, the model provides no further simulation beyond the onset of liquefaction (initial liquefaction) due to the drop of the mean effective pressure to zero. The measured and the simulated stress-strain behavior as represented by the deviator stress versus deviator strain are shown in Fig. 5.14. It is of interest to note that, even though the material reached the failure state, both experimental observation and the model simulation showed small deviator strains of about 1%.

5.4 Behavior of Crushed Quartz Sand

5.4.1 General

The factory crushed quartz sand was used before on many occasions at the University of Ottawa in model scale tests of footings, slopes, and reinforced earth. It is composed of angular grains with a solid density of $2,650 \text{ kg/m}^3$. The maximum and minimum void ratios are $e_{\max}=1.13$ and $e_{\min}=0.42$ determined by following the ASTM standards, designation D2049. The uniformity coefficient of the sand is 2.6 and the effective diameter is 0.2 mm.

The objective of the experimental program carried out on this sand was to investigate experimentally the effect of oil contamination on the stress-strain-strength behavior of this sand. Furthermore, it was intended to apply advanced constitutive relations to model this effect numerically, and to provide model parameters to be used in the analysis of some boundary value problems involving this sand.

A grade 50 Motormaster heavy-duty motor oil was used to saturate the sand to simulate oil contaminated samples. The measured density of the oil was 927 kg/m^3 and its viscosity was 455 mPas at 24°C . In the case of clean sand, water was used to saturate the samples.

Four series of tests were carried out using a standard triaxial testing equipment. In the first series, the samples were saturated with water, consolidated isotropically and sheared up to about 20% axial strain. In the second series, the samples were saturated with oil and tested with a similar procedure. In the third series, samples were saturated with oil, consolidated isotropically and sheared under undrained conditions. In the final series of tests, the samples were saturated with water and tested in undrained conditions. Three different effective confining pressures were applied. More details regarding the experimental program can be found in Evgin et al. (1989).

5.4.2 Drained Triaxial Compression Behavior

Three drained triaxial compression tests were carried out on clean and oil saturated (contaminated) sand having initial void ratio of 0.92. The effective confining pressures applied in these tests were 138 kPa, 276 kPa, and 414 kPa. The experimental data were used to obtain the parameters of the bounding surface plasticity model to make simulations for the measured response of soil. For the clean sand and oil contaminated sand, the model parameters were obtained from the results of one drained triaxial compression test ($\sigma_3=276$ kPa) and one isotropic compression test. The parameters are given in Table 5.3 for both sands.

Using the model parameters provided in Table 5.3, the drained response of the clean and oil contaminated sands were simulated. The simulations for the drained behavior of clean sand are provided in Fig. 5.15 with the measured data. The simulations for the oil contaminated samples in drained tests are shown in Fig. 5.16 also with the measured data. Figs. 5.15a and 5.16a show the deviator stress versus axial strain, whereas, Figs. 5.15b and 5.16b show the volumetric strain versus the axial strain. For the tests with initial effective confining pressure of 276 kPa, the agreement between the measured and simulated responses is forced through the calibration process which ultimately lead to the set of model parameter listed in Table 5.3. On the other hand, the agreement between the measured and simulated responses for the 138 kPa and 414 kPa initial effective confining pressure tests is an indication of the validity of the model for these tests. The simulations are in a very good agreement with the measured data for the clean as well as the oil contaminated sands.

5.4.3 Undrained Triaxial Compression Tests

Results of undrained triaxial compression tests carried out on samples of the clean and oil contaminated sand having similar initial void ratio to those tested in drained conditions are shown in Fig. 5.17. The initial effective confining pressure applied in both undrained tests was 138 kPa. As explained in Section 5.4.2, the results of these tests were not used in any way in the determination of the model parameters listed in Table 5.3. This means that the model parameters derived from the drained tests are used in simulating the undrained response of this sand similar to the simulation carried out earlier for the Sacramento River Sand and Fuji River Sand.

The measured and calculated pore fluid pressure versus axial strain are compared in Fig. 5.17. The comparison indicates the very good agreement between the measured and simulated undrained behavior for the whole range of tests which were conducted up to about 15% axial strain. These simulations show clearly that the bounding surface model is capable of predicting the undrained behavior for both clean and oil contaminated sands from the drained behavior of the same sands.

5.5 Behavior of a Sandy Silt

5.5.1 General

This soil consists of 50% of fire clay and 50% of Florida zircon sand. Zircon has a solid density of $4,650 \text{ kg/m}^3$. The soil was mixed with 10% of No. 5 SAE mineral oil to reduce the effect of loss of moisture during testing. The soil was classified as sandy silt and referred to as "Artificial soil" (Desai et al., 1981; Desai and Siriwardane, 1984; and Faruque and Desai, 1985).

The soil is highly compressible. Its total density is about $2,000 \text{ kg/m}^3$ and its maximum and minimum total densities are $2,650 \text{ kg/m}^3$ and $1,000 \text{ kg/m}^3$, respectively. A schematic representation of its fabric and the grain size distribution are given by Desai and Siriwardane (1984).

The results of extensive testing programs are available for this soil. In fact, the soil was prepared and tested under a variety of stress paths to validate a family of cap models developed by Desai and his students as well as by other researchers. The experiments were carried out using a true triaxial apparatus (Desai and Siriwardane, 1984) which permits tests with cubical samples of 100 mm x 100 mm x 100 mm size subjected to a wide range of stress paths. It allows the control of three major principal stresses independently.

The model parameters required to specify this sandy silt behavior under generalized loading conditions are given in Table 5.4. They were determined using the results of two triaxial compression tests (Tests CTC10 and CTC15) and one isotropic compression test (Test HC) reported by Desai and Siriwardane (1984). The results of these tests are shown in Figs. 5.18 through 5.20.

5.5.2 Back Calculation of Stress-Strain Response

Figs. 5.18 and 5.19 show the comparisons of the calculated and measured results of Tests CTC10 and CTC15, while Fig. 5.20 shows the comparisons of the calculated and measured results of Test HC. The initial effective confining pressures used in Tests CTC10 and CTC15 were 69 kPa and 103 kPa, respectively. The notation CTC signifies Conventional Triaxial Compression test, whereas, the numbers represent the initial confining pressure in psi.

The back calculated stress-strain response of the artificial soil indicated that the model parameters listed in Table 5.4 were satisfactory.

5.5.3 Soil Response in Different Stress Path Tests

The behavior of the sandy silt in three different stress paths was simulated. These tests were simple shear, triaxial extension, and triaxial compression. Figs. 5.21 through 5.23 give the soil behavior in simple shear, triaxial extension, and triaxial compression loading conditions. These tests were all stress controlled.

The simple shear test (Test SS10) was conducted by first consolidating the soil specimen isotropically under a pressure of 69 kPa. The test was then continued by holding one of the principal stresses constant while the other two were increased and decreased respectively in equal increments. This loading pattern kept the state of stress of the specimen on the same octahedral plane. The calculated and the measured stress-strain responses during this test are shown in Fig. 5.21.

In the second stress path test, the soil sample was consolidated under 69 kPa before a triaxial extension test (Test TE10) was carried out. Similar to Test SS10, this test started by consolidating the sample isotropically under a pressure of 69 kPa. After the consolidation stage was completed, one of the principal stresses was held constant while the other two were increased equally. The calculated and the measured stress-strain responses in this test are given in Fig. 5.22.

The third test in this series was a triaxial compression test (Test TC10) in which one of the principal stresses was increased while the other two were reduced after consolidating the sample to 69 kPa. The reductions in the two principal stresses were such that the octahedral normal stress remained constant. As pointed out by Desai and Siriwardane (1984), this test has special significance, since it isolates the effect of deviator stress. Fig. 5.23 compares the results of the calculations and the measurements for this test.

The comparisons between the measured and calculated soil behavior and the model calculations provide an additional support for the validity of the bounding surface plasticity model of Bardet (1986; 1987). In all three different stress paths, the calculated behavior agreed well with the behavior of the soil observed in laboratory experiments.

CHAPTER SIX
MODEL VALIDATION:
BOUNDARY VALUE PROBLEMS

6.1 Introduction

The final objective of developing a constitutive model is its incorporation in the analysis of different engineering problems with different boundary and loading conditions. However, before establishing the level of confidence with any constitutive model to qualify it for the use in the analysis of engineering problems, some validation steps are required. In Chapter 5, the bounding surface plasticity model was validated with respect to the behavior of different soils subjected to variety of stress and strain conditions. A common characteristic of the cases considered in Chapter 5 is that the whole soil sample was assumed to be subjected to a uniform state of stress and strain. Therefore, the validation process described in the previous chapter is not sufficient to make conclusions regarding the validity of the constitutive model for more complicated boundary value problems.

For this reason, the analysis of three boundary value problems were considered in this chapter. The first problem considered is the analysis of the behavior of a model-scale footing resting on the surface of a sandy silt. The details of the model-scale footing test were reported in Desai et al. (1981), Desai and Siriwardane (1984), and Faruque and Desai (1985). The second problem is the analysis of the behavior of 14/25 Leighton Buzzard sand in the Cambridge Simple Shear Device. A constant vertical load test provided in Budhu (1979) was used for this purpose. The third boundary value problem

considered here is the analysis of a foundation resting on the ground surface for three different conditions of the supporting soil. In the first analysis, the foundation soil was considered as a clean loose sand. In the second analysis, the foundation was resting on the contaminated loose sand. This third problem was considered in order to illustrate the effect of changes in the crushed quartz sand caused by oil contamination considered in Chapter 5 on the behavior of an engineering structure. Comparative results are presented to show the differences among the behavior of a foundation resting on a clean sand, on an oil contaminated sand, and on a sand contaminated locally.

6.2 Analysis of Footing on Sandy Silt

6.2.1 General

The first boundary value problem considered in this chapter is the analysis of the behavior of a model-scale footing resting on the surface of a sandy silt. The details of the model-scale footing test were reported in Desai et al. (1981), Desai and Siriwardane (1984), and Faruque and Desai (1985). A rigid rectangular box of size 114 x 203 x 876 mm was used as a container for the soil used. The footing was 76 mm wide, 19 mm thick, and 114 mm long as shown in Fig. 6.1. It was placed at the center of the box. Vertical load was applied on the center of the footing in increments. Measurements were taken for vertical displacements corresponding to each load increment.

6.2.2 Soil Characteristics and Model Parameters

The soil used in the model-scale footing test consisted of 50% of fire clay and 50% of Florida zircon sand. The behavior of this soil, when tested in a variety of stress- path test, was considered in Chapter 5. This soil was referred to as "Artificial Soil". The model parameters of this soil were also evaluated from conventional triaxial compression tests in Chapter 5 and were given in Table 5.4. No modification was made to any of these parameters in the analysis of the footing on sandy silt.

6.2.3 Initial Stresses in Soil

Initial vertical stresses in the soil mass were calculated on the bases of the soil density ($2,000 \text{ kg/m}^3$). Horizontal stresses were taken equal to vertical stresses ($K_0=1$) as reported by Desai et al. (1981) based on the insitu measurements they performed, where K_0 is the coefficient of earth pressure at rest. Since the soil surface was made level in the preparation for this test, shear stresses were taken as zero in vertical and horizontal planes everywhere in the soil.

6.2.4 Finite Element Idealization

The model-scale footing and the soil mass were analyzed as a plane strain case. Because of the loading and geometrical symmetry, only one half of the soil-footing system was considered in the analysis. Fig. 6.2 shows the finite element mesh used in the analysis. The vertical side of the container and the vertical boundary along the footing center line were assumed smooth (only vertical displacements permitted). Vertical and horizontal displacements were prevented at the bottom of the container.

6.2.5 Finite Element Results versus Experimental Results

The observed load-displacement relation of the model-scale footing and the results of the finite element analysis obtained in the present analysis are shown in Fig. 6.3. In this figure, the load is represented by the average contact pressure (applied vertical load divided by the footing width), and the displacement represents the vertical displacement measured at center line of the footing. The figure also includes results of finite element analysis reported in Desai and Siriwardane (1984) and Faruque and Desai (1985).

As illustrated in Fig. 6.3, the use of the Drucker-Prager model gave unsatisfactory results as compared with those measured in the experiment. The critical state model, on the other hand, provided better results than that of the Drucker-Prager model, but not as good as the modified cap or the bounding surface plasticity model. The bounding surface plasticity model provided excellent correlation with experimental data as well as with the modified cap model predictions. These two models, similar to the critical state

model, have hardening rules based on plastic volumetric strain. Changes in void ratio takes place as a result of changes in volumetric strains, and this affects the quality of predictions of the behavior of some models. The Drucker-Prager model lacks this refinement.

Photographs provided in Desai and Siriwardane (1984) show that the soil experienced large volumetric strains during the test. Volumetric change was localized in the soil below the footing. This observation is in contradiction with the results obtained using the Drucker-Prager model which shows an almost uniform movement underneath the footing and in the soil zones away from it. Furthermore, a considerable amount of heave away from the footing was calculated using the Drucker-Prager model. In contrast, the capability of the bounding surface plasticity model in reproducing the localized nature of the soil deformation is good. The displacement field (which gives a representative indication of volume change experienced by the soil) under an applied pressure of 102 kPa is shown in Fig. 6.4. Furthermore, the deformed finite element mesh is shown in Fig. 6.5. The agreement between the experimental results and the calculations of the bounding surface model is excellent.

6.3 Leighton Buzzard Sand in Cambridge Simple Shear Device

6.3.1 General

It has been stated by several researchers (Desai and Siriwardane, 1984; Desai, Phan, and Sture, 1981; and Faruque and Desai, 1985) that the use of volumetric-strain hardening type constitutive models would provide satisfactory results in the solution of boundary value problems where volume changes dominate the soil behavior. The same researchers have also indicated that using such models in the solution of problems dominated by stress paths such as in the simple shear device would not provide satisfactory results. The second statement was based on a comparative study where significant discrepancies were observed between the simulations using a volumetric-strain-hardening cap model and the sandy silt behavior in a simple shear and triaxial extension stress path test (Desai and Siriwardane, 1984). Since Bardet's bounding surface plasticity model uses the plastic volumetric strain as the only internal variable to

control the evolution (hardening/softening) of the bounding surface, the selection of the soil behavior along a simple shear stress path is a useful test in validating this constitutive model.

The Cambridge Simple Shear Device (Roscoe, 1953) accepts a cuboidal sample of 100 mm x 100 mm x 20 mm high. Samples in this device are confined between rigid boundaries. Testing soils is ordinarily carried out in two stages: consolidation and shearing. There is a similarity between loading condition in the simple shear test and the loading condition of soils in some practical problems such as the soil adjacent to pile foundation shaft, foundation soils of offshore platforms, and others.

Testing soils in the simple shear device has been criticized in many studies due to the nonuniformities of stresses and strains developed in the soil sample. However, most criticism has been based on elastic analysis which exaggerates the nonuniformities of stresses and strains. Carefully conducted experiments using simple shear devices have confirmed the nonuniformities in stresses and strains. However, measurements also showed that the middle one-third of soil sample deforms uniformly (Budhu, 1979). In addition, measured shear stresses within the middle one-third of the sample have been shown to be quite close to that based on average shear stress along the full length of the sample.

The consideration of soil behavior in the simple shear device is of two fold advantage: the validation of the constitutive model and the examination of the uniformity of stresses and strains in the soil tested in such a device.

6.3.2 Soil Used in the Analysis

In the present analysis, the behavior of 14/25 Leighton Buzzard sand in the Cambridge Simple Shear Device is considered. A constant vertical load test provided in Budhu (1979) was used for this purpose. The soil is modelled using the bounding surface plasticity model. The model parameters of this sand are determined based on the sand properties provided in Budhu and Britto (1987). These parameters are shown in Table 6.1.

6.3.3 Finite Element Model

Plane strain idealization was adopted for this problem. The finite element mesh used in the analysis is shown in Fig. 6.6. It consists of three different parts, the upper metal platen (ABCD), the soil sample (CDEF), and the lower metal platen (EFGH). The metal components of the simple shear device included in the analysis were modeled as linear elastic material. The platen surfaces in contact with the soil sample are assumed to be rough and no slip was considered along these boundaries. The two side flaps of the device are not included in the analysis since their surfaces in contact with soil are lubricated in testing. This eliminates any shear forces to be develop between these two flaps and the soil sample. A similar finite element model was used by Budhu and Britto (1987) for the same simple shear device.

In the finite element analysis, the sample was first consolidated and then sheared by applying horizontal displacements in increments. Each specified increment of horizontal displacement was applied in full for the nodes located along AD and BC. The nodes located along DE and CF were displaced at different fractions of the full displacement increment according to their vertical location in the mesh. This was done to preserve the effect of the two side flaps on the deformation pattern of the sample. The lower metal platen (EFGH) was considered to be fixed in its initial position and no displacement was allowed throughout the analysis.

6.3.4 Results of Analysis and Comparison with Test Results

Fig. 6.7 compares the results of the finite element analysis and the measured stress-strain response of the Leighton Buzzard sand. The stress was expressed in terms of shear stress divided by the effective applied vertical stress, whereas the strain represents the shear strain calculated by dividing the horizontal displacement by the initial thickness of the soil sample. Two curves are shown for the finite element analysis (dashed) and two curves for the experimental observation (solid). The curves labeled as "core" correspond to the measured or calculated results at the core of the soil sample

(the middle one third of the sample). The next two curves labeled as "average" correspond to the results for the full length of the soil sample along the top boundary.

Two observations can be made from Fig. 6.7. First, the present finite element analysis confirmed the experimental observations that the difference between the measurements on the sample core and those normally made in routine test along the full top boundary is less than 5%. Second, the agreement between the present finite element analysis and the experimental measurements is a good indication of the ability of the bounding surface plasticity model to simulate the behavior of sand in this test.

6.4 Foundations on Clean and Oil Contaminated Sand

6.4.1 General

The experimental results presented in Chapter 5 show that oil contamination affects the strength and deformation characteristics of the crushed quartz sand used in tests. Further, it was shown in the same chapter that the bounding surface plasticity model is capable of simulating the behavior of this sand in its clean and oil contaminated states.

Containment of oil in the soil used to construct caisson retained islands is one of the methods proposed to prevent pollution of the Beaufort Sea if a blow out occurs during drilling operations. It is therefore necessary to determine the impact oil contamination may have on the stability of islands (Evgin et al. 1999). The analysis can also be used to assess the integrity of foundation soils or pipeline shore approaches contaminated by oil spills.

In order to illustrate the effect of changes in the soil properties on the behavior of an engineering structure, a finite element analysis was carried out. Comparative results are presented to show the differences among the behavior of a foundation resting on a clean sand, on an oil contaminated sand, and on a sand contaminated locally. The selection of the type of foundation considered here is based on confidence gained by applying the model to the analysis of the footing presented in Section 6.2. Furthermore, the numerical simulations showed that the experimentally observed behavior of the crushed

quartz sand was suitably modelled by the bounding surface plasticity model as shown in Chapter 5.

6.4.2 Soil Used and Model Parameters

The foundation soil used was the factory-crushed quartz sand considered in Chapter 5. It is composed of angular grains with a solid density of $2,650 \text{ kg/m}^3$ and void ratio of 0.92. The same kind of oil used in the experimental program was assumed in the finite element analysis so as to have an oil contaminated soil similar to that tested in the experimental programme.

As explained in Chapter 5, the experimental data was used to obtain two sets of parameters of the bounding surface plasticity model. One of these sets is representative for the clean crushed quartz sand and the other is representative for the oil contaminated crushed quartz sand. These two sets of model parameters were given in Table 5.3 and will be used in the present analysis without any modification. This means that the soil assumed in the finite element analysis of this section is identical to that tested and its behavior is described in the previous chapter.

6.4.3 Foundations Analyzed

A six meter wide foundation resting on the ground surface was analyzed for three different conditions of the supporting soil. In the first analysis, the foundation soil was considered as a clean loose sand. In the second analysis, the foundation was resting on the contaminated loose sand. In the third analysis, only a section of the foundation soil was assumed to be contaminated with oil. The finite element analysis, for comparative purposes, was carried out using both rigid and flexible foundations. One half of the mesh shown in Fig. 6.8 was used in the first two cases of analysis due to the loading and geometrical symmetry. In the third case of analysis, where a part of the foundation soils was contaminated, it was necessary to utilize the full finite element mesh in the analysis. For all three cases of the analysis, the foundation load was uniformly distributed and it was applied in the vertical direction along the upper surface of the foundation. The load-displacement, rotation, and vertical stresses in soil close to the

foundation face were studied. In all the cases analyzed, no slip was allowed between the foundation and the soil underneath.

6.4.4 Analysis of Rigid Foundation

Two curves given in Fig. 6.9 show the load-displacement behavior of the rigid foundation on the clean and contaminated sands. The load is represented by the average contact pressure which was calculated by dividing the total applied vertical load by the foundation width. The displacement represent the vertical movement at the centerline of the foundation. It was assumed in these two cases, that the sand was either clean or contaminated everywhere in the soil mass considered in the analysis. As can be seen from Fig. 6.9, the vertical displacements of the foundation on the contaminated sand was about 40% larger than the displacements calculated for the foundation on clean sand at 150 kPa applied pressure. This can be attributed to the higher compressibility of the oil contaminated sand as compared with that of clean sand as measured in the laboratory tests as well as in the numerical simulation presented in Chapter 5.

When a small section of the foundation soil was contaminated with oil, the load-displacement behavior of the foundation changed significantly. In this case, the area ABCD, shown in Fig. 6.8 was assumed to be contaminated. The rest of the foundation soil was assumed to be clean sand. Fig. 6.10 shows the vertical load-vertical displacement curves for the corners of the foundation on the clean sand side and oil contaminated side. The difference between the vertical displacements at these two points was about 35% (90 mm) at an average loading level corresponds to 150 kPa. It is of interest to note that, even though the foundation was not subjected to horizontal loads, the center of the foundation displaced about 19 mm horizontally toward the contaminated sand side as shown in Fig. 6.11. The differential displacement of the foundation resulted in the rotation of the foundation shown in Fig. 6.12. Horizontal displacements as well as rotation of the foundation base might be more important to the design engineer than an additional uniform settlement caused by an oil spill.

In order to show the effect of oil contamination on the stress distributions in foundation soils, the vertical normal stresses one metre below the base of the rigid foundation were

plotted. For uniform soil conditions, where the soil is either clean or contaminated throughout, the vertical normal stresses are shown on the left side of the foundation center line in Figs. 6.13 and 6.14. The stress distributions for the case where a small section of the foundation soils is contaminated are shown in Figs. 6.15 and 6.16.

6.4.5 Analysis of Flexible Foundation

The analysis carried out for the rigid foundation was repeated for the flexible foundation. Only the stress distributions are provided here. As expected, the distribution of stresses for the flexible foundation was different than those for the rigid foundation. As illustrated in Figs. 6.13 and 6.14, the maximum vertical normal stress developed at the centerline below the flexible foundation. When only the area ABCD was contamination, this caused a redistribution of the vertical stresses in soils as shown in Fig. 6.16. The stress distribution in soils below the base of the foundation was affected significantly when only a section of the foundation soils was contaminated. This result occurred in the flexible as well as rigid foundations considered here.

CHAPTER SEVEN

REVIEW OF RESPONSE OF PILES TO AXIAL LOADS

7.1 Resistance of a Pile

The axial resistance of a pile is commonly separated into two components, the toe resistance, R_t , and the shaft resistance, R_s . The sum of these two components provides the total axial pile resistance as follows:

$$Q_u = R_t + R_s \quad (7.1)$$

where

Q_u = total axial pile resistance

R_t = toe resistance

R_s = shaft resistance

Many approaches are available for the calculation of the shaft and toe resistances of a pile. Each of these two components is treated separately in the following.

7.1.1 Toe Resistance

The total toe resistance, R_t , is defined as follows

$$R_t = A_t r_t \quad (7.2)$$

where

R_t = total toe resistance

A_t = pile toe cross sectional area

r_t = unit toe resistance

The unit toe resistance, r_t , is generally presented in the following form

$$r_t = N_t \sigma'_{z=D} \quad (7.3)$$

where

N_t = toe bearing capacity coefficient

$\sigma'_{z=D}$ = effective vertical stress at the pile toe level

Theoretically, the effective vertical stress, $\sigma'_{z=D}$, is the vertical effective stress at the pile toe level when failure occurs. However, there is no available method to determine this stress. Instead, the effective overburden stress is used.

As given in Eq. 7.3, for a known in-situ effective stress, the determination of the pile toe resistance is reduced to the determination of a single coefficient, N_t . Many theoretical solutions are available for the determination of N_t . Most of them are borrowed from bearing capacity solutions developed originally for metals, for example, the classical work of Prandtl (1921) and Reissner (1924). These theories do not account

for the soil compressibility, volume changes associated with shearing, and the dependency of the effective angle of friction on the mean pressure (the use of a linear Mohr-Coulomb failure envelope defined by a constant effective angle of friction). According to Vesic (1967), the first to apply these solutions to the problem of bearing capacity of deep foundations were Caquot (1934) and Buisman (1935).

Fig. 7.1 shows assumed failure patterns under deep foundations as suggested by different researchers (Vesic, 1967). Patterns a and b assume that general shear failure mode idealizes the failure state at pile toe. Pattern c, on the other hand, assumes that a local shear failure idealizes the failure. The failure pattern used by Bishop et al. (1945) and by Skempton et al. (1953), Pattern d, differs from the other failure patterns. In the latter, a limiting pressure was used for an expanding spherical cavity.

Bearing capacity coefficient, N_t , for circular piles based on different solutions available is shown in Fig. 7.2 (Vesic, 1977) for a wide range of effective angle of friction, ϕ' . As can be seen from this figure, the toe bearing capacity coefficient, N_t , is expressed mainly as a function of the effective angle of internal friction and to a smaller degree on the method of pile placement in the ground. Fellenius et al. (1989) pointed out that the bearing capacity coefficient, N_t , depends on soil composition in terms of grain size distribution, angularity, and mineralogical origin of the grains, original (before pile installation) soil density, and density changes due to installation technique. It is understandable that all of the above factors cannot be accounted for by the effective angle of friction alone and some additional other properties of the soil are required in order to provide a more realistic N_t value and, ultimately, a better estimation of the pile toe resistance.

Fig. 7.2 shows that even for the same soil (as represented by the effective angle of internal friction in this figure), the variability in the toe bearing capacity coefficient is large. These differences are partially due to different shapes of slip surfaces assumed in different theories, and due to some other assumptions related to the state of stress at the failure state in the soil mass surrounding the pile.

Later, Vesic (1972; 1973; 1977) applied the theory of expansion of a spherical cavity to develop a solution for the toe resistance of a pile. This approach had been used

earlier by other researchers such as Skempton et al. (1953), however, the idealization of the soil was different in the Vesic's approach. Vesic included soil compressibility and volume changes and developed a revised set of toe bearing capacity coefficient. He idealized the soil as an elastic-plastic solid characterized by strength parameters (c and ϕ) and the deformation parameters (E and ν). A volume change parameter, D , was used to represent the average volumetric strain in the plastic zone surrounding the expanded cavity. Further, Vesic pointed out that the toe resistance of a pile is not governed by the vertical effective stress in soil at the toe level, $\sigma'_{z=D}$, but by the effective normal octahedral stress in the soil at the toe level, $\sigma_o'_{z=D}$. This stress was related to vertical stress in soil at the toe level, $\sigma'_{z=D}$, by the equation

$$\sigma_o'_{z=D} = \frac{1 + 2K_o}{3} \sigma'_{z=D} \quad (7.4)$$

in which K_o is the coefficient of earth pressure at-rest. The revised equation for the unit toe resistance of a pile in a cohesionless soil was given by

$$r_t = \sigma_o'_{z=D} N_\sigma \quad (7.5)$$

in which N_σ is a toe bearing capacity coefficient. The conventional toe bearing capacity coefficient, N_t , can be related to the new bearing capacity coefficient by the following equation

$$N_t = \frac{1 + 2K_o}{3} N_\sigma \quad (7.6)$$

The effect of soil compressibility on the toe bearing capacity coefficient was expressed by using a factor called the rigidity index, I_r , given by (Vesic, 1977)

$$I_r = \frac{E}{(1 + \nu)(\sigma' \tan \phi')} \quad (7.7)$$

where ν is the Poisson's ratio, and ϕ' is the effective angle of friction. Fig. 7.3 provides a revised toe bearing capacity coefficient, N_{ϕ} , calculated for a wide range of rigidity index and effective angle of internal friction. As shown in this figure and emphasized by Vesic, the rigidity index is at least as significant for bearing capacity as the angle of internal friction.

Vesic (1977) incorporated the simultaneous effect of volume change of cohesionless soil on toe bearing capacity by introducing a volume change factor, ξ_v , which is related to the average volumetric strain at failure in the plastic zone surrounding the foundation by the following expression

$$\xi_v = \frac{1}{1 + \frac{1}{2 I_r D}} \quad (7.8)$$

and the reduced rigidity index, I_{rr} ,

$$I_{rr} = \xi_v I_r \quad (7.9)$$

Fig. 7.4 shows the reduced rigidity factor as a function of the volume change factor, ξ_v , rigidity index, I_r , and the average volumetric strains. This figure indicates that a drastic reduction in the toe resistance of a pile takes place with the increase in the volume change at failure. Vesic made the following statement with regard to the results presented in this figure:

... the effects of volume change on (toe) bearing capacity in otherwise incompressible soil may be even more significant than both I_r and ϕ .

The effective angle of friction of cohesionless material depends on the magnitude of the mean pressure, a fact always not accounted for. A cohesionless material does not show a unique effective angle of internal friction for all levels of mean pressure to which the

material is subjected. This means that the failure envelope of such materials is curved and not a straight line as assumed by all the limit equilibrium methods to determine pile toe bearing capacity coefficient. For a cohesionless material having a given density, the peak friction angle decreases with the increase in the mean effective pressure. This reduction can be as much as 10° for the range of mean pressure change taking place in the vicinity of a pile toe.

7.1.2 Shaft Resistance

Generally, the calculation of unit shaft resistance, r_s , is based on a simplified failure model. The sliding resistance of rigid bodies resting on soil is the analogy used for this purpose.

The ultimate unit shaft resistance, r_s , can be calculated from (Bozozuk, 1972)

$$r_s = M K_s \tan \phi' \sigma'_z \quad (7.10)$$

or

$$r_s = \beta \sigma'_z \quad (7.11)$$

and the total ultimate shaft resistance, R_s , is calculated by integrating the unit shaft resistance along the embedment depth of the pile, D , as

$$R_s = \int_0^D A_s r_s dz \quad (7.12)$$

where

R_u = total ultimate shaft resistance

r_s = unit shaft resistance at depth z

A_s = unit circumferential area of the pile

σ'_z = effective overburden stress at depth z

M = quotient of wall friction = $\tan\delta'/\tan\phi'$

K_s = earth pressure coefficient

ϕ' = effective friction angle (degrees)

δ' = effective soil-pile friction angle (degrees)

β = Bjerrum-Burland beta coefficient

D = pile embedment depth

The introduction of the parameter β in the above equations is a useful practical simplification. The earth pressure coefficient, K_s , was used extensively in the early research into the unit shaft resistance. However, a significant variability exists in the values recommended for K_s by different researchers as shown in Table 7.1. It can be stated that the coefficient of earth pressure, used to determine the unit shaft resistance, ranges from active earth pressure coefficient on its lower end to passive earth pressure coefficient in its upper end. Values of K_s reported in the literature range from a low of about 0.1 to a high of about 5 (Kulhawy, 1984).

Pile installation affects the state of stress in the soil in the pile vicinity. Changes occur in the vertical stresses, horizontal stresses, and direction of the principal planes. If a pile is driven or pushed into the ground, the horizontal stresses increase depending on initial magnitude of stresses, amount of soil displacement, and possibly some other factors. If instead the pile is placed into the ground by pre-drilling, then, the horizontal stresses decrease as compared with initial values.

Vesic (1977) stated that for short, driven, high-displacement piles in sand, the coefficient of earth pressure, K_s , can reach in its value the coefficient of passive earth pressure. Furthermore Vesic (1970), Sulaiman and Coyle (1971), and Touma and Reese (1974) provided test results of full-scale pile loading tests which show a drastic drop in the magnitude of K_s with the increase in the penetration depth. This was attributed by Vesic to an "assumed" considerable reduction in the effective stresses in the vicinity of the pile toe.

In the approach used to evaluate the unit shaft resistance, the soil is characterized as a rigid-plastic solid. No attention was paid to compressibility, volume change characteristic, and non-linearity in the failure envelop. As in the case of toe resistance, these factors are important for the development of the shaft resistance.

Very little attention has been paid in the literature to changes of horizontal and vertical stresses in the soil adjacent to the pile. These changes can be due to pile placement and loading to failure. Instead, the attention was focused on the initial in-situ horizontal and vertical stresses in the ground before pile installation.

7.2 Limiting Unit Shaft and Unit Toe Resistance

7.2.1 General

The calculation of unit shaft resistance and unit toe resistance according to Eqs. 7.3 and 7.11 indicate that both resistances increase with greater depth and proportional to the effective vertical stress. However, the early work, for example Kerisel (1961; 1964), Vesic (1964; 1967; 1970), and Tavenas (1971), indicated that the average shaft and toe resistances of a pile in a homogeneous sand deposit would increase only when the pile toe is located above a certain depth called the critical depth. For depths below the critical depth, the average unit shaft resistance and the toe resistance would remain constant.

The critical depth concept was attractive and it was rapidly included in several texts (Kulhawy, 1984). Meyerhof (1976), for instance, supported the concept and provided several design charts incorporating the critical depth concept as a fact. Furthermore, the Canadian Foundation Manual (1985) recommended the concept to use in design of piles.

An elaboration on the correctness of the concept of the critical depth is required: how the concept was arrived at and the reasoning for its existence as argued by different researches such as Kerisel, Vesic, and Tavenas.

7.2.2 Work by Kerisel

Kerisel (1964) used a relatively large circular container of 6.40 m diameter and 10.2 m depth to conduct loading tests on penetrometers of relatively large diameters as compared to other tests in the laboratory environment. The container was filled with homogeneous sand following special procedures. Penetrometers of diameter ranging from 42 mm to 320 mm were installed in sand having dry densities ranging from 1,580 kg/m³ through 1,750 kg/m³.

Fig. 7.5 shows the observed point stress versus depth of the penetrometers of different diameters showing that the point stress appears to be quasi-constant at a certain depth of penetration. Furthermore, the higher the density of the sand used in the test, the greater the observed depth of this phenomenon.

The results presented in Fig. 7.5 were based on the assumption that the penetrometers were stress free before the start of the test. In other words, that no residual stress had been induced due to installation.

7.2.3 Work by Vesic

Vesic (1964; 1970) performed both laboratory loading tests and full-scale pile loading tests. A steel pipe pile of 457 mm diameter (18 inches) made up of 5 segments of equal length was driven and tested. Before splicing on a segment, a static loading test was

performed. That is, tests were performed at embedment depths of 3.3 m, 6.6 m, 9.9 m, 13.2 m, and 16.5 m. The segments were strain-gage instrumented to allow the determination of axial loads along the pile. The pile was driven into a dominantly sand deposit.

Fig. 7.6 provides the observed variation of unit toe resistance and average shaft resistance as a function of embedment depth as presented by Vesic (1970). The indicated shaft resistance is the average value of unit shaft resistance evaluated along the whole length of the pile segment. As shown in the figure, the toe resistance and the average unit shaft resistance show approximately constant values after reaching a certain depth of penetration.

To obtain the distribution of unit shaft resistance along the pile, Vesic (1970) differentiated graphically the curve of axial load distribution along each segment. It is of interest to note that the distribution of the unit shaft resistance along the pile is parabolic as shown in Fig. 7.7. This distribution shows a concentration in the upper portion of the pile for the shorter segments and a concentration close to the toe for the longer segments. It is also of interest to note that, for the different segments, the unit shaft resistances evaluated at the same depth were not equal. For instance, the unit shaft resistance at a depth of 2 meters varied from about 20 kPa to about 100 kPa, a five times difference.

Vesic (1970) tests results confirmed earlier results of laboratory loading tests on 100 mm piles (Vesic, 1964). In both of the Vesic's studies, however, it was again assumed that the piles were stress free before loading, that is, residual loads on the piles were not considered.

7.2.4 Work by Tavenas

Tavenas (1971) presented results of loading tests on wood piles, a steel H-pile, and a hexagonal precast concrete pile. The precast concrete pile and the H-pile were driven and tested in 3 m increments to a final depth of about 21 m. The piles were equipped with telltales to allow the measurement of pile shortening. The pile shortening and the

pile properties were then used to calculate toe resistance and shaft resistance corresponding to an applied load at the pile head.

Soils at the site consisted of a surface fill layer of 7.2 m thickness, followed by a medium uniform sand layer of a thickness of about 16 m. The latter soil layer was underlain by a thin layer of very dense gravel of 0.6 m thickness overlying a stiff clay layer. The granular backfill layer was removed before driving the piles and backfilled with 4.90 m thick layer of a crushed stone.

Fig. 7.8 shows the results of applied load versus displacement of the precast concrete pile. The arrows marked with Q1 throughout Q6 indicate the failure load for each pile segment, J1 through J6, as determined by visual inspection of the load-displacement curves. Similar results were obtained for the H-pile segments.

The variation of the unit toe resistance and average shaft resistance with depth is shown in Fig. 7.9. The unit toe resistance and average shaft resistance for the reinforced concrete pile segments increased with depth up to a depth corresponding to depth-to-diameter ratio of 23 after which both resistances remained constant. The results of the steel H-pile differed from the reinforced concrete pile (the unit toe resistance and average shaft resistance showed an increase with depth).

As in the previous two studies, the piles were assumed to be stress free before the start of the loading tests, that is, residual loads were not considered in the interpretation of the data.

7.2.5 Some Explanations to the Parabolic Distribution of Shaft Resistance

Vesic (1963) argued that as a deep foundation is loaded to failure, an increase of vertical stress under the foundation base and a decrease of vertical stress "above the foundation base" taking place. The stress conditions in the soil immediately surrounding the foundation shaft may be similar to those existing in a silo or above a

"trap door" in an infinite soil mass. For this reason, the distribution of unit shaft resistance along a foundation shaft is generally parabolic.

De Beer (1963) attributed the reduction in unit shaft resistance with depth to arching taking place "close to the pile toe" which results in a decrease in the vertical stress at the toe level. This will ultimately lead to the reduced unit shaft resistance close to the pile toe.

Reese et al. (1976) presented results of axial loading tests on several drilled piers. Fig. 7.10 shows the distribution of the unit shaft resistance of a pier which was extensively instrumented. The unit shaft resistance was obtained by fitting a curve to the axial load distribution and differentiating this curve. This plot is typical for a large number of piers considered by Reese et al. (1976). The measured unit shaft resistance was very low near the toe of the pier. The shape of the distribution of the unit shaft resistance was parabolic and similar to distributions obtained in other pile loading tests (Vesic, 1970, for instance). According to Reese et al. (1976), such a distribution was caused by an interaction between the load transferred as shaft resistance in vicinity of the toe and the load transferred as toe resistance. This interaction, however, was not explained by the authors.

Kulhawy (1984) does not support the concept of limiting values for unit shaft resistance and unit toe resistance. He argued that the so called limiting unit shaft resistance can be explained as follows: most natural soils are overconsolidated rather than normally consolidated. Furthermore, the overconsolidation ratio is larger at shallow depths. Therefore, the value of the earth pressure coefficient, K , or, β , is larger for shallow depths with its maximum value close to the ground surface. This leads to a distribution of the unit shaft resistance similar to the distribution shown in Fig. 7.11. This kind of distribution will, indeed, cause the distribution of the average unit shaft resistance to show approximately constant value with depth but not a near-zero value close to pile toe.

In the Kulhawy's argument, however, there was no explanation for the limiting unit shaft resistance obtained in laboratory testing which was, most probably, carried out in normally consolidated soils.

Also Fellenius (1984; 1989b) does not support the concept of critical depth. He attributed the observed limiting values to the neglect of residual load in the interpretation of test data. Furthermore, Fellenius (1984; 1989b) suggested and used two methods to evaluate residual load in piles as shown in the following sections.

7.3 Residual Load

Residual load is the load locked in a pile when the external applied axial load on the pile head is equal to zero. The presence of residual load has been demonstrated by several researchers to be the result of pile installation and loading and unloading. In a broad sense, the presence of residual load in a pile can be the result of pile driving (Hunter and Davisson, 1969) or pushing the pile into a soil (Kerisel and Adam, 1969), consolidation settlement of surrounding soil due to external surcharge (Bozozuk et al., 1979; Fellenius, 1972), reconsolidation of surrounding soil following the installation disturbance (Fellenius and Broms, 1969; Fellenius, 1972; Hanna and Tan, 1973), and unloading a pile from a higher loading level to a lower load at the head (O'Neill and Reese, 1972).

It is important that residual load is identified quantitatively when analyzing results from loading tests to arrive at the correct sharing of total resistance between the toe and the shaft and the correct distribution of the unit shaft resistance along the pile. The neglect of residual load causes the shaft resistance to be overestimated and the toe resistance to be underestimated. Furthermore, a significant decrease will result in the interpreted unit shaft resistance in the lower portion of the pile.

7.4 Residual Load Measurements

Hunter and Davisson (1969) were the first to identify quantitatively residual loads in full-scale piles tested in sand. They presented load transfer data for six steel pipe and H-piles embedded in sand from Arkansas River Navigation Project. Table 7.2 summarizes the load transfer of the six piles included in this test program.

Residual toe loads of 25 tons through 48 tons were observed for conventionally driven piles. For instance, a 16-inch pile showed a 40 ton residual toe load. This load was about 80% of the measured toe load assuming zero residual load. Only Pile 10, also a 16-inch pile, showed no residual toe load due to installation. This pile was driven using a vibratory driving hammer.

The distribution of the unit shaft resistance as a result of the presence of residual loads was not provided by Hunter and Davisson (1969) for any of the reported piles.

Kerisel and Adam (1969) reported measurement of load transfer of a tubular steel pile (430 mm by 580 mm) in a hard clay. The pile was jacked 5 meters into the clay by applying 88 metric tons to the pile head.

Fig. 7.12 (Kerisel and Adam, 1969) shows that the removal of the pushing force left the pile compressed. The figure shows that the pile was in equilibrium under a negative shaft resistance starting from the pile head to a depth of 3 m (about 60% of pile length) and a positive shaft resistance at the lower portion of the pile and a toe resistance. A maximum residual load of more than 35 tons was evident at a depth of 3 m from the pile head. A residual load of 22 tons was measured at the pile toe after the full removal of the load on the pile head. This load is almost equal to the toe resistance of the pile.

Fellenius (1984; 1989) pointed out that immediately after a pile is installed in a soil, whatever the method of installation, and as a result of soil disturbance caused by the pile installation, a reconsolidation of the soil will take place. This was based on actual measurements of axial load distribution on two 300 mm precast concrete pile driven into a clay layer of 40 m thickness. Fellenius and Broms (1969) and Fellenius (1972) measured, immediately after pile driving, a relatively uniform increase of load along the full length of the pile. The measured load was approximately equal to the pile buoyant weight. However, with time the soil surrounding the pile reconsolidated and residual loads built up. Five months was required for the soil to fully reconsolidate.

In the Fellenius and Broms (1969) and Fellenius (1972) studies, a complete history of the mobilization of residual loads was successfully recorded. This enabled the detailed analyses of the different stages of pile driving, wait period, and the subsequent loading.

Fig. 7.13 shows the distribution of the axial load in the pile as a result of pile driving and subsequent soil reconsolidation. At zero applied load at the pile head, the pile was in equilibrium under the negative shaft resistance mobilized in the upper two thirds of the pile length, positive shaft resistance mobilized in the lower portion of the pile and the mobilized toe resistance.

From the work of Fellenius and Broms (1969) and Fellenius (1972), some important and useful observations were made. The distribution of the residual load, as mentioned above, was one of them. The other observation was that, a very small settlement was required to mobilize the residual load. A settlement of the ground surface of only 2 mm was measured in this case.

In their analysis of pile loading tests, Bozozuk et al. (1979) provided measurements of residual loads (negative and positive shaft resistances) on four instrumented steel piles in compressible silty soil. Fig. 7.14 shows the distribution of the axial load on the four piles as a result of soil consolidation due to the application of a surcharge load. The observed trend of the unit shaft resistance from negative skin friction in the upper portion and positive shaft resistance in the lower portion plus some toe resistance is very similar to the results obtained earlier by Fellenius and Broms (1969) and by Kerisel and Adam (1969).

Hanna and Tan (1973) performed loading tests on small-scale steel piles in the laboratory. The piles were placed in an empty container that then was filled with sand. This way of placing the piles in the soil is neither representative for driven nor for bored piles. However, it might be considered as an intermediate case between the two extremes. A very significant residual load was built up in the piles when the sand was placed.

In the cases presented above, the development of residual loads was obvious in different soils which included sand, soft clay, and stiff clay. The distributions were characterized by negative skin friction along the upper portion of the pile and positive shaft resistance along the lower portion. Some toe resistance was mobilized to a degree governed by the soil properties close to the toe and combining with the positive shaft resistance to maintain the pile equilibrium under zero external load.

The significant effect of the residual loads on interpretation of results of pile loading tests has been shown by several researchers. It is surprising that this was not recognized in some of the recent works such as the 1989 prediction event of axial capacity of piles which was held at the Lakefill site on the Evanston Campus of Northwestern University in connection with the 1989 Foundation Congress. Only two predictors (Fellenius; 1989 and Poulos; 1989) out of 21 attempted the analysis of the influence of residual loads on the loading tests.

7.5 Determination of Residual Load

Very little work has been carried out as to the development of methods to determine residual load on piles. Two groups of methods for quantifying residual load should be distinguished. In the first group of methods, results from loading tests on an instrumented pile are required. In the second group of methods, dynamic or static analysis are performed to estimate residual load.

7.5.1 Group 1 Methods

Available methods of determining residual load in a pile making use of results from loading tests on an instrumented pile can be divided into the following.

7.5.1.1 Direct Measurement Method

In the direct measurement method, residual loads are measured directly by reading instrumentation before and after pile driving or installation. This method is the most reliable one provided that the instrumentation used for the purpose are reliable and designed to determine the loads induced during the driving as well as afterward. That is, the instrumentation is active throughout the whole testing program; no zeroing of instrumentation is made at any stage. The work by Fellenius and Broms (1969) and Fellenius (1972) are examples of this method.

7.5.1.2 Hunter-Davisson (1969) Push/Pull Method

Hunter and Davisson (1969) showed the effectiveness of conducting a tension test (pull test) after a compression test (push test) in estimating residual load in instrumented piles. The procedure is useful for cases when all instrumentation is zeroed after pile driving and, therefore, causing the true zero to be lost. Hunter and Davisson (1969), however, assumed that a tension loading test eliminates all residual loads in the pile. This assumption is not correct (as examined in Chapter 9) because the tension test leaves the pile with tension residual load (as opposed to the usual compression residual load).

7.5.1.3 Fellenius (1990) Push/Pull Method

Fellenius (1990) presented a method to determine the range of potential residual load in a pile by making use of a compression (push) test followed by a tension (pull) test. The method is suitable for the case when the strain measurements are available for a complete loading cycle, that is, compression loading, unloading, tension loading, and unloading.

The principle of estimating the range of residual compression using this method is illustrated in Fig. 7.15. In this figure, a complete strain cycle is shown. Point O' represents the zero reading at the start of the compression test. This reading is not the true origin which is represented by point O. The difference between Point O' and Point O is the residual compression. Point A' represents the strain after the pile unloaded from the compression test. It is also the zero point for the tension test. Unloading from the tension test is shown as point B'. The difference between O' and B' represents the maximum possible residual strain.

7.5.2 Group 2 Methods

Residual loads in driven piles can be estimated by dynamic analysis (wave equation analysis, Holloway et al.; 1975). Briaud et al. (1984), Briaud (1988), and Poulos

(1987) used static analysis instead of the dynamic analysis used by Holloway et al. (1975).

As developed by Fellenius (1989b) the residual load can be estimated by means of static analysis in a simple effective stress procedure as to both magnitude and distribution.

7.5.2.1 Dynamic Analysis

In the dynamic analysis, the pile is modeled as a system of concentrated masses connected by weightless springs. The soil is modeled as a spring and a dashpot. Displacements and velocities of each of the pile segments and the forces between the segments are then computed using formulas applied by Smith (1964).

Briaud and Tucker (1984) examined this approach by applying it to the estimation of residual loads of 18 piles in sand (Table 7.3). The comparisons between the calculated and the measured residual toe loads are shown in Fig. 7.16. As shown in this figure, the correlation is very poor.

In an attempt to improve the correlation presented in Fig. 7.16, Briaud and Tucker (1984) assumed that the ultimate static resistance of a pile at the time of driving is the same as its capacity determined from the compression loading test. Further, the capacity was determined from the load displacement results as the load corresponding to a displacement equal to one tenth of the pile diameter. The revised correlation using the same data base considered in developing Fig. 7.16 is shown in Fig. 7.17. A value of 1.116 was obtained for the mean ratio of the calculated residual toe load over the measured residual load, a coefficient of variation of 0.657 was obtained. This correlation is poor and no improvement resulted from the revised analysis.

7.5.2.2 Static Analysis

Briaud et al. (1984) and Briaud (1988) proposed a procedure to estimate residual load induced in a pile during driving. The method was based on an argument that residual

load is a load that is locked-in upon unloading of a pile during driving or a loading test. The analysis, therefore, started from the condition of stress and load distribution at failure and ended after the pile unloaded to zero load applied to the pile head. In the analysis, unloading of the shaft and the toe was assumed to be linear elastic as shown in Fig. 7.18. Considering the equilibrium of the pile, an equation was derived for the residual load in a pile at a certain depth (see Briaud et al., 1984 for details of formulation). The residual load depends on pile length, pile modulus of elasticity, pile cross sectional area, axial load in the pile at the depth considered, pile capacity, pile ultimate toe resistance, and most importantly, the unloading stiffnesses for the shaft and the toe.

In an attempt to suggest expressions for the design purpose for the unloading stiffness modulus of the shaft, K_s , and the unloading stiffness modulus of the toe, K_p , Briaud et al. (1984) made use of a sizable data base to calibrate their equations for both stiffnesses. They used the Standard Penetration Test index, N , as the parameter from which both stiffnesses can be obtained. Figs. 7.19 and 7.20 show the correlation between the actual toe and shaft unloading stiffnesses, calculated from actual test result on instrumented piles, and the Standard Penetration Test index at the toe level, N_{pt} , and the average Standard Penetration Index for the shaft, N_{sdc} . The figures also show the proposed equation for the two stiffnesses. It is obvious that the results show no dependency, not even a trend. Furthermore, the Standard Penetration Test index is no more than a rough index and its usefulness as a fundamental parameter controlling the determination of residual load in a pile is questionable.

With the increase in the data base used originally by Briaud et al. (1984), Briaud (1988) proposed constant values for the unloading stiffnesses of the shaft and the toe. A value of 500 tsf/inch was assigned for the toe unloading stiffness, and a value of 10 tsf/inch was suggested for the shaft unloading stiffness.

It should be clear that this method is incapable of calculating residual loads due to consolidation similar to the case reported by Fellenius and Broms (1972) since the method does not attribute the presence of residual load to the same reasoning given in Fellenius and Broms (1969) as discussed earlier.

Theoretically, according to procedures by Briaud et al. (1984), there should be no difference between residual loads calculated for a pile due to its driving and residual loads developed due to unloading after loading in compression.

Following the same reasoning for the mobilization of residual loads as given by Briaud et al. (1984), Poulos (1987) used a more elaborate approach to quantify residual load in a driven or jacked piles. The method is based on a simplified boundary element procedure which enables a numerical analysis of loading a pile to failure in compression and then unloading back to zero load at the pile head. The determination of residual load distribution in a pile using this method requires input parameters in addition to the distribution of Young's modulus and Poisson's ratio of the soil along the pile and beneath the toe. These input parameters are the distribution of ultimate (called limiting in the Poulos's paper) shaft resistance along the pile for both tension and compression loading and the ultimate toe resistance of the pile for both tension and compression loading.

The Poulos's method is more elaborate than what Briaud et al. (1984) proposed. However, it suffers from some limitations which might affect its applicability. The distribution of the ultimate unit shaft resistance and the ultimate toe resistance are input quantities. These can be obtained from a testing program that combines compression and tension tests on an instrumented pile.

Poulos (1987) applied his method to examples taken from the literature and showed that the method resulted in reasonable values of residual load. He also included an example comparing load-displacement of a hypothetical 50 m long circular concrete pile with a diameter of 1 m placed in sand with and without residual loads. He demonstrated that the pile with residual load required smaller movement to reach the same resistance as shown in Fig. 7.21.

7.5.2.3 Fellenius (1989b) Method

Fellenius (1989b) presented a method for determining the residual load as to distribution and magnitude from the effective stress calculation of the distribution of

ultimate shaft resistance described in Section 7.1.2. The method is presented in Fig. 7.22, showing four load distribution curves in a pile of a certain embedment depth. Curve A, a "resistance curve", is the integration of the shaft resistance along the pile. Because the unit shaft resistance increases linearly with depth (it is proportional to the effective overburden stress), the load in the pile increases progressively with depth. Curve B, a "load curve", represents the load distribution in the pile immediately after applying a load to the pile head. The load is equal to the total shaft resistance according to Curve A plus a small toe resistance. The intersection between Curves A and B indicates the neutral plane or point of equilibrium between, on the one hand, the negative shaft resistance above the neutral plane and, on the other hand, the positive shaft resistance below the neutral plane plus the toe resistance. The shaft resistance is fully developed, but, because the movements are very small, the toe resistance is only partially developed.

In reality, the change from Curve A to Curve B occurs over a transition length. Fellenius (1989b) constructs this by means of a vertical line marked E in Fig. 7.22. The residual load in the pile is then the shaded area between Curve A, Line E, and Curve B. That Line E is vertical means that no residual shear acts in the transition zone, that is, the residual load does not change along this length of the pile.

In a static loading test to failure, all the shaft resistance acts in the positive direction and the toe resistance is fully developed. The load distribution is then according to Curve C which is parallel to Curve B.

If the pile is equipped with gages to measure load and the gages are assumed to read zero before the test, then, Curve D will be plotted from the measurements. Curve D is obtained from Curve C, the true load distribution in the pile, when subtracting the residual load.

7.6 Effects of Pile Installation

Considerable changes take place in the density and state of stress of cohesionless soils due to pile driving. These changes are radial compaction or loosening, axial

compaction of the soil in the vicinity of the pile toe, shearing along the shaft, bearing failure of the soil close to the pile toe, and rotation of the principal stresses. This effect is much greater in the soil close to the pile shaft and pile toe. Density changes are much smaller for soil at larger distance from the pile shaft. All of these changes depend on the pile and soil properties such as pile size, soil initial density, shear strength, compressibility, and volume change characteristics of the soil.

Several researchers investigated the zone of soil affected by pile placement into sand soils. The work conducted by Platema and Nolet (1957), Meyerhof (1959), Robinsky and Morrison (1964), Broms (1965), Kishida (1967), Petrasovits (1973), VanWeele (1979) and Davidson et al. (1981) are some examples.

Platema and Nolet (1957) compared cone sounding resistance in a loose sand before and after compacting a cast-in-place concrete pile. For the sand located directly under the pile toe, the affected zone extended to a distance of about five times pile diameter. Close to the pile toe, the cone point stress was increased by four times due to pile placement. At a distance of three pile diameters below the pile toe, the point stress increased to 1.5 of its original value.

Robinsky and Morrison (1964) used circular metal model piles of 36 mm diameter and 0.50 m length to study sand displacement and compaction caused by pile placement. Piles of different surface properties (smooth or rough), and sand of density index (relative density) of 17% or 35% were included in the study. A special radiography technique was used to determine the extend of the soil zone which is affected by the pile placement.

The displacement of sand particles was considered as the indication of the extent of the affected zones. In the case of looser sand, a zone extending about 3 pile diameters away from the pile shaft was affected and a zone also extending about 3 pile diameters below the pile toe level was affected. In the case of denser sand, these zones were about 5 and 3.5 pile diameters, respectively. For both soil densities, the piles with rough shafts produced larger zones of affected soil.

The study by Robinsky and Morrison (1964) indicated also the existence of zones located along the pile shaft with erratic changes in sand density. These include both higher and lower densities as compared with the initial density before pile placement. A similar observation was made earlier by Szechy (1961).

The displacements and strains produced by pile driving in the surrounding soil is shown in Fig. 7.23 (Robinsky and Morrison, 1964). The soil under the pile toe experienced vertical compression and horizontal expansion, while the soil adjacent to the pile shaft experienced vertical expansion. The largest vertical displacements were concentrated within a quarter pile diameter distance from the pile shaft. Moreover, the densification of the soil along the pile was very irregular as shown in Fig. 7.24.

Broms (1965) assumed that, due to a pile driving, the zone of influence extend 5-6 diameters from pile shaft and 3-5 pile diameters below pile toe.

Kishida (1967), suggested that the zone of soil affected by pile placement is 3.5 pile diameters from the pile shaft and below the pile toe.

Displacement field and volume changes taking place in dense and loose sands due to jacking a cone penetrometer were studied by Davidson et al. (1981). A dry sand was used in this study. Stereo photogrammetry was used to measure displacement at different locations within the sand. As shown in Fig. 7.25, for the test involving dense sand, downward and radial (away from the pile) movements are evident for the sand located under the toe. On the other hand, the sand along the shaft moved upward and away from the pile. Denser sand was observed along the pile shaft and looser sand was observed under the pile toe.

Displacements were concentrated within a distance of 2 times the pile diameters in the loose sand and 4.5 times the diameter in the dense sand along the shaft. Below the toe, the zone of influence extended between 2 and 2.5 times the pile diameter for loose and dense sand, respectively.

The jacking of the cone penetrometer into the loose sand produced results that differed from those of jacking into the dense sand. For the loose sand, the density increased in

the zone below the toe, while an erratic densification was observed along the shaft.

Ekstrom (1989) compared the extend of soil zone affected by pile placement in sand as obtained in different studies. Fig. 7.26 shows this comparison. It is obvious that a great deal of variability exists in the results presented in this figure. This might suggest that there exists extreme difficulty in quantifying effects of pile driving into sand.

7.7 Scale Effects

De Beer (1963) defined the scale effect as the influence of the ratio of the transverse dimensions of the pile to the diameter of the base of the cone utilized in the test. In this definition, De Beer was concerned with the use of penetration testing to estimate the bearing capacity of large-scale piles. This kind of scale effects have been discussed by Kerisel (1964), Vesic (1969), and, more recently, Meyerhof (1983).

Kerisel (1961; 1964) showed that a remarkable scale effect exists when tests were carried out in dense sand. Negligible scale effect was observed, at the other hand, when the same tests were carried out on loose and medium sands. De Beer (1963) criticized the conclusions made by Kerisel (1961) with regard to the tests in dense sand. He attributed the observed severe scale effect to the fact that these tests were conducted with extreme case of a dry, very dense sand, with a very low stress level (no surface surcharge was used in these tests). The Kerisel's analysis, as explained earlier, did not include residual loads on the piles.

Franke (1981) correlated test results from relatively large model piles to that of large diameter piles. In this investigation, Franke pointed out that no general conclusion was possible. Residual loads were not addressed and the critical depth concept was taken as a fact.

In a more recent contribution, Meyerhof (1983) compiled a large number of available data to quantify the scale effects. However, considerable scatter is evidenced in all of the presented data. For driven piles in sand, a very pronounced scale effect was observed as shown in Fig. 7.27. The data presented in this figure suggest that the larger

the driven pile, the smaller the ultimate unit toe resistance. Meyerhof (1983) selected a pile diameter of 0.5 m as a reference size and presented the results of the compiled data as pile diameter versus a reduction factor to be applied for the ultimate toe resistance as presented in Fig. 7.28. The Meyerhof's suggestions included in this figure indicate that, for an example, a one meter diameter pile driven in a dense sand would have less than one half the unit toe resistance of a half metre diameter pile driven into the same soil.

The work presented by Meyerhof (1983) suggested also that the unit shaft resistance of driven and bored piles in sand of a given density is practically unrelated to the pile diameter. Furthermore, he also found that the shaft resistance measured using cone penetrometer can be used directly to estimate unit shaft resistance. The decrease in the toe resistance is explained by a reduction of angle of internal friction with increasing overburden pressure due to grain crushing and soil compressibility. However, no analysis, was included or referred to support this explanation.

Yazdanbod et al. (1984) studied the performance of instrumented model pile tested in a special laboratory device. The device was capable of applying different horizontal stresses to simulate tests at large depths. For a lateral pressure corresponding to stresses subjected to a 30 m long pile on dry sand, no evidence of grain crushing was observed for the sand close to the shaft of the pile. These results suggested that, if the crushing of grains is taken as a measure of the magnitude of stresses to which the soil was subjected due to pile loading to failure, the scale effect on unit shaft resistance is less than that on toe resistance.

The noticeable scatter in the data presented by Meyerhof (1983) suggests the difficulties involved in quantifying the scale effects. This might be due to the extreme difficulty involved in scaling soil behavior for different stress conditions. As shown in Chapters 3, 5, and 6, cohesionless soils, for instance, respond in a different manner when tested under different stress levels. Loose sands can even dilate when subjected to shear under small mean pressure as demonstrated by Seed and Lee (1967). On the other hand, dense sands can suffer a large reduction in volume (compress) when sheared under high mean pressures as shown by Vesic and Clough (1966) for the Chattahoochee River sand and Seed and Lee (1967) for the Sacramento River sand.

7.8 Variation of Ultimate Resistance with Time

In his comments on the Vesic's (1970) field loading tests, Tavenas (1970) pointed out that the time allowed between completion of driving and load testing was only 12 h and considered this as a relatively short period.

Datta (1982) reported that relatively short time periods are required to dissipate the excess pore water pressure caused by pile driving. He showed that the excess pore pressures disappear within 5 minutes in a coarse sand and within 45 minutes in a silty fine sand. The largest pore pressure changes corresponded to 20% of the overburden pressure.

To illustrate the time effect on bearing capacity, Tavenas (1971) presented a variation of bearing capacity with time for 15 pile tests. As shown in Fig. 7.29, the bearing capacity increased during 20 days after driving (the bearing capacity in Fig. 7.29 normalized to the capacity at 12 h after driving). The phenomenon of increasing bearing capacity with time, according to Tavenas, can not be explained by the changes taking place in the pore pressure, instead, he attributed it to changes in the sand structure around the pile. In addition, these changes in the structure may also affect the behavior of the pile-soil system during loading and, possibly, partially be a cause of residual load.

CHAPTER EIGHT

AXIAL LOAD TRANSFER FOR PILES IN SAND:

TESTS ON AN INSTRUMENTED PRECAST PILE

8.1 Introduction

A pile test programme was carried out in 1984 at the Baghdad University Complex, Iraq, close to the bank of the River Tigris. An instrumented test pile consisting of a 285 mm square, precast concrete pile was driven 11.0 m into a sand deposit and subjected to three axial compression and one tension static loading tests.

The purpose of the test was to obtain information for use in the design of pile foundations for the expansion of the university campus.

By means of strain gage instrumentation, the load imposed in the pile during the tests were determined. The observed load distribution appeared to suggest the existence of a critical depth. The shaft resistance degraded slightly from test to test. There was no clear indication that the tension shaft resistance is smaller than that the compression resistance.

This chapter describes the site condition and the loading test programme, and presents the test results. The Fellenius (1989b) method for determining residual load, described in Chapter 7 is applied in the analysis of the test pile.

8.2 Soil Profile

The soil profile next to the test pile location consisted of two main soil layers: one upper, 3.0 m layer of clayey silty sand deposited on a lower, thick layer of uniform sand with some silt. Fig. 8.1 shows the particle size distribution envelopes of the two layers.

The area is arid and the groundwater table fluctuates seasonally. The pore pressure below the groundwater table is hydrostatically distributed. Table 8.1 shows that the depth to the groundwater table varied between the testing occasions between 6.36 m and 5.02 m below the ground surface. The groundwater table depths shown in Table 8.1 were measured by means of a standpipe piezometer with a filter tip placed at a depth of 11 m. Notice that the groundwater table rose during the time between the compression and tension testing.

Fig. 8.2 illustrates the Standard Penetration Test SPT index (N-value) obtained from a borehole located 7 m away from the test pile. The N-index in the lower layer varied between 12 and 25 blows/0.3 m with an average of about 15 blows/0.3 m in the 3 m through 11 m zone, which classifies the compactness of the soil to a compact (medium) condition.

Fig. 8.3 presents the results from two friction-jacket cone penetrometer tests put down at a distance of 2.5 m from the test pile. The cone point stress, q_c , is shown in Fig. 8.3a, the local friction, f_s , in Fig. 8.3b, and the friction ratio, f_s/q_c , in Fig. 8.3c. The data are quite scattered. The boundary between the two soil layers is discernible, however. Further, both the cone point stress diagram and the local friction diagram indicate a looser soil at a depth of about 9 m. The average friction ratio in the sand layer between the depth of 10.0 m and 12.0 m is 2.4 percent. This value is larger than the ratio normally found in a sand.

8.3 Test Pile and Instrumentation

The test pile was a square, nominally 285 mm diameter, 12.0 m long, precast concrete pile reinforced with four 22 mm bars with a yield strength 420 MPa. The concrete largest aggregate size was 20 mm and the concrete 28-day cube strength was 45 MPa. The axial tensile strength of a cracked pile section depends only on the strength of the steel reinforcing bars, which combined strength is about 600 kN. The total weight of the pile is about 20 kN.

The pile was equipped with load cells of a type modified from the original Mustran cell used for instrumentation of cast-in-place piers (Reese et al., 1976; Reese, 1978). Each cell consisted of a 16 mm diameter, 200 mm long steel bar on which electrical resistance strain gages are affixed. The strain gages were sealed and the bars covered with a water-tight rubber hose. The dimensions were designed to give the finished cell an axial stiffness similar to that of concrete.

In the laboratory, the cells work equally well in compression and tension. However, when placed in the pile and subjected to tension they will cease to provide correct load data once the tensile strength of the pile is reached and the concrete starts to crack. Naturally, when a crack occurs near a cell, its calibration is lost and its reading becomes unreliable. For a driven pile, such as the test pile, cracks can also be caused by the driving

Each cell to be placed in the pile was calibrated in the laboratory under static, repeated, and sustained loading. A straight line relation was observed between strain and applied load and no zero-drift was indicated for loads sustained for 48 hours.

Before pouring the concrete to make the pile, eight cells were placed inside the reinforcing cage in the center of the casting form and parallel with the pile longitudinal axis. The cells were placed at a center-to-center spacing between cells of 1.5 m with the lowest cell 0.5 m above the pile toe and the uppermost cell 1.0 m below the pile head. Thus, with the pile embedment length of 11.0 m, the uppermost cell, Cell 1, was level with the ground surface.

After pouring and curing of the concrete, no more individual load-cell calibration was made. Instead, Cell 1 was used as benchmark for determining the relation between the measured strain and the load in the pile, that is, a benchmark toward the EA-value of the pile. (E is modulus of elasticity and A is pile cross sectional area; No resistance acted on the pile between the pile head and Cell 1).

8.4 Driving Data and Test Arrangement

The reaction to the static loading was obtained from four piles driven before the test pile to 11 m depth in the corners of a rectangle with long and short sides of 5.0 m and 2.5 m, respectively. The test pile was driven in the center of the rectangle.

The piles were driven on June 17, 1984 by a Delmag D12 diesel hammer. All five piles had a gradually increasing penetration resistance with depth. The test pile penetration resistance at end of driving was 4 blows/25 mm. No restriking was performed.

The tension test reaction was obtained by means of two hydraulic jacks reacting against a girder resting on concrete blocks on the ground.

The compression tests were conducted by means of a hydraulic jack reacting against heavy beams connected to the four anchor piles. Vertical movements were measured with two diametrically opposed dial gages having a 0.01 mm gradation. The applied load was determined by means of a separate load cell.

8.5 Test Procedure and Pile Head Load-Movement Data

The first compression loading test consisted of applying load in increments of 100 kN and maintaining each load level for about one hour up to a total load of 1,000 kN and unloading from this level in decrements of 200 kN.

The second compression loading test was undertaken one week after the first (see Table 8.1) and consisted of reloading the pile in increments of 250 kN to the same maximum load, 1,000 kN. Unloading was in decrements of 250 kN.

The third compression loading test was undertaken two weeks after the second test. The pile was again loaded in increments of 100 kN. At a load level of 900 kN, the movements became progressively larger and the following increments were reduced to 50 kN. The maximum load applied was 1,100 kN, which load was reached at a pile head movement of 120 mm and could not be maintained.

Five months after the third compression test, a tension test was conducted with the pile loaded by tension increments of 50 kN. When the applied load had reached 400 kN, movements started to become progressive. The maximum load applied was 580 kN, at which load the upward pile head movement was 65 mm and the load could not be maintained.

Fig. 8.4 presents the load-movement diagram from the four static loading tests. Failure did not occur in the first two loading tests, that is, because full shaft resistance was mobilized, the toe resistance was not fully developed. In the third test, failure occurred between 900 kN and 1,000 kN. In the fourth test, the tension test, failure appears to be starting to occur at about 500 kN and be fully developed at 580 kN. It may be noted that the tensile failure load is uncomfortably close to the mentioned tension strength of the reinforcement. However, no cracks were visible in the shaft portion of the pile that was above ground.

8.6 Measurement Results

The strain recorded by uppermost pile cell, Cell 1, was matched to the applied load to calibrate the pile modulus. For small load and up to a load of 300 kN, the induced strain was a curvilinear function of the load. A strain-load relation was established by curvilinear regression giving a regression coefficient of 0.999. In the interval between 300 kN and 1,000 kN, the load-strain relation was linear with a regression coefficient of 0.999.

The pile modulus, E , at the location of Cell 1 resulting from the linear relation was 35 GPa. The load in all other pile cells was determined by taking the recorded strain

times the EA-value determined from Cell 1, where A is the cross sectional area of the test pile.

The assumption of constant modulus in the pile implants an error, of course, because the pile modulus along the pile must vary. Furthermore, in the saturated zone below the groundwater table, the modulus is expected to have increased beyond the modulus for dry conditions.

In the tension test, the loads rapidly exceeded the tension strength of the concrete. Therefore, the data from the uppermost cell do not correctly represent the conditions for the cells lower down the pile. Despite that the cells nearest the pile toe received tension loads smaller than the concrete strength, no calibration can be established. Consequently, no reliable calculation of the loads near the pile toe is possible for the tension test.

The strain induced by the loading was determined to one unit of microstrain. One microstrain calibrates to a load of 35 kN. About half of this value is the maximum precision of the data and also the minimum error involved.

One cell, Cell 3, did not survive the pile driving, but all other cells worked and provided consistent data. However, the load values obtained from Cell 2, located 1.5 m below the ground surface, were consistently smaller than reasonably expected. For example, at the applied load of 1,000 kN, Cell 2 indicated a load of 937 kN, that is, a shaft resistance of 63 kN, which is an unrealistically high value. (To obtain this amount of shaft resistance in an effective stress analysis (see below), a beta-value as high as 3.2 must be assigned to the soil between the ground surface and to a depth of 1.5 m, which is 6 to 10 times larger than commonly assigned values). However, in the interest of maintaining a consistent treatment of all measurements, no attempt was made to "correct" the readings taken by Cell 2 when plotting the load data.

Fig. 8.5 presents a plot of the loads measured in the test pile during the first compression test. In eyeballing the diagrams, the results appear to suggest that down to a depth of about 7 m, the load in the pile decreases progressively, which means that the unit shaft resistance can be stated to be approximately proportional to the effective

overburden stress. Below about 7 m, the load distribution is linear, which would mean that the unit shaft resistance is constant, that is, the normal stress against the pile is constant and independent of the effective overburden stress. A constant unit shaft resistance is unlikely because it would mean that the basic physical principle of resistance to sliding movement being a function of normal stress is invalid, which would be hard to accept. However, similar results are after all available in the technical literature. For example, Vesic (1970), Tavenas (1971), and Meyerhof (1976). The Canadian Foundation Engineering Manual (1985) quotes such results and names the depth where the unit shaft resistance becomes constant "the Critical Depth".

Further, the diagram in Fig. 8.5 implies that the maximum load at the pile toe was about 190 kN. The balance, 810 kN, to the 1,000 kN maximum load occurring in the compression test is shaft resistance. The tension resistance is, of course, all shaft resistance. Therefore, the results appear to indicate that the shaft resistance in compression is much larger than the shaft resistance in tension. In contrast to the Critical Depth "conclusion", this finding does not violate fundamental principles, but is it a correct conclusion?

Figs. 8.6 and 8.7 present plots of load measured in the test pile during the second and third compression tests respectively. The results shown in these two figures do not differ significantly from that obtained for the first compression test as shown in Fig. 8.5.

8.7 Preliminary Evaluation of Measured Results

In most field testing programmes of instrumented piles in sand reported in the literature (e.g., Vesic, 1970; Tavenas, 1971), it was assumed that the residual load due to pile installation was negligible. Therefore, when planning the present study, no attempt was made to measure the residual load resulting from the installation of the test pile, and all gages were zeroed before commencing each loading test.

A preliminary analysis of the field test data using a bearing capacity equation, gave an apparent toe bearing capacity factor of about 10. The toe bearing capacity factor

calculated in this way is much smaller than any value reported in the literature for piles driven in sand. This discrepancy raised questions about the validity of the assumption that the residual load due to pile installation was negligible. Furthermore, the unit shaft resistance obtained from the measured test data was a function of the overburden stress only at shallow depth. Deeper down, it appeared to be about constant suggesting the presence of a critical depth. However, any conclusions drawn from the load distribution shown in Figs. 8.5 through 8.7 include the mistaken assumption of no load being present in the pile at the start of the test. In fact, the pile is subjected to a considerable load from the soil before any load is applied to the pile head.

As discussed in Chapter 7 and explained by Fellenius (1984; 1989a), because of the stiffness difference between the pile and the soil, there will always be a transfer of load between the pile and the soil whether or not there is a load applied to the pile head. In the upper portion of the pile, load is transferred from the soil to the pile and in the lower portion the load is transferred from the pile to the soil. At the neutral plane, there is an equilibrium of the load between the two portions. The load transfer is associated with small movements of the soil relative to the pile. As already a very small relative movement is sufficient to mobilize the shear strength of the soil, the induced shear forces along the pile can be assumed equal to fully developed shaft resistance. The load induced in the pile is called "residual load".

A wave equation analysis has been performed by means of the GRLWEAP program (GRL, 1990) with input values of quake, damping, and hammer characteristics as recommended in the GRLWEAP manual for sand soils. The program was run with a linearly increasing unit shaft resistance with a total shaft resistance of 400 kN chosen low in order to represent the temporary breakdown of static shaft resistance occurring during initial driving in most soils.

The capacity developed in the test agrees reasonably well with the capacity computed by the wave equation analysis. For the observed penetration resistance of 4 blows/25 mm, the bearing graph obtained by means of the wave equation analysis indicated an ultimate resistance in the range of 700 kN to 800 kN, that is, a toe resistance ranging from 300 kN through 400 kN. When repeating the analysis with the toe quake value increased from the manual recommended value of 2.5 mm to 5.0 mm

and reducing the value of hammer efficiency from 0.72 to 0.6, the computed capacity boundaries decreased by about 100 kN. As soil set-up is common also in sand soils, in restriking the pile, the penetration resistance might well have increased to about 5 blows/25 mm or 6 blows/25 mm. Assuming that the shaft resistance at restrike has increased to 600 kN, the analysis results indicated a capacity in the range of 1,000 through 1,200 kN, suggesting a toe resistance in the range of 400 kN through 600 kN at the time of the static loading test. The results of the wave equation analysis of the driving data were used as guide at the outset of the static analysis described below.

Had a restrike been carried out after the static testing with careful observation of the penetration for the first couple of blows, the wave equation would have been approximately calibrated to the site conditions. Then, wave equation analysis would have become a useful tool in the inspection and quality control in the driving of the construction piles.

8.8 Analysis of Test Data Using Fellenius (1989b) Method

To analyze the test data using the Fellenius (1989b) method described in Chapter 7, one starts with assuming a shear strength distribution along the pile, a residual toe load, and a pile toe resistance. These assumptions govern the residual load in the pile and when adding the residual load to the measured load values, a load distribution is determined which is then compared to the assumed values. The process is iterative and, through a trial-and-error approach selecting different soil parameters and residual load values, a final solution is obtained wherein measured and calculated load distributions agree.

For the analysis, the effective stress method, beta-method, described in Chapter 7 was applied with the shaft and toe resistances controlled by only the effective overburden stress (no cohesion intercept).

The effective overburden stress is a function of the density of the soil at the site. Density was not determined, however, and has to be assumed. The sand layer was taken to consist of three layers with different density; the upper, the lower, and an

intermediate layer. The value assumed in the calculations for the upper layer was $1,600 \text{ kg/m}^3$, which is dry density, as applicable well above the groundwater table. The lower layer, the sand layer, has a dry density of $1,800 \text{ kg/m}^3$ and a total saturated density (applicable below the groundwater table) of $2,000 \text{ kg/m}^3$. A density of $1,900 \text{ kg/m}^3$ was assigned to the intermediate layer, a 1.2 m thick zone above the groundwater table, to account for partially saturated soil.

The beta values that resulted from the iterative calculations are compiled in Table 8.2. The table also includes the toe bearing capacity coefficient, N_t , resulting from the calculations. The calculations were performed by means of the UNIPILE program (Goudreault and Fellenius, 1990). No compensation for the buoyant pile weight (13 kN) was included.

Table 8.2 does not include any change in beta-value in the saturated sand above and below the 8 m depth despite that a change in strength was suggested by the static cone penetrometer data. Attempts were made to fit the theoretical distributions to the measured loads using a higher beta-value in the upper 1.5 m and a lower value below 8.0 m, but no good fit was accomplished. Furthermore, it was not possible to achieve the fit by applying a smaller beta-value when calculating negative shaft resistance than when calculating positive shaft resistance.

The values in Table 8.2 resulted in a calculated total capacity of 1,027 kN, made up of 667 kN of shaft resistance and 360 kN of toe resistance. Fig. 8.8 presents the theoretical "true" load distribution in the pile, Curve I. The diagram also shows measured load value (small circles) determined according to the principle given in Chapter 7 (Fig. 7.22) and a residual toe resistance of 170 kN with the residual positive shaft resistance (Curve B in Fig. 7.22) calculated using the same beta-values, the vertical portion (Line E in Fig. 7.22) matched to the measured load values, and a curve, Curve II, representing the theoretical "measured" load distribution.

The curves shown in Fig. 8.8 are unique. That is, only the values given in Table 8.2 will satisfy the conditions that the same unit shaft resistance be used for both the residual load and the load distribution in fitting the calculated "measured" load distribution to the actually measured load values. Changing the beta-values in the moist

and saturated sands by more than 0.02 up or down and/or the N_t -value by more than 2 units will destroy the fit.

As mentioned, the measured load values include small errors and are only correct within, perhaps, 20 kN. Furthermore, the density values are only correct within about 100 kg/m³. Therefore, the beta-values given in Table 8.2 are only reliable within the first decimal and the N_t -value within a few units.

The beta-values in Table 8.2 agree well with recommendations given for design by Fellenius et al. (1989a). The N_t -value of 30 is smaller than the recommendation, however. It is of interest to note that the ratio between the beta-value at the pile toe and the N_t -value is 2.2 percent, which is very similar to the static cone penetrometer friction ratio of 2.4 percent.

The primary result of the analysis, however, is not the actual beta and N_t values, but the clear indication that the pile is subjected to residual load and that neglecting this in any evaluation of the test data would result in completely erroneous conclusions of the test results.

Figs. 8.9 through 8.11 present the test data for all three tests without and with the correction for residual loads established from the first test. The diagrams have been supplemented with the calculated "measured" load curve (Fig. (a) diagrams) and with the calculated distributions of "true" load and residual load (Fig. (b) diagrams). No attempt was made to adjust to the obvious fact that the first test changed the residual load in the pile for the second test and the second test again changed the residual load in the pile for the third test. The changes consist mainly of increased residual toe resistance and are indicated in the diagrams by the progressive loss of agreement between the data of the second and third tests and the distributions calculated for the first test.

The gross pile head movement at the applied load of 1,000 kN and the net movement after unloading are presented in Table 8.3.

The compression of the pile is small in relation to the movement of the pile. When combining the load distributions for the applied load of 1,000 kN for the three tests with the elastic modulus of 35 GPa, the compression of the pile induced by the test load calculates to about 2 mm. Deducting the pile compression from the pile head movement indicates that the gross and accumulated toe movement at Tests I through III for the applied load of 1,000 kN was about 8 mm, 19 mm, and 61 mm, respectively, or in relation to the pile diameter: 4 %, 7 %, and 18 %, respectively. Obviously, the toe movement in the first two tests was not large enough to mobilize fully the toe capacity.

As judged from Fig. 8.4, the load-movement diagram, failure had not occurred in the first test at the applied load of 1,000 kN. Yet, in the third test, the pile clearly failed at the 1,000 kN load. However, the first test had almost reached failure and a small increase of the load would probably have showed this. In the third test, the shaft resistance had deteriorated somewhat and the toe resistance was appreciably engaged already before the start of the test. Consequently, the pile reached failure at the 1,000 kN load.

At the time of the tension test, the groundwater table had risen to a depth of 5.02 m. The rise resulted in a reduced effective stress. The calculated reduction of shaft resistance due to the subsequent reduction in effective stress is 60 kN and the new shaft resistance is 607 kN. This value is higher than the tension capacity of 500 kN to 580 kN. The difference can be explained by degradation of the sand strength induced by the compression tests. However, the tested tension capacity is very much in doubt, as mentioned above, and the apparent smaller tension resistance cannot be taken as any reliable indication of that the negative shaft resistance is smaller than the positive shaft resistance.

The rise of the groundwater table from a low of at least 6.7 m in October 28 to 5.0 m measured at the time of the occasion of the tension test in February 5 (Table 8.1) caused a reduction of effective stress of 17 kPa. This unloading of the soil must have resulted in a slight expansion of the sand. Considering a probable elastic modulus of 100 MPa for the sand, the expansion would be a movement of about 0.2 mm/m in the saturated sand accumulating up and down from the neutral plane. This movement is

small, but although it is too small to change the direction of the forces along the pile, it must have had some reducing influence on the residual load in the pile. The actual amount of the reduction, however, is not known.

8.9 Confirmation Test

To verify the analysis results, a second test pile, Pile 2, identical to the first test pile, Pile 1, but driven 4 m deeper (embedment 15.0 m), was installed 5.0 m away from the first pile and tested in an identical procedure. The first two stages (with four weeks interval between the tests) consisted of loading the pile in increments of 100 kN to a maximum load of 1,000 kN and unloading. In the third stage one week after the second stage, the pile was loaded to failure. The maximum load was 1,600 kN and failure occurred when increasing the load from 1,500 kN to 1,600 kN. Fig. 8.12 presents the load-movement diagram from the tests on Pile 2.

The beta-values and N_t -coefficient given in Table 8.2 were applied unchanged to the longer pile and used to calculate (by means of the UNIPILE program) the capacity of the pile. The calculated capacity was 1,650 kN, which agrees well with the maximum load applied in the static loading test, 1,600 kN.

8.10 Final Remarks

The procedure recommended by Fellenius (1989b) for using an effective stress analysis to determine the distribution of residual load in an 11.0 m long instrumented test pile was successfully applied to the test data proving that the pile was subjected to residual load.

The data were corrected for residual load and the beta-values and the toe bearing capacity coefficient in the soil at the site were determined.

Without the correction for residual load, considerable errors in the evaluation of the test results with regard to shaft and toe resistances would have resulted. As to the toe resistance, the error would have been a 50 percent too low value -190 kN instead of 360 kN- which error is due to the neglect of a residual toe load of 170 kN.

The analysis indicated no difference between the shaft resistance in tension and the shaft resistance in compression.

Once the residual load was added to the measured loads, no Critical Depth was discernible.

The same beta-ratios and N_t -factor (Table 8.2) were applied to the results of compression tests on a second test pile, a 15 m long identical pile, and the calculated capacity agreed with the capacity found in the static loading test suggesting that the procedure of evaluation applied to the first test has general validity.

CHAPTER NINE
AXIAL LOAD TRANSFER FOR PILES IN SAND:
NUMERICAL ANALYSIS

9.1 Introduction

The load transfer behavior of the 11 meter long test pile described in Chapter 3 was modelled using the finite element analysis developed for the subject research work. The analysis was performed in a continuous manner for all stages of the testing programme: the pile installation, the three compression tests in sequence, and the tension test. The numerical results from each event provided the initial state of stresses and strains in the modelling of the subsequent event.

The soil model (Bardet, 1986; 1987) used in the finite element program was chosen to account for the behavior of soil under repeated loading conditions. One set of model parameters was used for each soil layer for all stages of the numerical analysis.

In order to model correctly the behavior of the test pile, it was necessary to include the effect of pile installation. A new procedure was developed for the calculation of the residual stresses and strains in the soil-pile system resulting from the pile installation.

As in the conventional approach described in Chapter 7 and applied in Chapter 8, a good agreement was obtained between the finite element results and the measurements only when the effect of pile installation was included in the analysis.

9.2 Model Parameters

The in-situ tests reported in Chapter 8 included two friction-jacket cone penetrometer tests and a standard penetration test. At present, there is no method which can provide the parameters of advanced soil models from such tests. However, since the objective here is not to validate the capabilities of the constitutive model, which has already been done in Chapters 5 and 6, the model parameters required for the present study were determined as follows.

The soil profile was divided into the same four layers given in Chapter 8 - an upper, 3.0 m layer of clayey silty sand, and a sand layer consisting of three loose to compact layers with the same dry density but with different degree of saturation; the upper (dry sand), the lower (saturated sand), and an intermediate layer (moist sand; see Table 8.2). For each layer, the parameters were based on the results of the friction-jacket cone penetrometer tests, and the experience with the sensitivity of the model simulations to each one of its parameters (Altaee et al., 1991). The model parameters assigned to each soil layer are shown in Table 9.1.

The soil parameters of the model were divided into three groups. The parameters in the first group included the effective ultimate angle of friction in compression, the effective ultimate angle of friction in extension, and the effective peak angle of friction. Since no triaxial compression and triaxial extension laboratory tests on samples from the site were available to determine these parameters, the following assumptions were made for the sand at the site: No strain-softening characteristic was assumed (when sheared in a stress path corresponding to the conventional drained triaxial test). As a result of this assumption, the peak angle of friction became equal to the ultimate angle of friction along the triaxial compression stress path. The preceding assumption regarding the two angles of friction has been found reasonable for loose to compact sand as in the present case (Lee and Seed, 1967; Tatsuoka and Ishihara, 1974; Altaee et al., 1988; and Evgin et al., 1989). Furthermore, the effective ultimate angle of friction in extension was assumed to be equal to that in compression.

A relationship developed by Durgunoglu and Mitchell (1975) between effective angle of friction and cone resistance for different vertical effective stresses was used to

estimate the peak effective angle of friction. According to this method, the angle of friction varied between 32° and 36° for the sand below the 3 m upper layer. This method gives a reasonable lower bound of angle of friction for sands with the quartz fraction dominating as reported by Robertson and Campanella (1983). No further refinement was attempted to modify the effective angle of friction. Thus, a value of 34° was assigned to the second and third soil layers, whereas a value of 36° was used for the last soil layer, as shown in Table 1. For the 3 m upper silt and sand layer, an estimated value of effective angle of friction of 32° was used.

The second group of parameters included the Poisson's ratio which was assigned a value of 0.3. This is a typical value used by others (e.g., Bardet, 1987). The second parameter in this group was the hardening parameter which was assigned a value of 1. This parameter has little effect on the overall performance of the model. The third parameter in this group was the bounding-surface aspect ratio which was assigned a value of 2.0. Previous experience with the model has indicated that an aspect ratio equal to 2.0 is suitable for loose to compact sand as in the present case.

The parameters in the third group are related to the compressibility of the soil and they were selected from the literature for similar soils.

Using the model parameters in Table 9.1, the behavior of the saturated sand was simulated in an isotropic compression test and three conventional triaxial drained compression tests at different confining pressures. Fig. 9.1 shows the simulated isotropic stress versus volumetric strain relation in a loading and unloading path. The simulations of the conventional triaxial tests are shown in Fig. 9.2. The general trends in these simulated cases are similar to those obtained from actual testing of different types of loose sand (Lee and Seed, 1967; Tatsuoka and Ishihara, 1974; Altaee et al., 1988; and Evgin et al., 1989).

9.3 Initial Stresses in the Soil

Initial stresses are important for the nonlinear analysis of soil masses or soil-structure interaction problems. Previous finite element studies of piles, (e.g., Desai and

Holloway, 1972; and Desai, 1974) have not included the presence of residual load. Instead, they back-calculated the initial horizontal stresses in the soil from the results of analysis of field loading tests. This means that the horizontal stresses at failure were assigned as the initial horizontal stresses in the analysis of the loading tests. Furthermore, in order to get the calculations to match both the shaft resistance measured in compression and in tension, different initial horizontal stresses were used in the analysis of compression loading and tension loading.

In the present work, in contrast, the initial soil stresses employed in the analyses are those before the pile installation (calculated using the densities and the groundwater table location reported in Chapter 8).

The calculations assumed that the soil was normally consolidated. The Jaky's equation was used to calculate the coefficient of earth pressure at rest, K_0 . Values of K_0 of 0.47, 0.44, 0.44, and 0.41 were obtained for the first, second, third, and forth soil layers, respectively. The initial (before pile driving) effective horizontal stress at a specified depth was then calculated as K_0 times the effective overburden stress at that depth. Since the soil was at-rest condition, no shear stresses were present in horizontal and vertical planes.

9.4 Finite Element Idealization of Soil-Pile System

The soil pile system was modeled using an axisymmetric idealization. The selection of the mesh was based on convergence requirements (Desai and Abel, 1972). A parametric study was carried out to find the optimum mesh size. Further refinements in the mesh size showed negligible differences in the computed load versus pile head movement response of the test pile. Far-field soil boundaries were selected using the same principle.

The length and the axial stiffness of the idealized pile were of course the same as those of the test pile. However, in order to have an axisymmetric idealization of the pile soil system, the square test pile had to be approximated as a circular pile. The selection of a representative diameter, however, posed a difficulty, because finding a circular pile

which has the same toe and shaft area as the square test pile is not possible. Three practical options were considered in the selection of the diameter. One option was a circular diameter of 362.8 mm that gave the same shaft area as the test pile but exceeded the actual toe area by 27.3%. A second option was a diameter of 321.6 mm that satisfied the equality of the toe area but gave a shaft area 11.4% smaller than that of the actual pile. The third option was to choose a pile diameter in between these two extreme cases. The third option was a circular diameter of 340 mm and toe area 11.7% larger than that of the test pile and a shaft area 6.4% smaller than that of the test pile.

The use of any of the above options requires adjustments in the total load, and total shaft resistance or total toe resistance. However, if these adjustments are made proportional to the ratios of the toe or shaft areas of the test pile and the analyzed pile, the potential influence of the scale effects cannot be accurately compensated for. In the present finite element analysis, the shaft resistance and toe resistance were adjusted by the following method: the finite element analyses were carried out for three, 11.0 m long piles with the mentioned three diameters (362.8 mm, 340.0 mm, and 321.6 mm). The results were plotted to show the ultimate shaft resistance as a function of the shaft area and the ultimate toe resistance as a function of the toe area.

9.5 Modelling the effect of pile installation

In the present work, a new procedure is used to determine the magnitude and distribution of residual load in a test pile. The procedure differentiates between residual loads developed in piles in sand as a result of the installation method (whether pushed-in, buried, or driven; the first two methods are used extensively in laboratory testing).

For piles pushed into sand, residual load in the pile caused by the installation is modelled by loading the pile in compression (the pile being at its final embedment depth) till failure and then unloading it back to zero load at its head. This type of residual load development is denoted Type A. The axial load remaining in the pile after complete unloading is considered to be the residual load in the pile. A typical case of pile installation in this manner is reported by Chan and Hanna (1980).

Two aspects control the distribution of Type A residual load in the pile: first, the relative magnitude between the total toe resistance and the total shaft resistance, and, second, the length and stiffness of the pile. If the pile is long and the ultimate toe resistance is larger than the ultimate shaft resistance, residual load in the pile will increase all the way to the pile toe and a significant residual toe resistance is developed. If, on the other hand, the pile is short and stiff, the magnitude of elastic rebound of the pile may be too small and to do much more than to reverse the direction of movement, that is, the mobilized shear resistance is sufficient to eliminate the existing positive shaft resistance, but the movement is insufficient to build up negative skin friction. In contrast to the long flexible pile, a short stiff pile will have only small amount of residual load.

For piles which are buried into the soil (e.g., Hanna and Tan, 1973; Tan and Hanna, 1974), the residual load (denoted Type B) in the pile is calculated by simulating an incremental build-up of soil around the pile. This type can also be simulated by letting the surrounding soil compress around the pile.

Type B residual load is associated with only a small residual toe resistance also for the long pile and, therefore, the point of force equilibrium (neutral plane) will always be some distance above the pile toe.

A bored pile resembles more the buried pile situation and Type B residual load, therefore, is representative for a bored pile. However, the residual load for a driven pile is more complicated. When the effect of the last blow has gone, at first, the residual load is similar to the Type A condition. However, starting immediately and during some time - short or long - after driving, the soil recovers from the installation disturbance and additional downward movement is built up in the soil relative to the pile adding load to the existing Type A residual load. This can be simulated by combining the Types A and B effects: the Type B simulation uses the end result of Type A as its starting point. It should be realized that this is not a superpositioning of the two types of residual load distribution.

9.6 Computed Residual Load in the Subject Test Pile

The procedure outlined above was used to compute the magnitude and distribution of the residual load in the test pile considered in this study.

Curve A in Fig. 9.3 represents the computed Type A residual load in the test pile as installed by pushing. It represents the residual load due to the release of the load at the pile head upon unloading. The numerical simulation was achieved by applying increments of downward displacement at the pile head until there was no appreciable additional resistance to further movement (soil failure), whereupon the load at the pile head was removed.

Curve B in Fig. 9.3 shows the computed Type B residual load in the test pile as installed by burying taking an unloaded pile as starting condition. The analysis proceeded in a manner similar to the simulation of the Type A residual load but with the weight of an upper 1.5 m thick soil layer excluded from the analysis. Then, to produce the Type B residual load, the weight of this upper soil layer was then added in increments combined with the pile head being free to move. The selection of 1.5 m thickness for the upper soil layer was based on a preliminary analysis in which the thickness of the soil layer was varied in order to study the effect on the magnitude and distribution of residual load.

Curve C in Fig. 9.3 shows the computed Type C residual load in the test pile as described above. The numerical analysis was carried out combining the Type A and Type B approaches. Type B being performed immediately after Type A using the Type A condition of stress and strain as starting condition.

9.7 Applied Load versus Pile Head Movement

The finite element computations of the applied load versus pile head movement for all four tests are shown in Fig. 9.4. The figure also shows the measured values. The computations for the third compression loading was stopped at a displacement of about 35 mm rather than continuing to 120 mm as recorded in the actual test. After

35 mm displacement of the pile head, the pile was unloaded and the analysis of the tension test started.

The applied load versus pile head movement data from the first compression test were used in the selection of the unloading-reloading modulus. The agreement between the computed and measured values of applied load versus pile head movement in the first compression test is, therefore, imposed. However, as shown in Fig. 9.4, a very good agreement exists between the computed values and the actual measurements throughout the remaining three loading tests.

The finite element computations for the applied load versus pile head movement in the tension test gave a slightly stiffer response compared to the actual measurements. This difference is attributed, in part, to the use of same modulus of elasticity for the pile material in tension and compression. Since the actual modulus of elasticity in tension is slightly lower than that in compression, the elastic elongation of the pile was underestimated in the finite element calculations of the tension loading test.

The maximum load measured in the tension test was 580 kN (including the weight of the pile), whereas, the finite element calculations indicated a maximum load of about 600 kN (including the weight of the pile), overestimating the tension failure load. However, at the time of the tension test, the groundwater table had risen from 6.4 m to 5.0 m. This resulted in the reduction of effective stresses which, in turn, reduced the shaft resistance. In difference to the treatment in Chapter 8, the effect of the changes in groundwater table was not accounted for in the present calculations.

9.8 Effect of Residual Load on Applied Load versus Pile Head Movement

Residual load in a pile affects its load-movement response. This is demonstrated in Fig. 9.5 for the present test pile. The figure compares the computed applied load versus pile head movement in the first compression test for the test pile with initial residual load (same as the shown in Fig. 9.4) and the applied load versus pile head movement of the test pile without residual load prior to the loading test. Both analyses were carried to loads of 1,000 kN. The figure illustrates that a pile-soil system with no

residual load requires larger movements to mobilize the same load as for a pile-soil system with residual load. This difference is due to that the residual load preloads the pile toe.

The results shown in Fig. 9.5 are in agreement with the laboratory observations made by Hanna and Tan (1973) on small-scale instrumented piles tested in sand, the results of the analytical studies by Poulos (1987), and the results of the analysis of some case histories reported by Vesic (1977). Both the experimental and analytical results reported in the literature show that a stiffer response is to be expected (i.e., smaller head movement is required to mobilize the same resistance) when piles with residual load are tested in compression as opposed to piles with no residual load. This observation can be used to evaluate the consistency of any numerical analysis of axially loaded piles. Since the presence of residual load affect the applied load versus pile head movement response of a pile, no agreement should be expected between the results of the numerical analysis and the actual measurements if the residual load is not included in the analysis. In fact, if a good agreement is achieved without considering the residual load, this indicates that an error is made in the input of soil stiffness and/or that non-representative initial horizontal stresses in the soil are assumed in the analysis.

9.9 Load Transfer

In the present tests, the gages were zeroed at the start of each loading test. Consequently, the loads measured in the pile do not represent the true loads. In the numerical analysis, the measured load distributions were calculated by subtracting the computed residual load from the load distributed along the pile (explained further in Chapter 8). Computed distribution curves were compared with those obtained from the field measurements for the three compression loading tests.

The load values measured in the pile during the first compression test are labeled T1 in Fig. 9.6. The values computed from the finite element analyses for the same applied load at the pile head are shown in the figure as Curve F1 and include residual load effects. The computed residual load (Type C) in the pile prior to the first loading test is labeled R0. This residual load is the result of the pile installation. Curve TP1 is the

computed "measured load distribution" obtained by subtracting the computed residual load (Curve R0) from the load shown by Curve F1. The agreement between the finite element calculations, Curve TP1, and the test results, Curve T1, is excellent.

Similar agreements are obtained in the analyses of the second and third compression tests. The results of these comparisons are provided in Figs. 9.7 and 9.8. As shown in these figures, the residual loads used in the analysis of all tests are different from each other. This is due to that the residual load at the pile toe increases by each loading event as shown in Fig. 9.9.

9.10 Effects of Repetitive Loading on Load Transfer

Each test showed an increase in the load carried by the pile toe and reduction in the load carried by the pile shaft as shown in Fig. 9.10. The figure compares the load transfer in the three compression tests for 1,000 kN load at the pile head. The curves labeled F1, F2, and F3 correspond to the first, second, and third compression test, respectively.

The computed toe resistances for each of the three compression tests were 360 kN, 400 kN, and 430 kN and the shaft resistances were 640 kN, 600 kN, and 570 kN, respectively. The reduction in the shaft resistance is further illustrated by plotting the shear stresses on vertical planes in the soil elements adjacent to the pile shaft as shown in Fig. 9.11. The shear stress along the pile shaft decreased with each loading event. This reduction in the shear stress was higher in the vicinity of the pile toe compared to the reductions at other locations.

Gradual degradation of shaft resistance has been observed before. For example, Chan and Hanna (1980), Boulon et al. (1980), Nauroy et al. (1985), Puech and Jezequel (1980), and Turner and Kulhawy (1989). These authors attributed the observed degradation of the shaft resistance to a potential progressive decrease in the horizontal (lateral) stress in the soil. Indeed, the present study indicates that the horizontal stress decreases from test to test as shown in Fig. 9.12. Fig. 9.12 shows the variation of the horizontal stress in the soil elements adjacent to the pile shaft at a 1,000 kN load on the

pile head for all three compression tests. The variation in horizontal stress is similar to that of shear stresses shown previously in Fig. 9.11.

9.11 Tension Loading Test

No information regarding load transfer are available for the tension loading test for the reasons discussed in Chapter 8. However, load-movement data were available as presented and it was compared with the numerical calculations in Fig. 9.6.

The numerical results of the tension test were also used to compare the calculated ultimate shaft resistance of the test pile in tension and compression.

Fig. 9.13 gives the ultimate shear stresses on the vertical planes passing through the soil elements adjacent to the pile shaft. Curve F4 is related to the tension test and Curve F3 is related to the third compression test. The shear stress values are similar along the whole pile length and this suggests that, for all practical purposes, the ultimate shaft resistance in compression is equal to that in tension for this test pile.

The computed load transfer in the tension test obtained from the finite element analysis is shown in Fig. 9.14. Before the tension test started, the pile was under compression due to the residual load that remained after the third compression test. The distribution of this residual load is shown as Curve R3. This distribution implies that, before the tension test, the shear strength of the soil along the upper portion of the pile is already mobilized due to the presence of the residual load. Furthermore, the pile has a significant residual load at its toe.

With increasing tension load at the pile head, the soil along the lower portion of the pile starts to mobilize more and more shaft resistance, whereas unloading of the residual load at the toe continues. At the maximum tension load, the distribution of the axial load in the pile is shown as Curve F4.

With unloading, the load transfer is changing at each loading level. At the complete unloading, the distribution of the axial load in the pile is shown in Fig. 9.14 as

Curve R4. This curve represents the load remaining in the pile after the tension test. Note that the residual toe load diminishes to zero at the end of the test. It is of interest to note that a significant amount of residual tension load is produced as the result of the tension test. Such residual load at the end of the tension test was assumed not to exist in the Hunter and Davisson (1969) method of interpretation of loading test data from instrumented piles. Neglecting the tension residual load affects the shape of the distribution of the unit shaft resistance in compression by the Hunter and Davisson method.

9.12 Final Remarks

The results of the finite element analysis agree very well with the measured data and, also, with the results of the conventional method of analysis presented in Chapter 8. Throughout the testing programme, this agreement is noticeable in the applied load versus pile head movement response as well as in the load transfer along the pile shaft. No such agreement can be achieved if the pile is assumed stress-free before the start of static testing.

Using the residual load computed as an integral part of the analysis eliminates the need for the Critical Depth concept in order to explain the measured data.

The test results and the results of the finite element analysis demonstrate the importance of loading history on the development of residual load.

A gradual increase in the residual toe load develops with each repeated compression loading. This increase is due to the accumulation of the downward toe movement at the end of each load repetition.

The calculated values of the ultimate total shaft resistance in tension and compression loading are equal. In addition, there is no difference in the distribution of the unit shaft resistance during the compression and tension loading.

The ultimate shaft resistance degrades with each repetition of compression loading. Reduction in horizontal stresses is observed from the numerical computation which supports the hypothesis that the observed degradation of shaft resistance is due to a reduction in horizontal stresses. The degradation of the shaft resistance is accompanied by an increase in the degree of mobilization of the toe resistance.

Significant amount of residual tension load is calculated in the pile at the end of unloading from the tension loading.

CHAPTER TEN

CONCLUSIONS

10.1 Conclusions

The conclusions of this study are divided into three groups. These include,

1. Implementation of Constitutive Model
2. Validation of Constitutive Model
3. Application to Deep Foundation

Each of these groups is considered separately as in the following.

10.1.1 Implementation of Constitutive Model

Numerical problems are observed due to extreme softening of soil (very low values of bulk modulus). This is due to the use of a linear relation between the bulk modulus and the mean pressure for all levels of mean pressure. A linear relation leads to extremely low values for the bulk modulus for low values of the mean pressure. To overcome this difficulty, modification is made in the expression determining the bulk modulus. This is achieved by using one of three alternatives.

In integrating the constitutive equations numerically, it is observed that the size of strain increments is not the only factor controlling the accuracy of the integration over a solution step. States in the vicinity of the hydrostatic state of stress and those approaching the region of the critical state requires much smaller strain increments to

achieve numerical integration with acceptable accuracy. This observation has been made earlier in dealing with other constitutive models in which the critical state was the bound (Herrmann et al., 1986). A special selective multistep integration technique is employed to overcome the difficulties of performing accurate and efficient numerical integration.

The classical return procedures are used successfully to bring back predicted stress states lying outside the bounding surface.

No numerical difficulties are observed in simulating the generation of pore water pressure throughout the analysis. A combined bulk modulus for the soil particles and pore water is introduced in the formulation of the model and values as high as 100 GPa are used successfully for ideal undrained situations.

The correctness of the numerical implementation of the model into the finite element program is verified with respect to four cases. These include monotonic drained, monotonic undrained, cyclic drained, and cyclic undrained cases. Finite element results coincide with results obtained by integrating the constitutive equations numerically in all cases considered.

10.1.2 Validation of Constitutive Model

A two-stage validation analyses are carried out to demonstrate the capabilities of the constitutive model in simulating soil behavior in simple stress analysis and boundary value problems.

In the first stage of validation, the behavior of Sacramento River sand, Fuji River sand, a crushed quartz sand, and a sandy silt is considered in a number of conventional and non-conventional stress-path and strain-path tests. Comparisons between the model simulations and the measured responses of these soils show a very good performance of the model. The calculations, using the model, reproduce several important aspects of soil behavior observed in laboratory testing.

In the second stage of validation, the analyses of a model-scale footing, the behavior of Leighton Buzzard sand in the Cambridge Simple Shear Device, and the behavior of a six metre wide foundation are considered. The observed load-displacement relation and the localized nature of soil deformation of the model-scale footing compare very well with the results of the finite element analyses. The analysis of the behavior of the Leighton Buzzard sand confirmed the experimental observation that the difference between the measurements on the sample core and those normally made in a routine test along the full top boundary is less than 5%. The analyses of the six meter footing provide useful comparative results when the foundation soil was clean loose sand, oil contaminated loose sand, and locally oil contaminated loose sand.

From this study, it is concluded that the bounding surface plasticity model describes the behavior of cohesionless soils quite well, both in laboratory experiments and in boundary value problems.

10.1.3 Application to Deep Foundation

The procedures recommended by Fellenius (1989b) for using an effective stress analysis to determine the distribution of residual load in the 11.0 m long test pile is successfully applied to the test data proving that the pile was subjected to residual load before the start of static testing.

The results of the finite element analysis agree very well with the measured data, and also with the results of the Fellenius (1989b) method of analysis. Throughout the testing programme, this agreement is noticeable in the applied load versus pile head movement response as well as the load transfer along the pile shaft. No such agreement can be achieved if the pile is assumed stress-free before the start of static testing.

A new procedure is developed for the calculation of residual stresses and strains in a pile-soil system resulting from pile installation. Using the residual load computed as an integral part of the analysis (using the procedures developed herein) eliminates the need for the Critical Depth concept in order to explain the measured data.

The test results and the results of the finite element analysis demonstrate the importance of loading history on the development of residual load and the axial load transfer.

A gradual increase in the residual toe load develops with each repeated compression loading. This increase is due to the accumulation of the downward toe movement at the end of each load repetition.

The calculated values of the ultimate total shaft resistance in tension and compression loading are equal. In addition, there is no difference in the distribution of the unit shaft resistance during the compression and tension loading.

The ultimate shaft resistance degrades with each repetition of compression loading. Reduction in horizontal stresses is observed from the numerical computation which supports the hypothesis that the observed degradation of shaft resistance is due to a reduction in horizontal stresses. The degradation of the shaft resistance is accompanied by an increase in the mobilized toe resistance.

Significant amount of residual tension load is calculated in the pile at the end of unloading from the tension loading. Such residual load at the end of the tension test was assumed not to exist in some of the methods of interpretation of loading test data from instrumented piles.

10.2 Recommendations for Further Research

At present, research is underway in the following aspects which are related to this study:

1. Further verification of the numerical analysis method developed in this study with respect to the behavior of piles under axial loads. Preliminary results indicate the validity of the procedures to other case histories.
2. Study of scale effects of pile testing. Preliminary results indicate that substantial scale effects exist in most laboratory pile sizes reported in the literature.

3. Numerical analysis of piezocone and self boring pressuremeter in order to determine soil parameters from measurements made during these in-situ tests. Preliminary results indicate the feasibility of this objective.
4. Develop a simple constitutive soil model for cohesive soils in order to facilitate the study of piles in cohesive soils, as well as piles in soils consisting of both cohesive and cohesionless soils. Preliminary results with regard to the soil model is encouraging.
5. Further develop the finite element program to study pile groups and generalized loading conditions such as the case of combined lateral and axial loads.

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Table 4.1 Model parameters
used in sensitivity analysis

κ	=	0.01
Γ	=	0.90
λ	=	0.08
M_c	=	1.42
M_e	=	1.02
M_p	=	1.58
ν	=	0.20
ρ	=	2.20
h_o	=	1.00

Table 4.2 Summary of qualitative effect
of model parameters

An increase in	$q = \sigma_1 - \sigma_3$	ε_v
Γ	$\uparrow^{(1)}$	$\downarrow^{(2)}$
ρ	$\downarrow$	$\uparrow$
M_c or ϕ_c	$\uparrow$	$\uparrow$
M_e or ϕ_e	$\uparrow$	$\uparrow$
M_p or ϕ_p	$\uparrow$	$\downarrow$
λ or C_c	$\downarrow$	$\uparrow$
h_o	$\uparrow$	$\downarrow$
κ or C_r	$\uparrow$	$\downarrow$
ν	$\downarrow$	$\downarrow$

(1) $\uparrow$ means an increase is caused

(2) $\downarrow$ means a decrease is caused

Table 5.1 Model parameters for
Sacramento River sand

		Dense	Loose
κ	=	0.0075	0.01
Γ	=	0.84	0.97
λ	=	0.076	0.076
ϕ_c	=	36.2°	34.6°
ϕ_e	=	36.2°	34.6°
ϕ_p	=	43.8°	35.7°
ν	=	0.20	0.20
ρ	=	1.65	2.20
h_o	=	1.70	1.70

Table 5.2 Model parameters
for Fuji River sand

κ	=	0.01
Γ	=	0.92
λ	=	0.12
ϕ_c	=	36.9°
ϕ_e	=	32.0°
ϕ_p	=	38.0°
ν	=	0.30
ρ	=	2.20
h_o	=	0.70

Table 5.3 Model parameters for
crushed quartz sand

		Clean	Contaminated
κ	=	0.012	0.012
Γ	=	1.091	1.070
λ	=	0.036	0.036
M_c	=	1.28	1.03
M_e	=	0.90	0.72
M_p	=	1.30	1.05
ν	=	0.30	0.30
ρ	=	2.60	2.60
h_o	=	2.00	0.50

Table 5.4 Model parameters
for a sandy silt

κ	=	0.001
Γ	=	0.472
λ	=	0.120
ϕ_c	=	35°
ϕ_e	=	35°
ϕ_p	=	35°
ν	=	0.35
ρ	=	1.90
h_o	=	2.00

Table 6.1 Model parameters
for Leighton Buzzard sand

κ	=	0.005
Γ	=	0.927
λ	=	0.025
ϕ_c	=	35°
ϕ_e	=	35°
ϕ_p	=	35°
ν	=	0.37
ρ	=	2.20
h_o	=	1.00

Table 7.1 Summary of earth pressure coefficient, K_s , against pile shafts
(Modified after Kulhawy, 1984)

Source	K_s	Method	Notes
Brinch-Hansen, 1951	$\cos^2\phi$	theoretical	
Meyerhof, 1951	0.5	field data	loose sand
Meyerhof, 1951	1.0	field data	dense sand
Henry, 1956	K_p	theoretical	
Ireland, 1957	1.7-3.0	field data	pull tests
Mansur and Kaufmann, 1958	0.3	field data	
Mansur and Kaufmann, 1958	0.6	field data	pull tests
Kezdi, 1958	K_p	theoretical	
Broms, 1965	1.0-2.0		concrete piles
Lamb and Whitman, 1969	2.0	assumed	medium dense
McClelland, 1969	0.7	assumed	

Table 7.2 Load transfer measurements
(Modified after Hunter and Davisson, 1969)

Test Pile No.	Type of Pile	Total Load tons	Toe Resistance			Shaft Resistance			Ten- sion Loads tons
			Meas- ured tons	Adju- sted tons	Change %	Meas- ured tons	Adju- sted tons	Change %	
1	12.75 in OD	172	48	85	+77	124	87	-30	92
2	16 in OD	251	75	120	+60	176	131	-26	116
3	20 in OD	258	112	160	+43	146	98	-33	120
7	14BP73	220	65	90	+23	155	130	-16	75
10 ^(a)	16 in OD	228	83	80	-4	135	138	+2	110
16 ^(b)	16 in OD	165	50	90	+80	115	75	-35	73

^a Driven with Bodine resonant driver

^b Jetted to 40 ft

Table 7.3 Pile loading test information
(after Briaud and Tucker, 1984)

Site (1)	Pile (2)	Symbol (3)	Pile Type and Material (4)	Dimensions			Modulus of Elasticity psi = 10 ⁶ (8)	Type Test (9)	Strain Rods (10)	Strain Gages (11)	Soil Data				Other (15)	Reference (16)	
				Dia- meter ft (5)	Length ft (6)	Area, in. ² (7)					SPT (12)	CPT (13)	FW (14)				
Lock & Dam 4, Arkansas River (1963)	1	1	Steel Pipe	1.20	53.1	17.12	29.0	C,T	X							4, 9	
	2	2	Steel Pipe	1.50	52.0	23.86	29.0	C,T	X	X							
	3	3	Steel Pipe	1.70	53.0	27.36	29.0	C,T	X								
	7	4	Steel "WP" HP 14 x 73	1.34	52.1	29.33	29.0	C,T	X	X	X						
	10	5	Steel Pipe	1.50	53.1	23.86	29.0	C,T	X	X							
	14	6	Steel Pipe	1.42	52.7	29.42	29.0	C,T	X	X							
Low Sill Structure, Old River, La. (1954)	2	7	Steel Pipe	1.75	45.1	27.36	29.0	C,T	X							10	
	4	8	Steel Pipe	1.42	44.3	22.63	29.0	C,T	X		X						
	5	9	Steel Pipe	1.42	45.1	22.63	29.0	C,T	X								
	6	10	Steel Pipe	1.58	45.2	25.00	29.0	C,T	X								
Oppechee River (1969)	H-15	11	Steel Pipe	1.50	49.3	27.49	30.0	C			X	X	X		Donelline	14, 15	
Lock & Dam 26 Replacement Site	JIP- 1115	12	Steel "WP" HP 14 x 73	1.34	54.0	21.48	29.0	C,T			X	X				3	
Cragoan (1969)	A	13	Circular Prestressed Concrete	0.92	26.2	95.45	3.15	C			X					5	
	D/A	14	Circular Pres. Conc.	0.92	52.5	95.45	3.15	C			X						
	C	15	Tapered Circular Prestressed Concrete	Top 0.92 Bot. 0.66	26.2	Varied	3.15	C			X	X	X				
	B/C	16	Straight Cir. to 26.3 ft Tapered to bottom Pres. Conc.	Top 0.92 Bot. 0.66	52.5	Varied	3.15	C			X						
Seligman (1981)	A-1	17	Square Pres. Conc.	1.00	35.4	113.00	3.15	C			X				X	11	
	A-11	18	Square Pres. Conc.	1.00	35.4	113.00	3.15	C			X						

Notes: 1 ft = 0.305 m; 1 psi = 6.89 kN/m².

Notes: 1 ft = 0.305 m; 1 psi = 6.89 kN/m²

Table 8.1 Dates of testing and groundwater observations

Date	Depth to G.W. (m)	Event
June 17, 1984	--	Driving of the piles
June 20, 1984	--	Installation of piezometer
Aug. 22, 1984	6.20	Compression Test I
Aug. 29, 1984	--	Compression Test II
Sep. 13, 1984	6.36	Compression Test III
Oct. 28, 1984	6.70	
Feb. 5, 1985	5.02	

Table 8.2 Values resulting from the calculations

Layer	Depth	Dry	Total	β	N_t
--	(m)	Density (kg/m ³)	Density (kg/m ³)	--	--
Silt-Sand	0.0 - 3.0	1,600	1,600	0.40	--
Dry Sand	3.0 - 5.0	1,800	1,800	0.50	--
Moist Sand	5.0 - 6.5	1,800	1,900	0.65	--
Sat. Sand	6.5 - 11.0	1,800	2,000	0.65	30

Table 8.3 Gross and net pile head movements

Test	Per Test		Accumulated	
	Gross (mm)	Net (mm)	Gross (mm)	Net (mm)
Compression Test I	10.5	6.0	10.5	6.0
Compression Test II	14.8	10.7	20.8	16.7
Compression Test III	52.7	---	63.4	---

Table 9.1 Model parameters used in the analysis

Parameter	Soil Layer			
	-- I	dry II	moist III	sat. IV
Effective angle of friction				
compression	32	34	34	36
extension	32	34	34	36
peak	32	34	34	36
Poisson's ratio		0.3		
Aspect ratio		2.0		
Hardening parameter		1.0		
Critical void ratio at 100 kPa		0.95		
Slope of critical state line in e-ln(p) plane		0.05		
Unloading-reloading modulus		0.01		

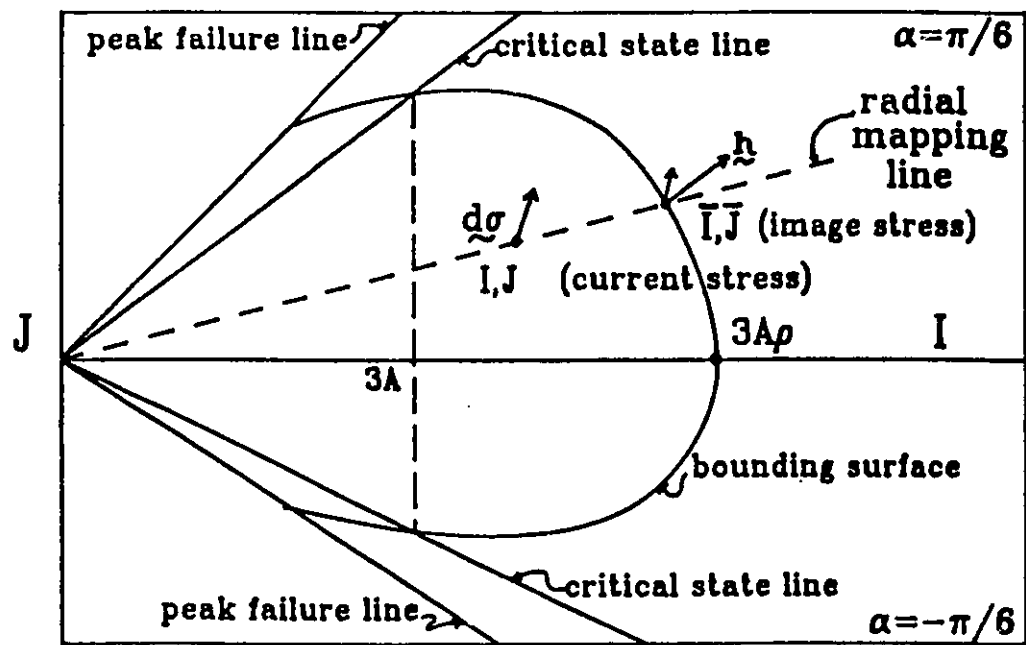


Fig. 2.1 Bounding surface, radial mapping, and image stress
in I-J invariant space
(Modified after Bardet, 1986)

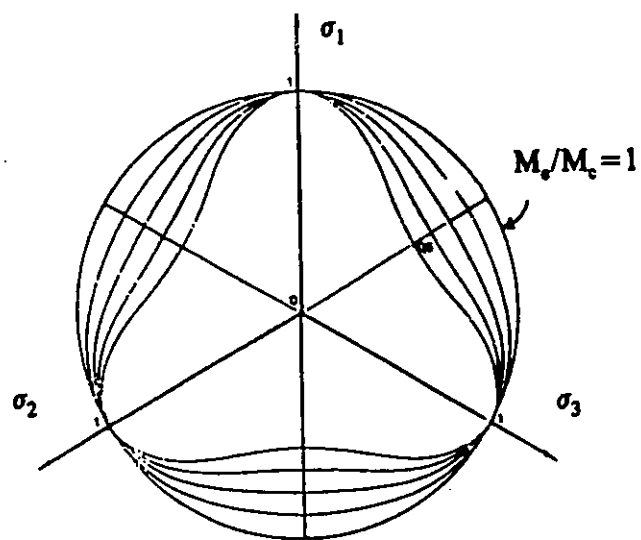


Fig. 2.2 Cross section of bounding surface in deviatoric plane
(after Bardet, 1986)

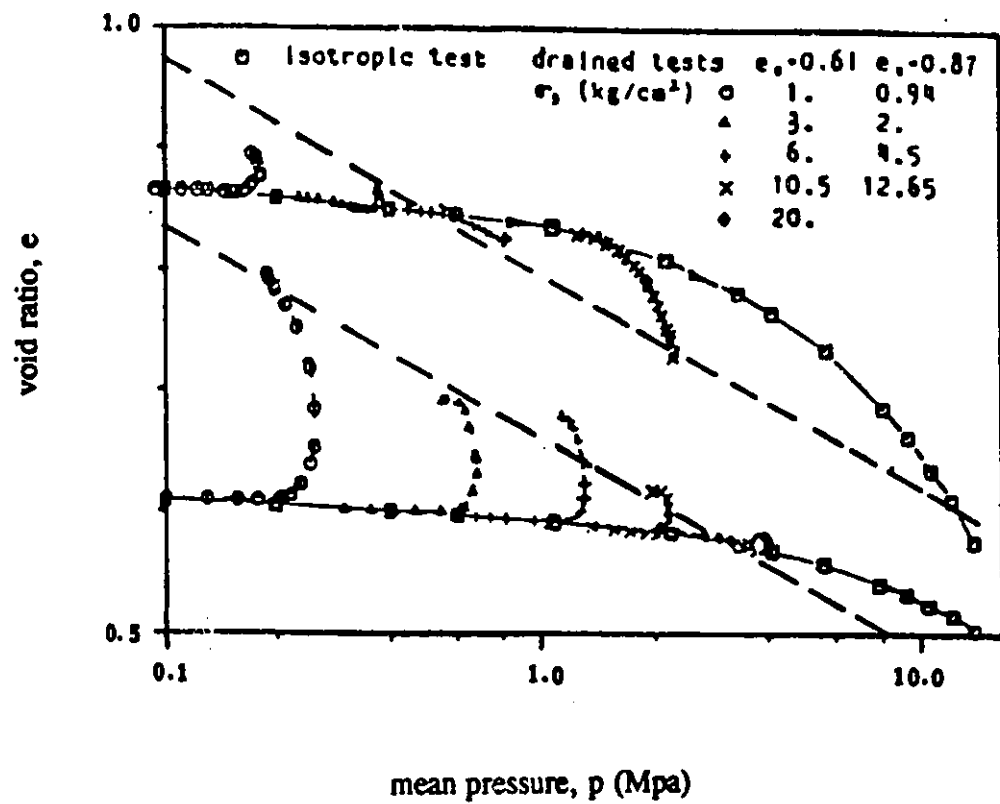


Fig. 2.3 Experimental results of drained tests at various confining pressure on loose and dense Sacramento River sand (after Bardet, 1986)

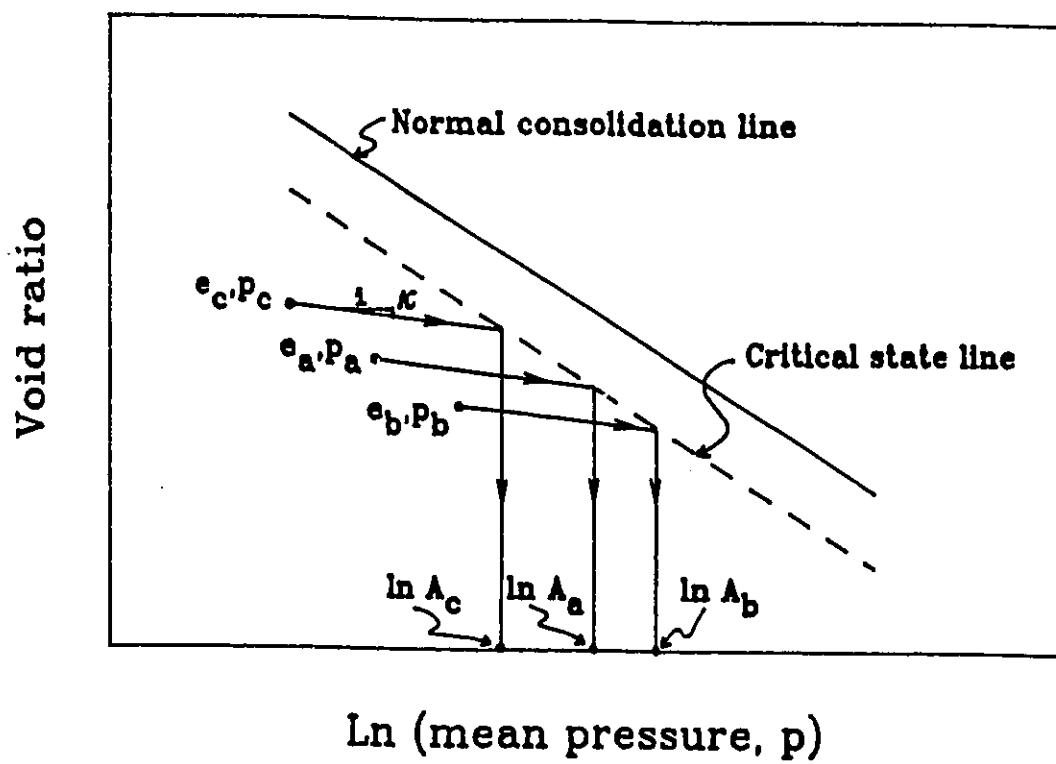


Fig. 2.4 Position of the bounding surface summit along the mean pressure axis

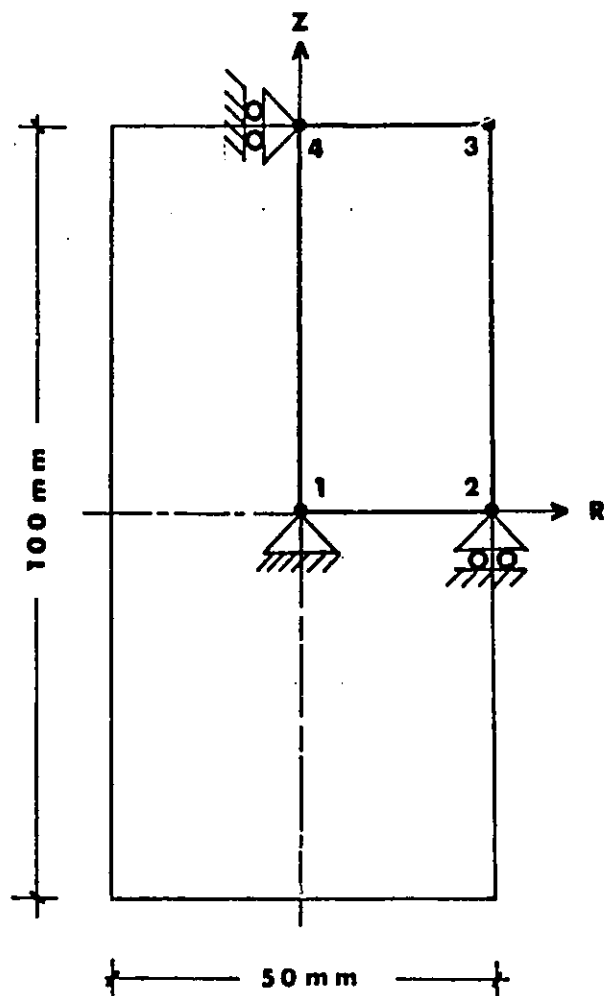


Fig. 3.1 One-element mesh for cylindrical specimen

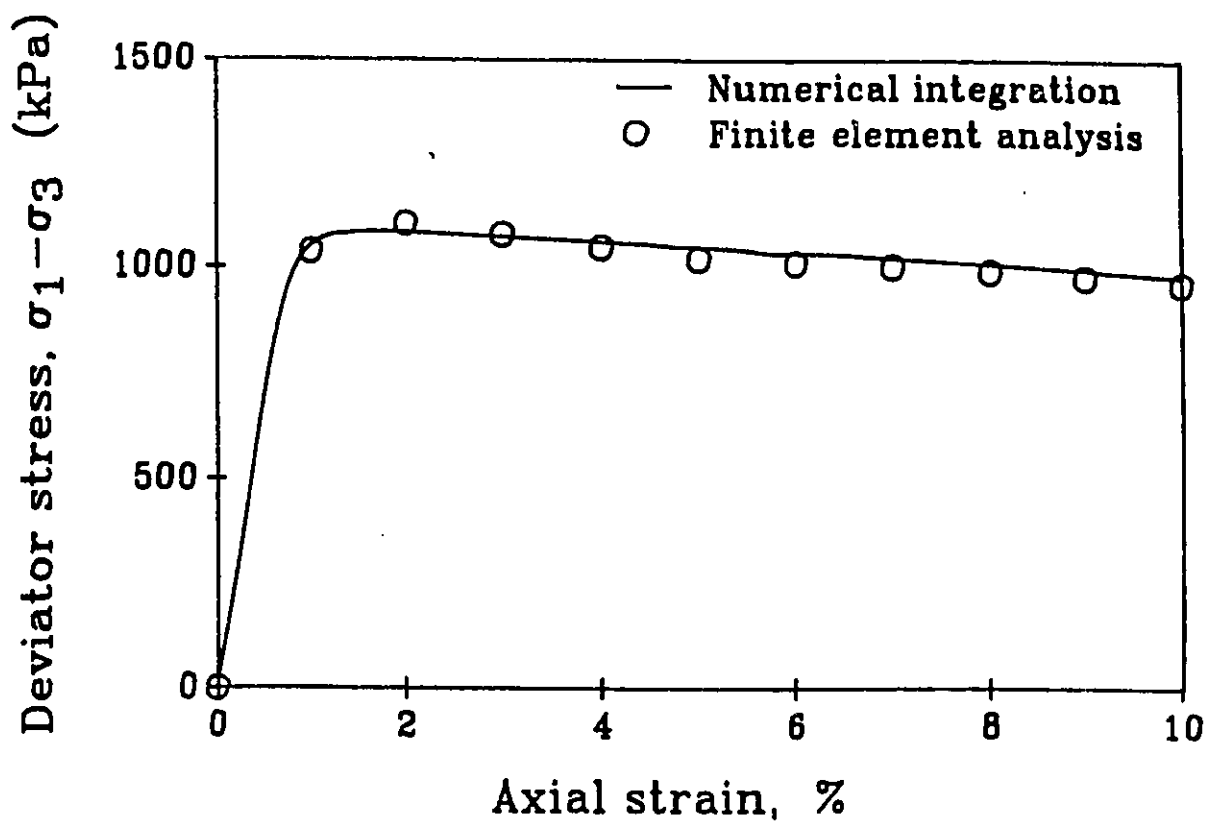


Fig. 3.2 Comparison of finite element and numerical integration results of deviator stress vs axial strain for dense Sacramento River sand

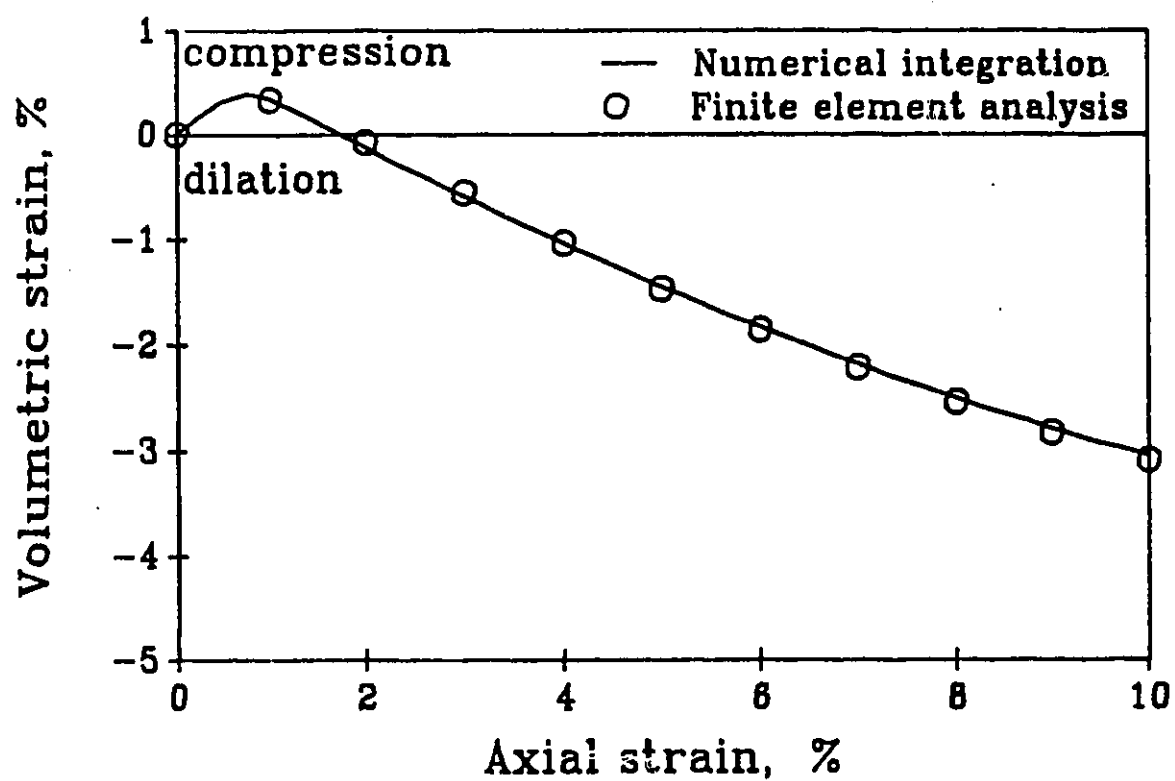


Fig. 3.3 Comparison of finite element and numerical integration results of volumetric strain vs axial strain for dense Sacramento River sand

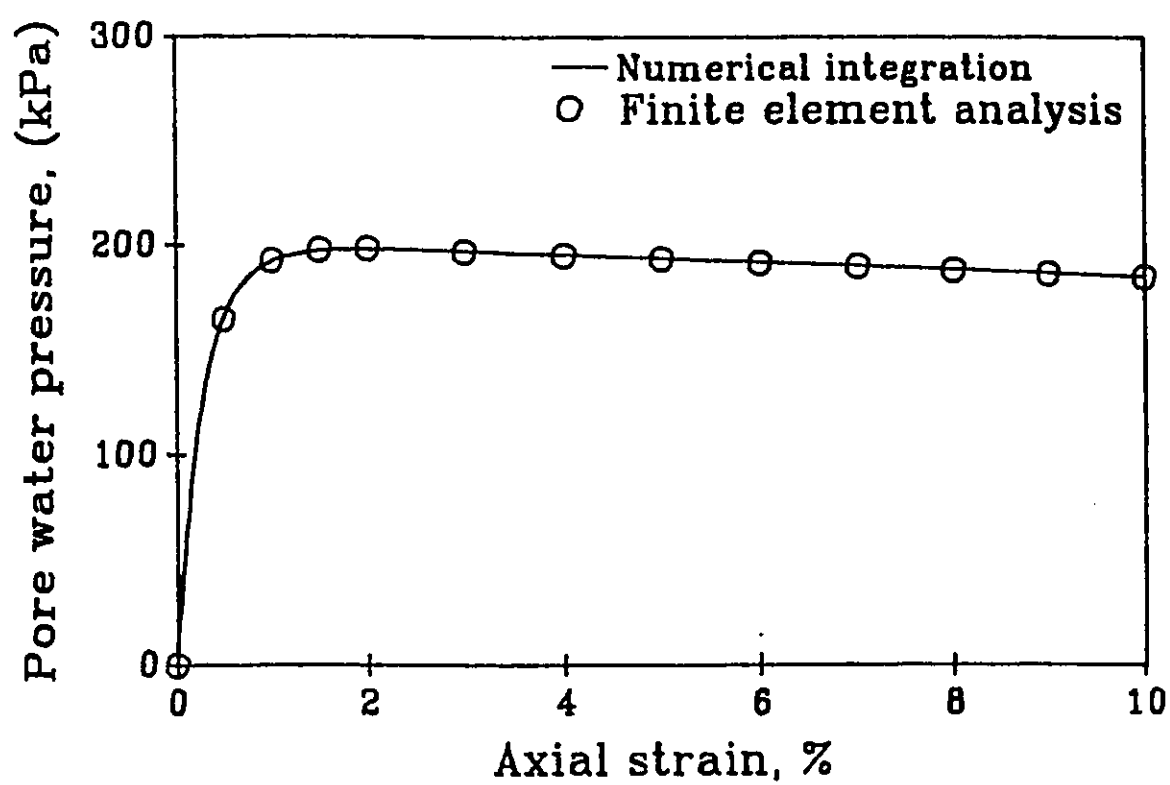


Fig. 3.4 Comparison of finite element and numerical integration results of pore water pressure vs axial strain for loose Sacramento River sand

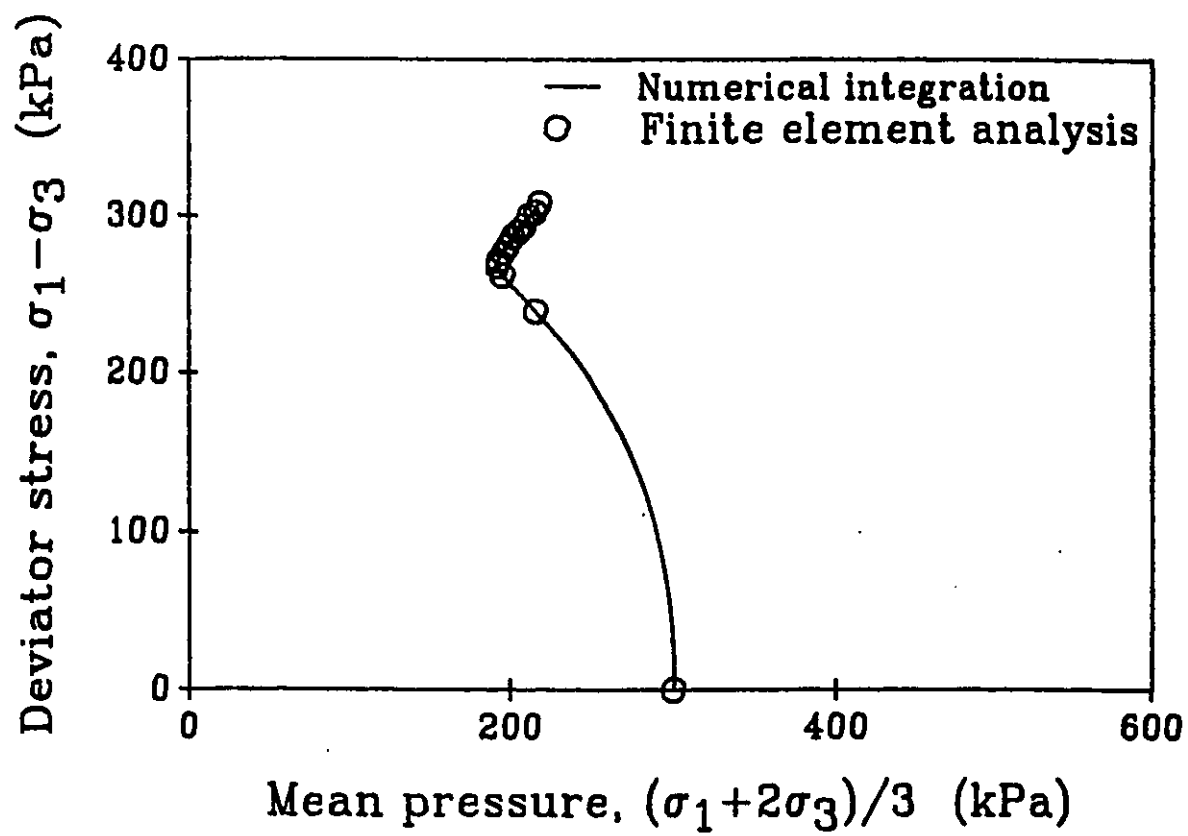


Fig. 3.5 Comparison of finite element and numerical integration results of deviator stress vs mean pressure for loose Sacramento River sand

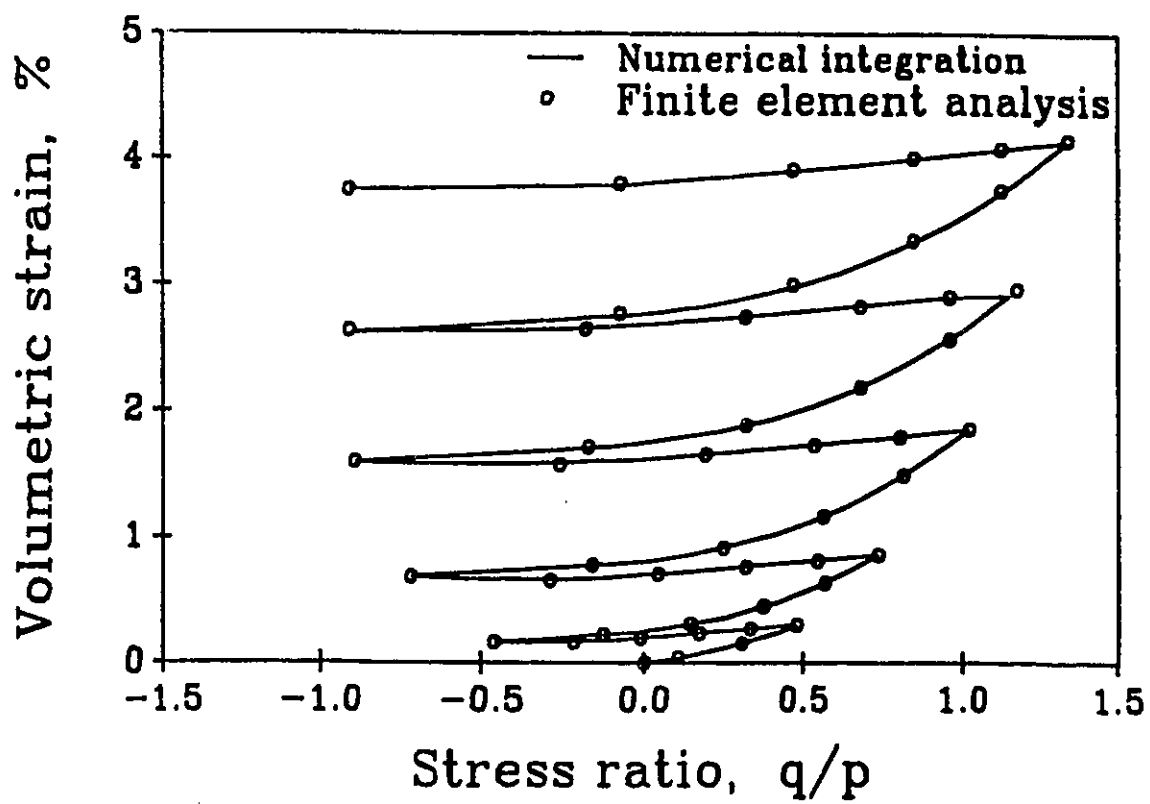


Fig. 3.6 Comparison of finite element and numerical integration results of volumetric strain vs stress ratio for Fuji River sand

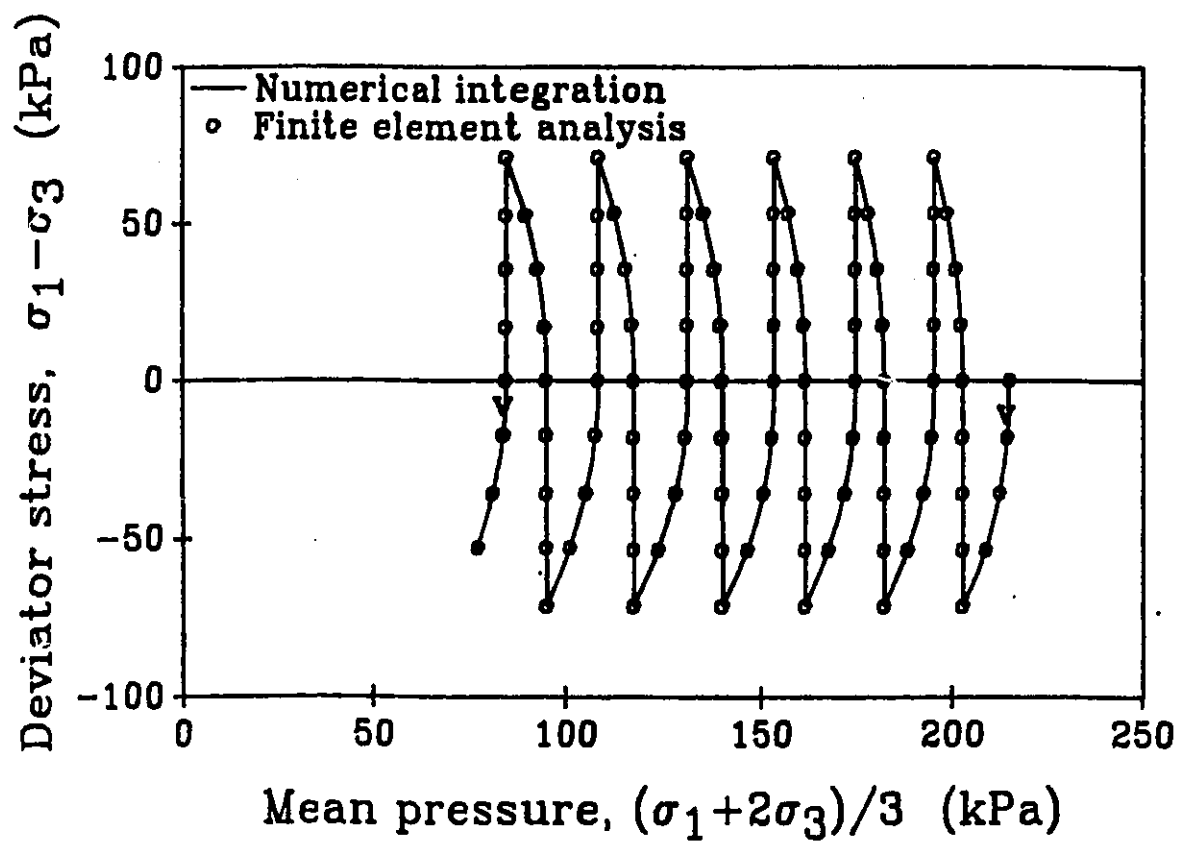


Fig. 3.7 Comparison of finite element and numerical integration results of deviator stress vs mean pressure for Fuji River sand

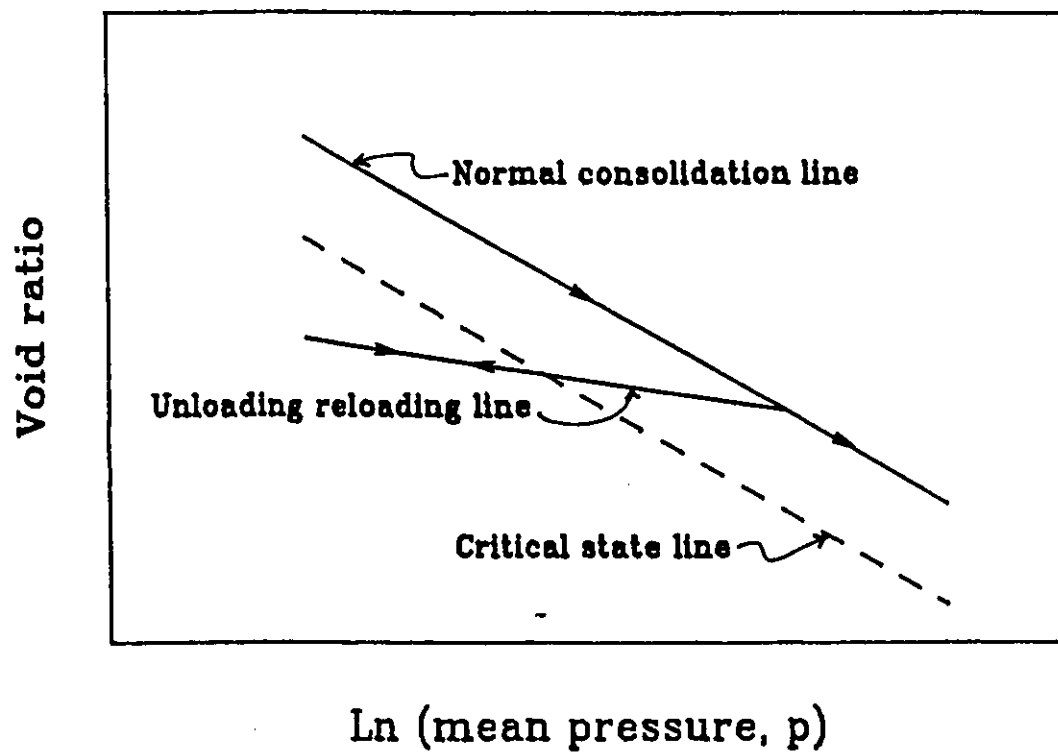


Fig. 4.1 Critical state, normal consolidation, and unloading reloading lines in e - $\ln(p)$ plane

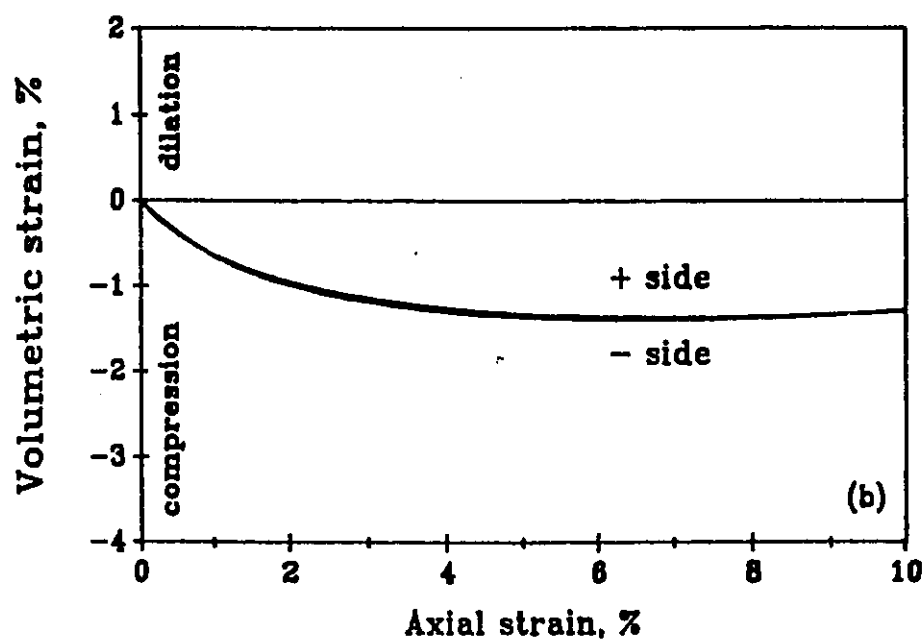
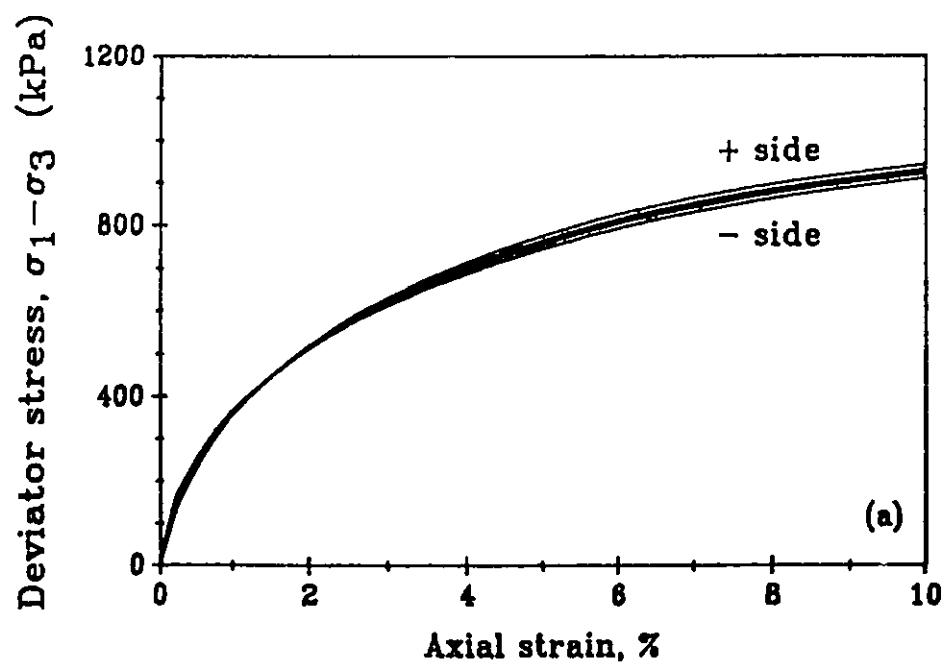


Fig. 4.2 Effect of unloading reloading modulus, κ

(a) deviator stress versus axial strain

(b) volumetric strain versus axial strain

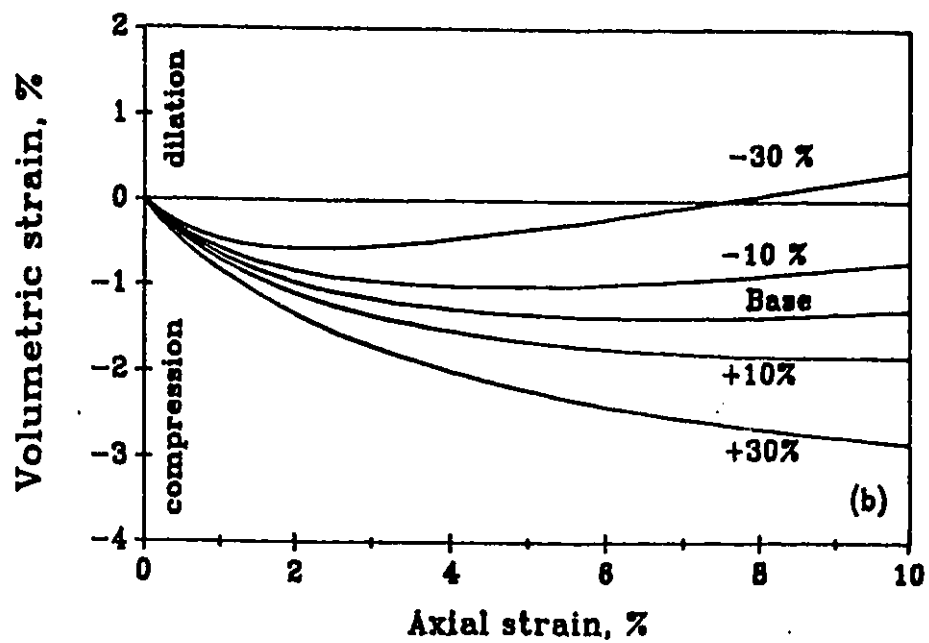
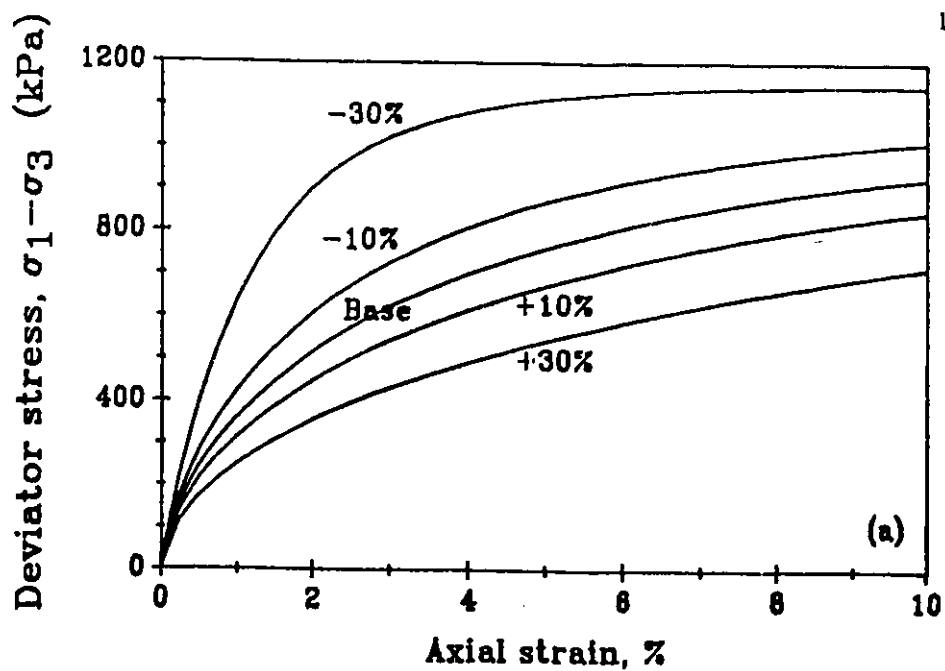


Fig. 4.3 Effect of slope of critical state line in $e-\ln(p)$ plane, λ

(a) deviator stress versus axial strain

(b) volumetric strain versus axial strain

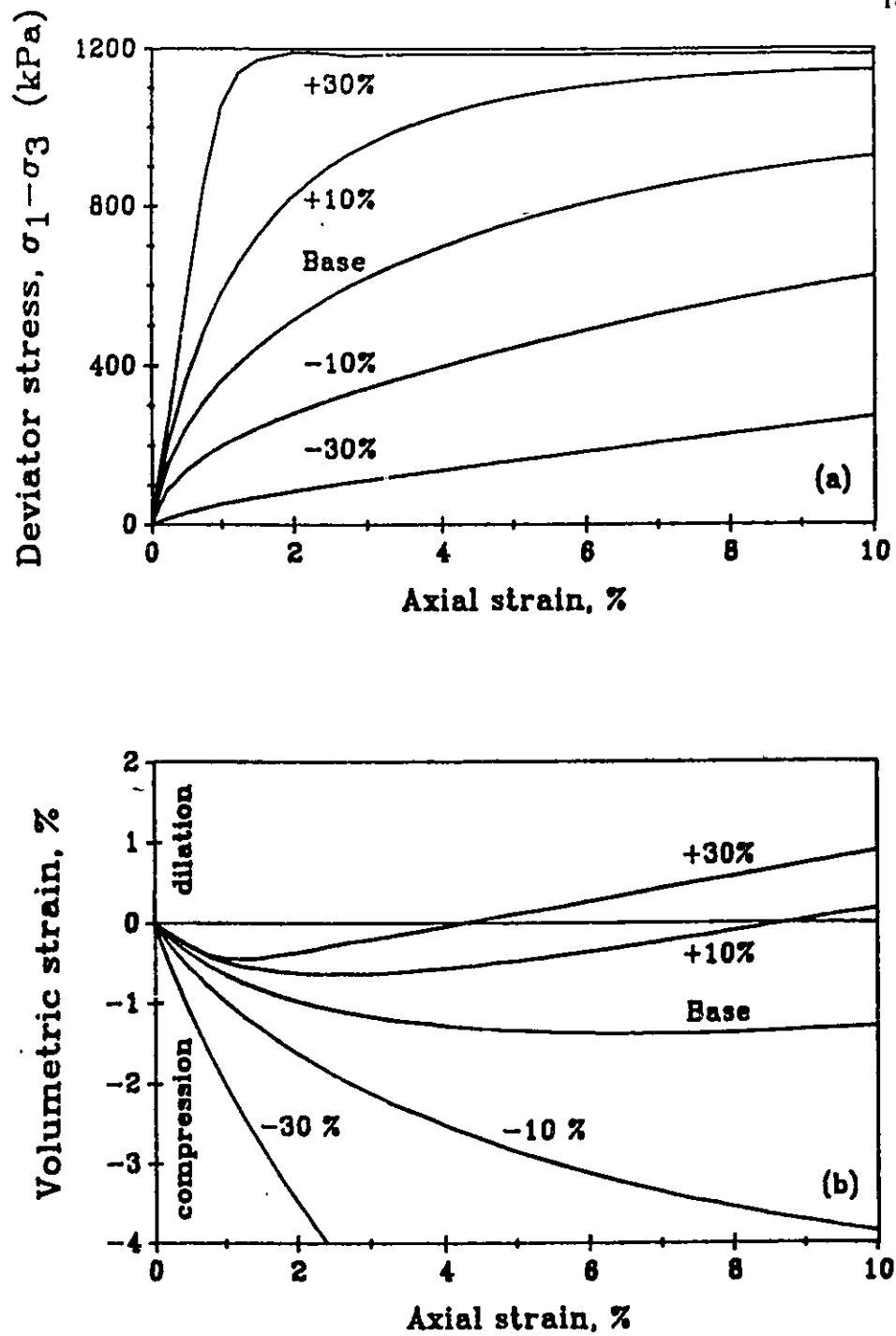


Fig. 4.4 Effect of critical void ratio at unit mean effective pressure, Γ
 (a) deviator stress versus axial strain
 (b) volumetric strain versus axial strain

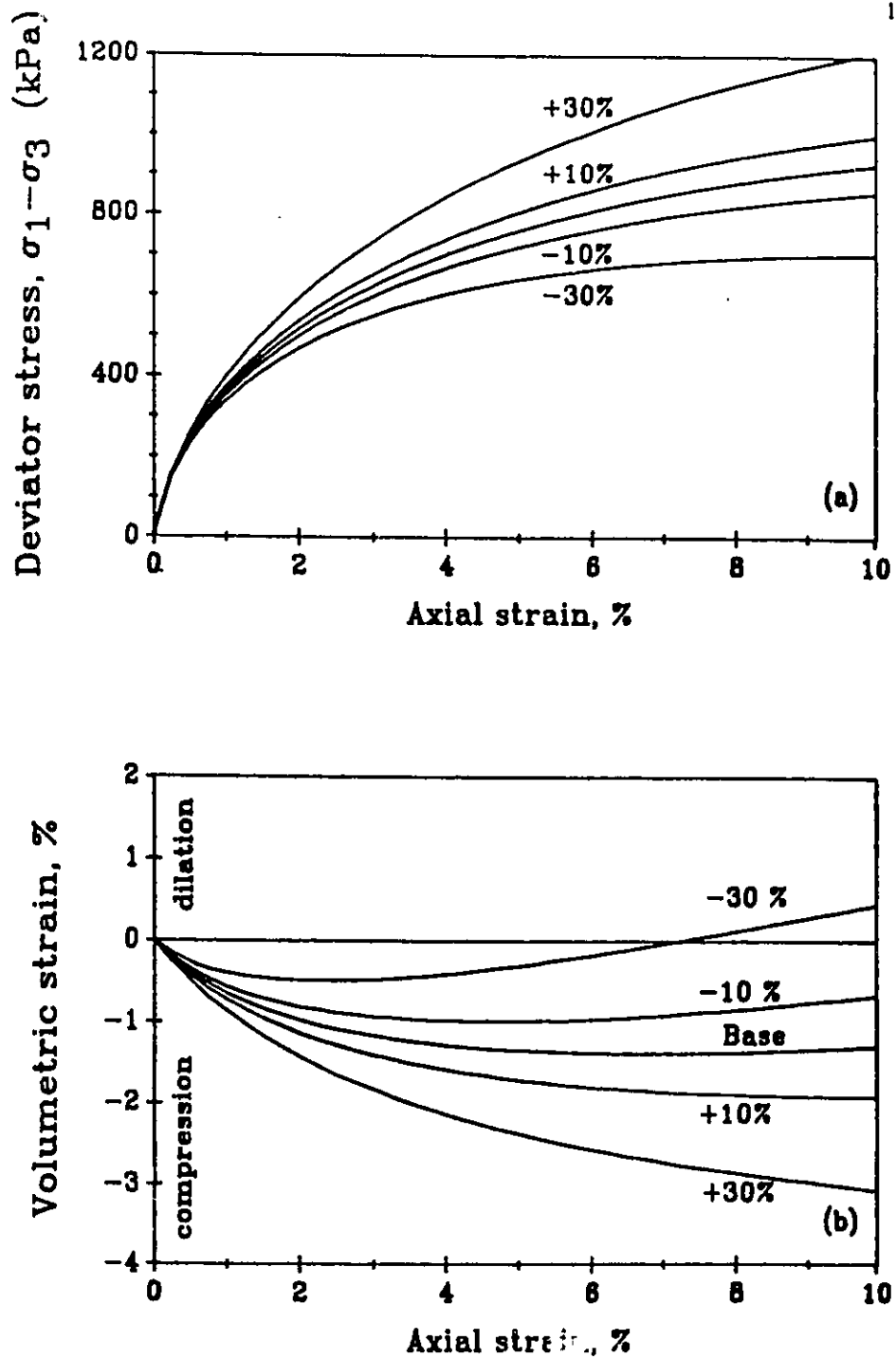


Fig. 4.5 Effect of ultimate friction angle in compression, ϕ_c

(a) deviator stress versus axial strain

(b) volumetric strain versus axial strain

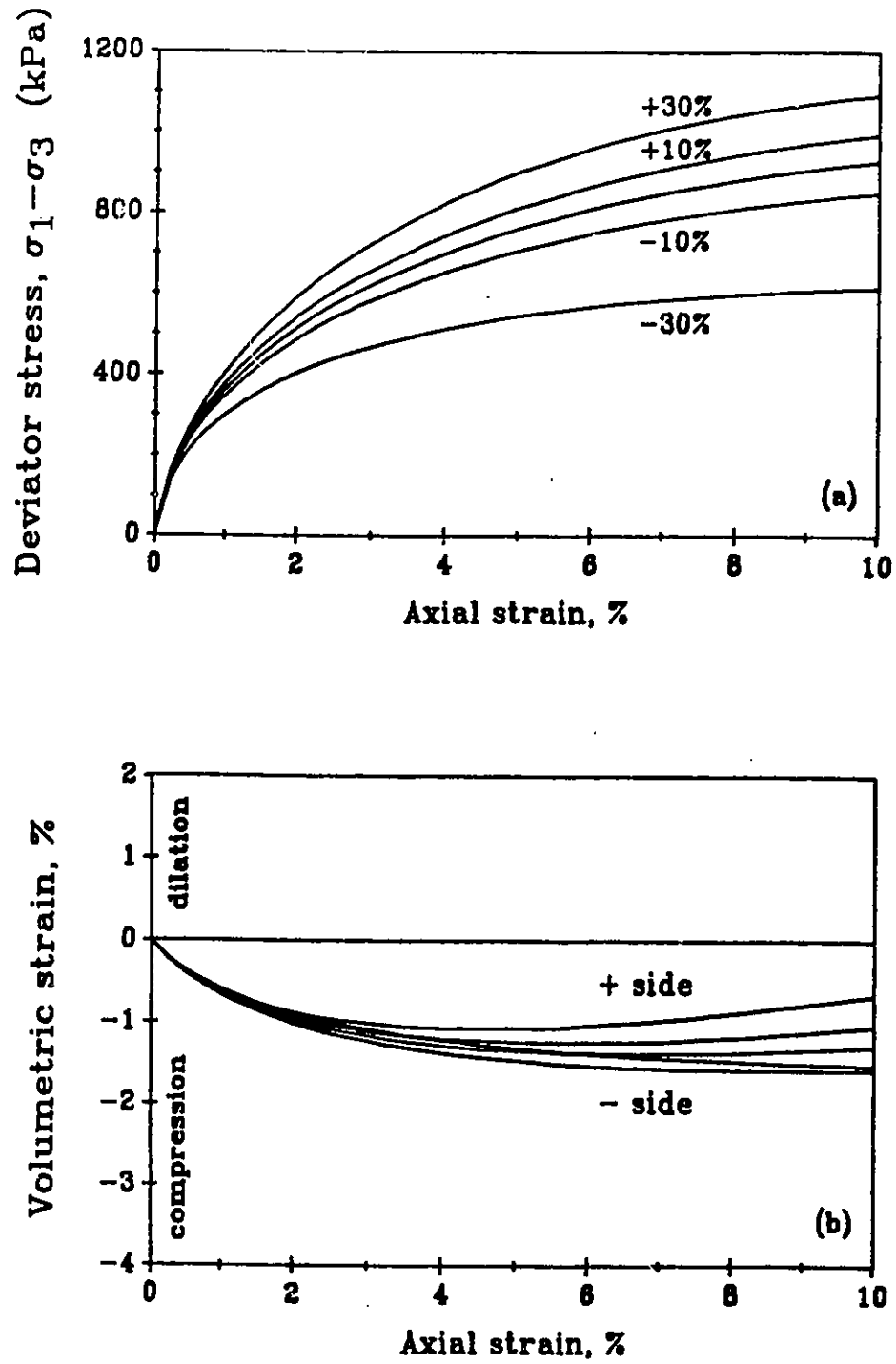


Fig. 4.6 Effect of peak failure friction angle, ϕ_p

(a) deviator stress versus axial strain

(b) volumetric strain versus axial strain

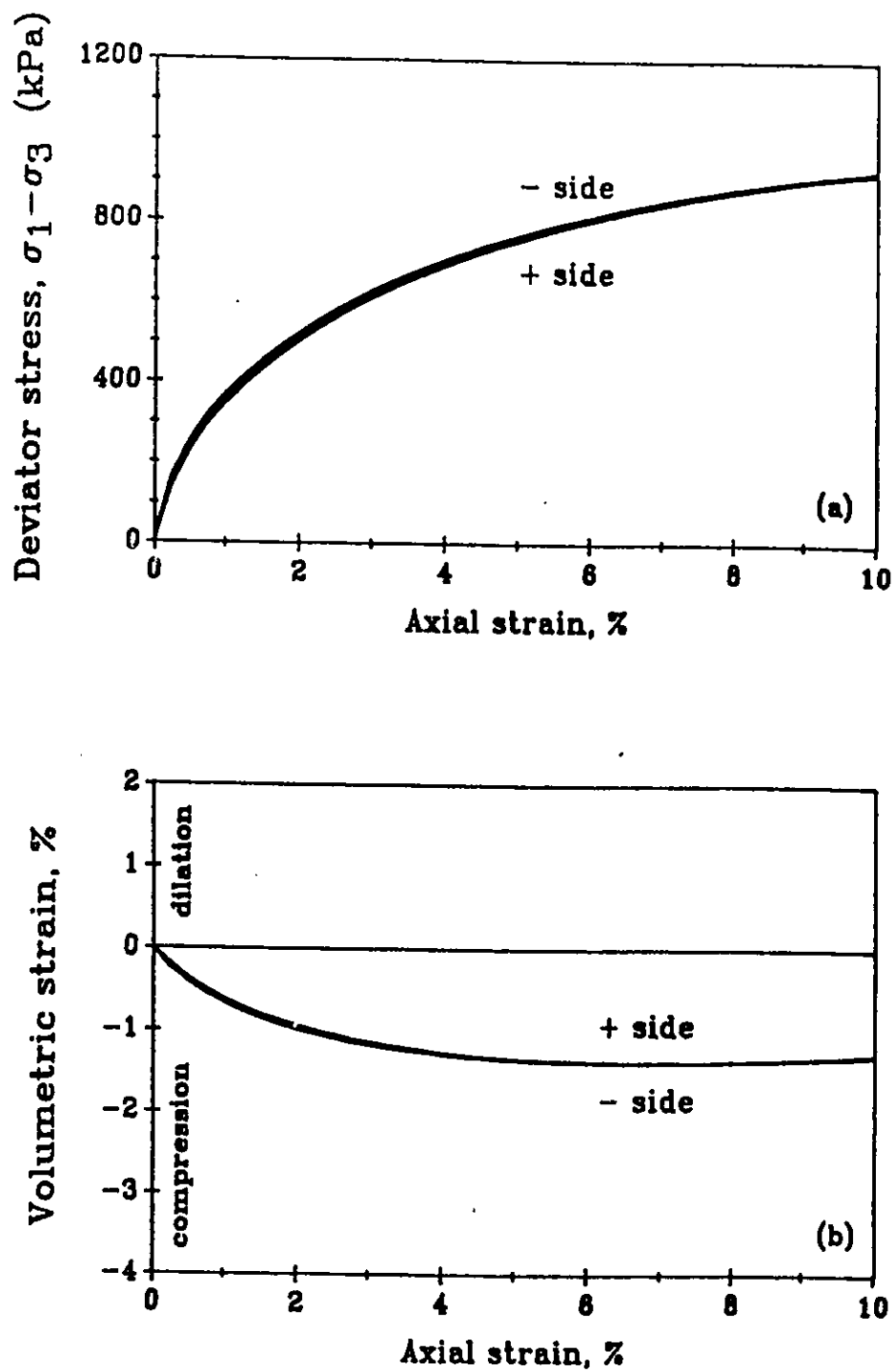


Fig. 4.7 Effect of Poisson's ratio, ν

(a) deviator stress versus axial strain

(b) volumetric strain versus axial strain

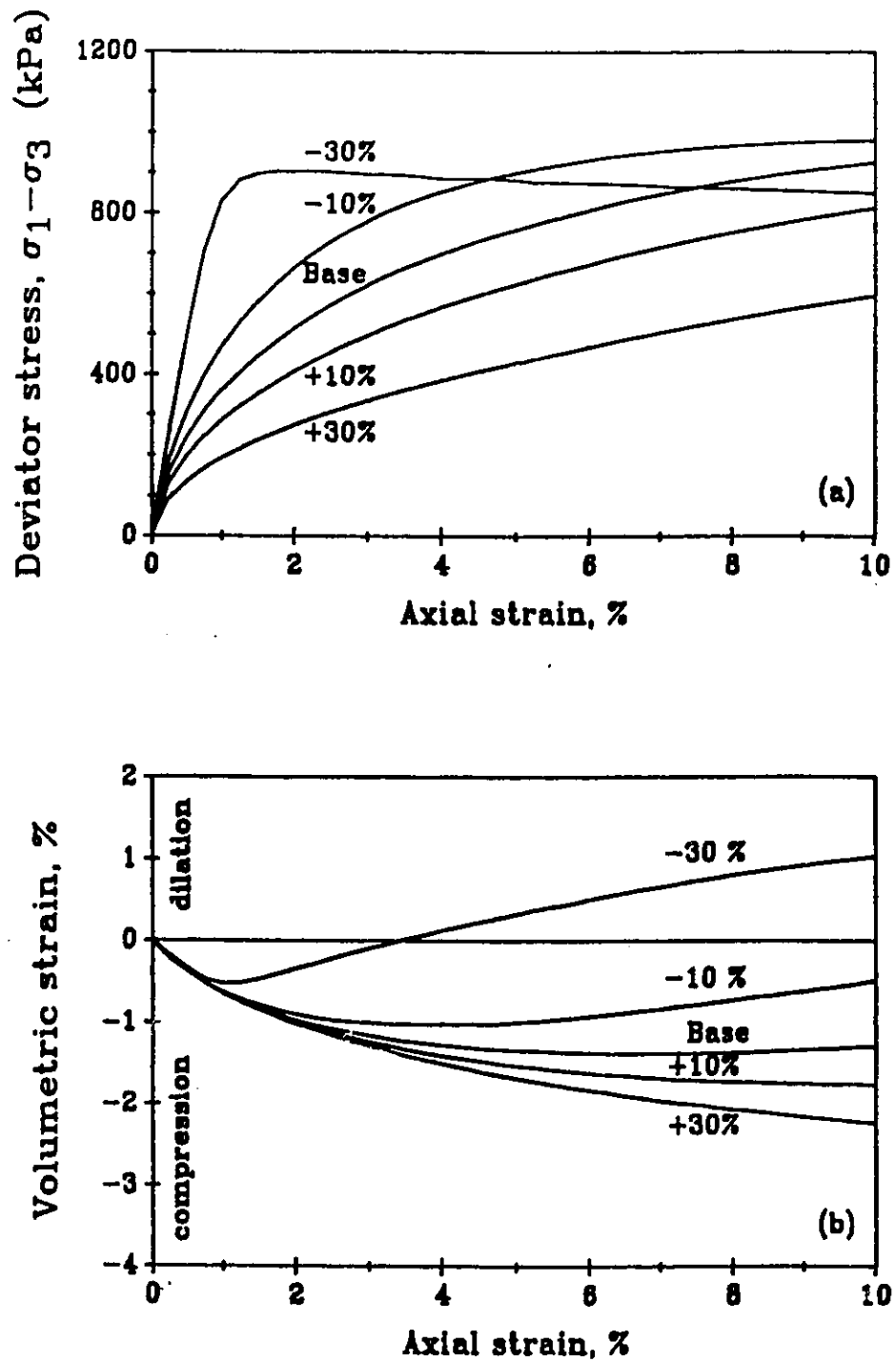


Fig. 4.8 Effect of bounding surface aspect ratio, ρ

(a) deviator stress versus axial strain

(b) volumetric strain versus axial strain

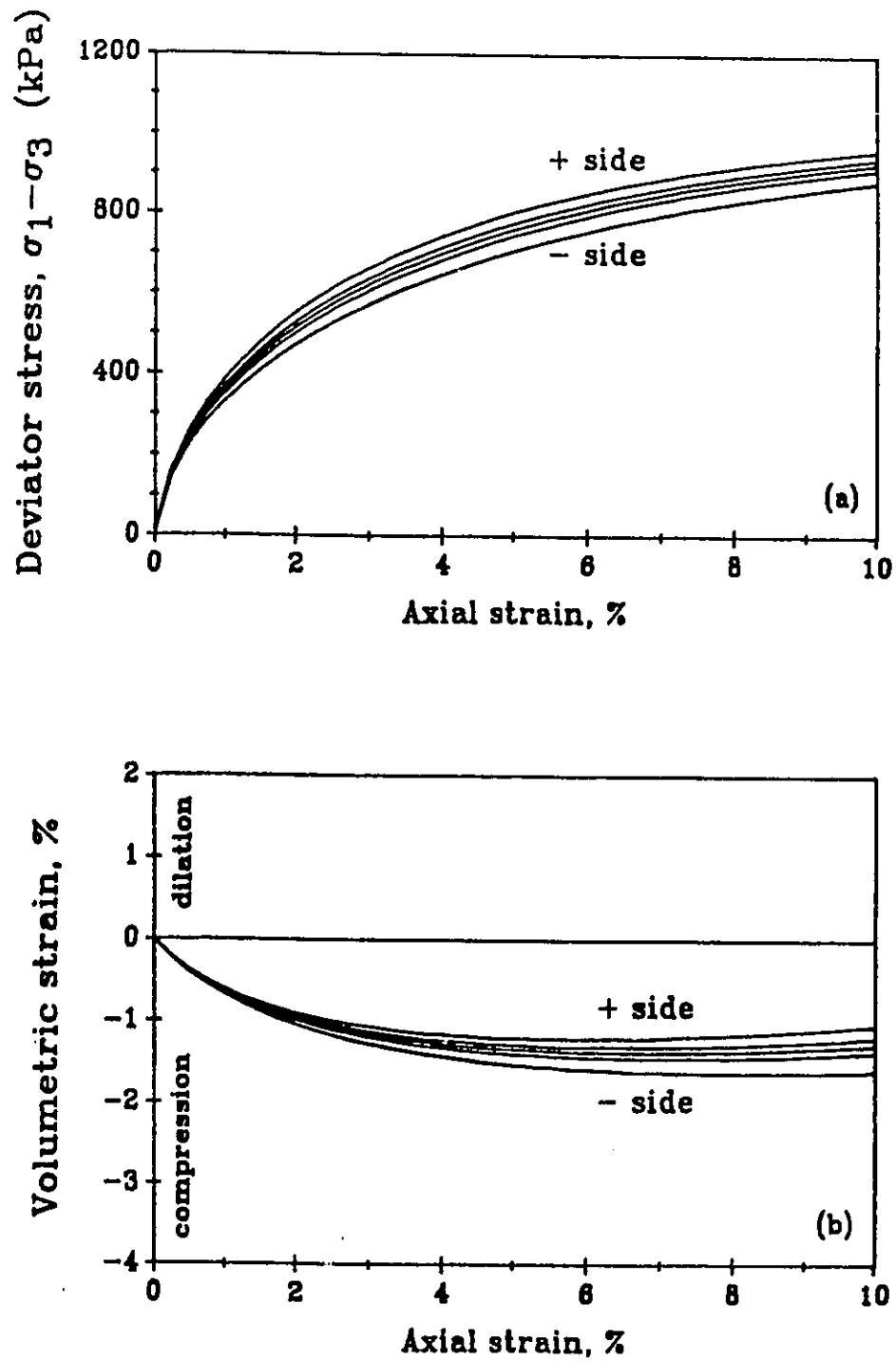


Fig. 4.9 Effect of plastic modulus parameter, h_0 .

(a) deviator stress versus axial strain

(b) volumetric strain versus axial strain

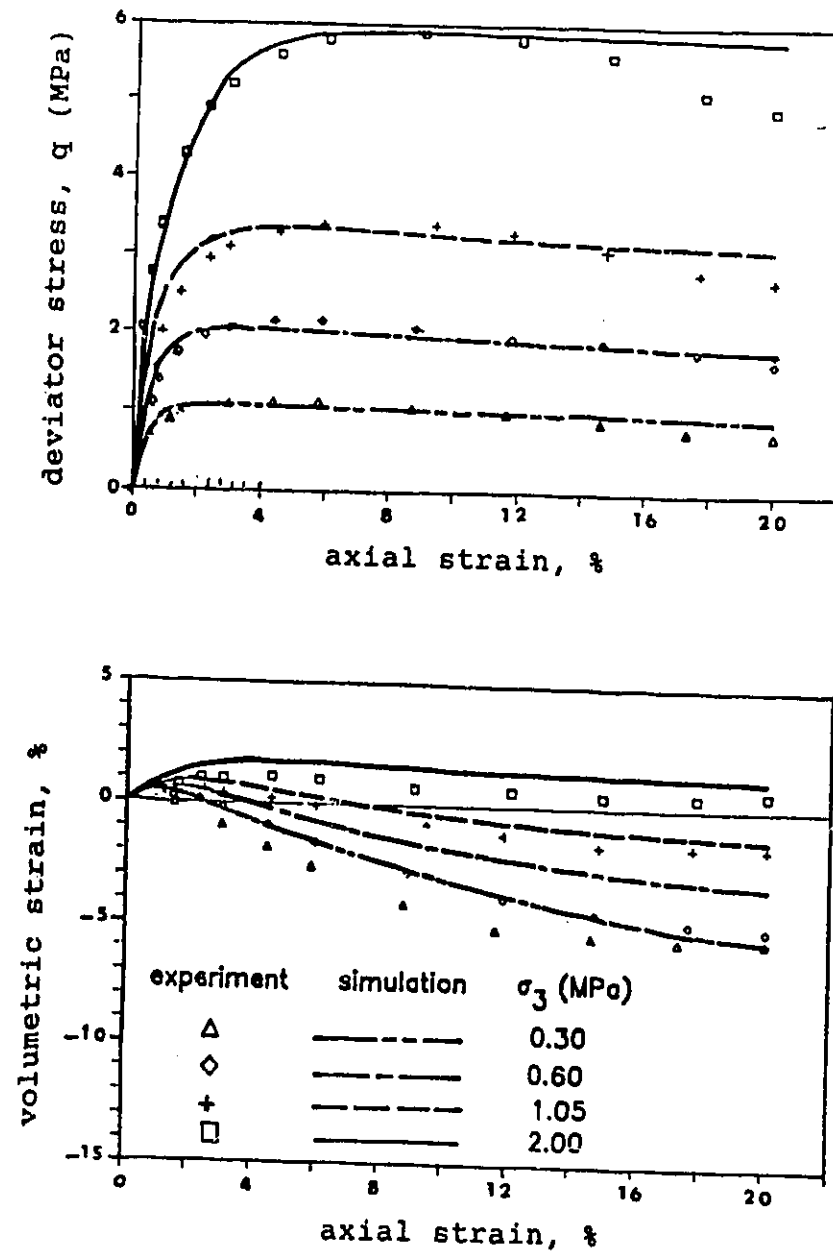


Fig. 5.1 Model simulation and experimental results of drained tests at different confining pressures on dense Sacramento River sand
(Experimental results are from Lee and Seed, 1967)

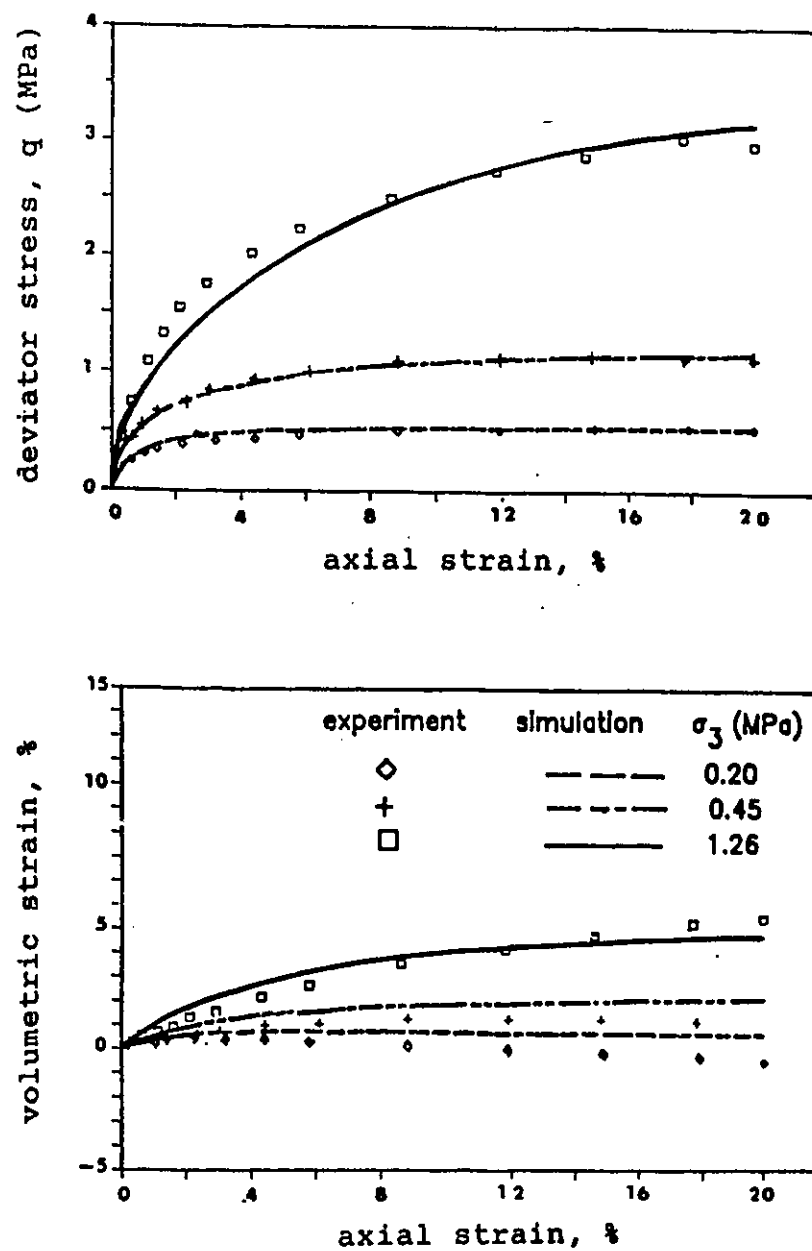


Fig. 5.2 Model simulation and experimental results of drained tests at different confining pressures on loose Sacramento River sand
(Experimental results are from Lee and Seed, 1967)

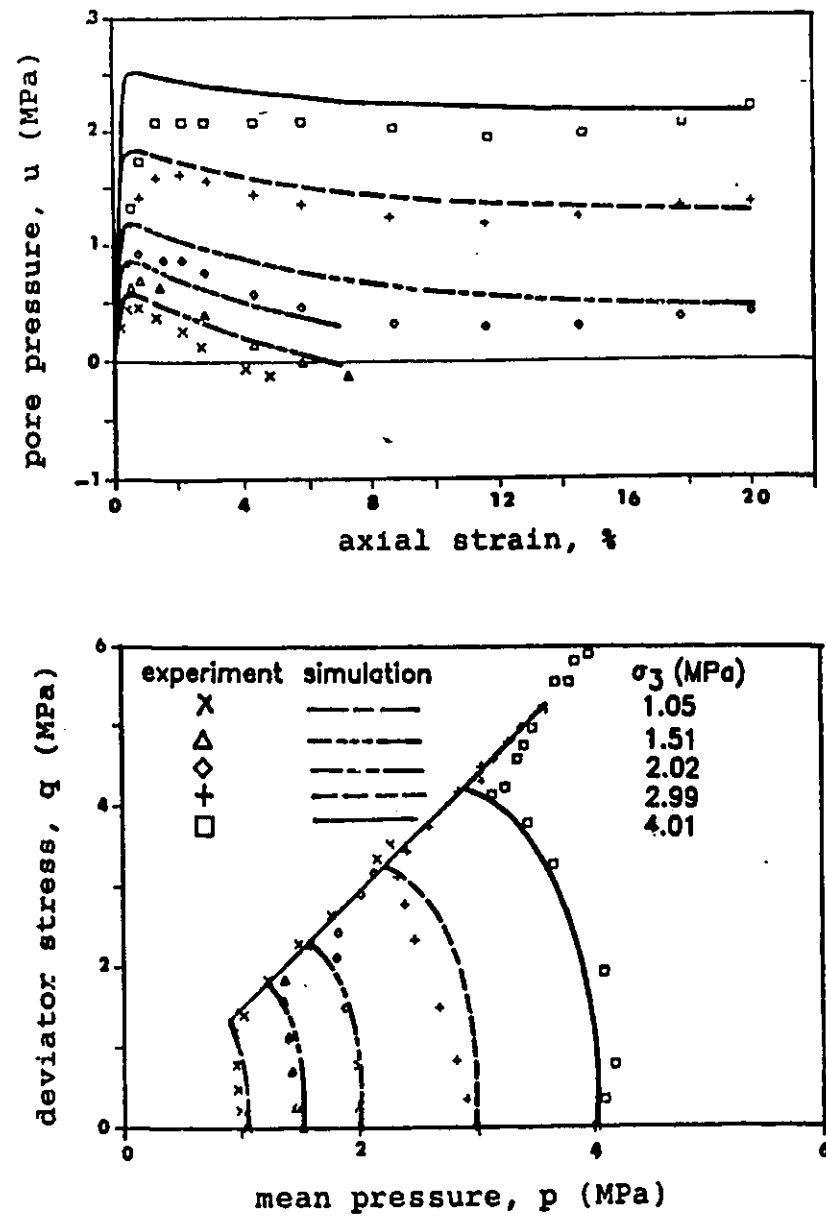


Fig. 5.3 Model simulation and experimental results of undrained tests at different confining pressures on dense Sacramento River sand
(Experimental results are from Seed and Lee, 1967)

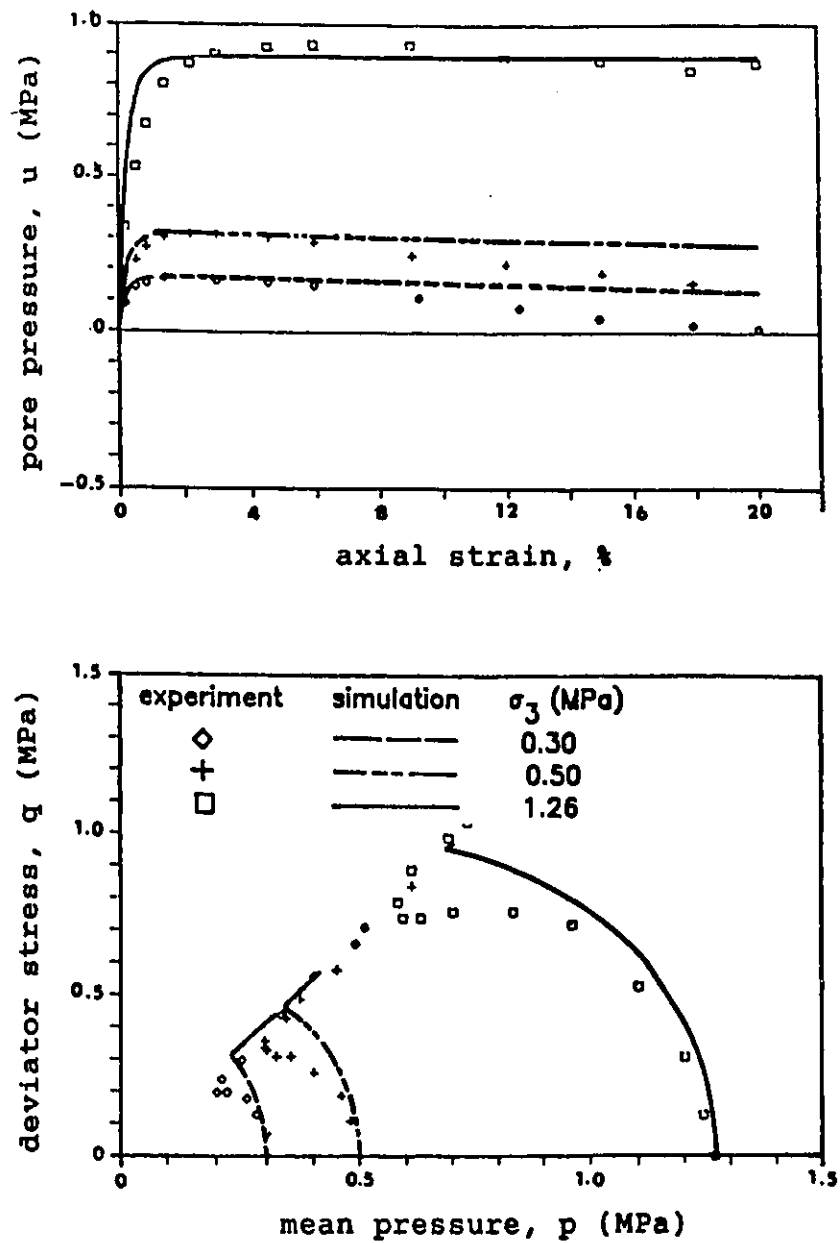


Fig. 5.4 Model simulation and experimental results of undrained tests at different confining pressures on loose Sacramento River sand
(Experimental results are from Seed and Lee, 1967)

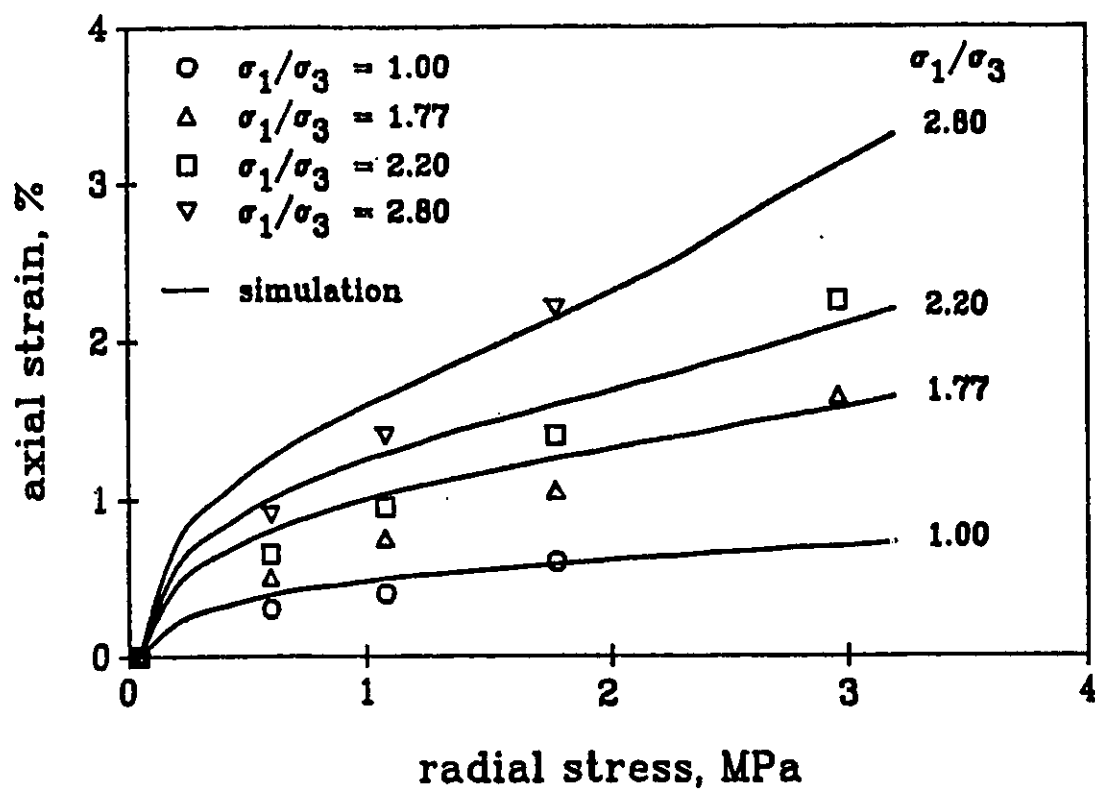


Fig. 5.5 Model simulation and experimental results of proportional loading test on dense Sacramento River sand, axial strain vs radial stress
(Experimental results are from Lade, 1975)

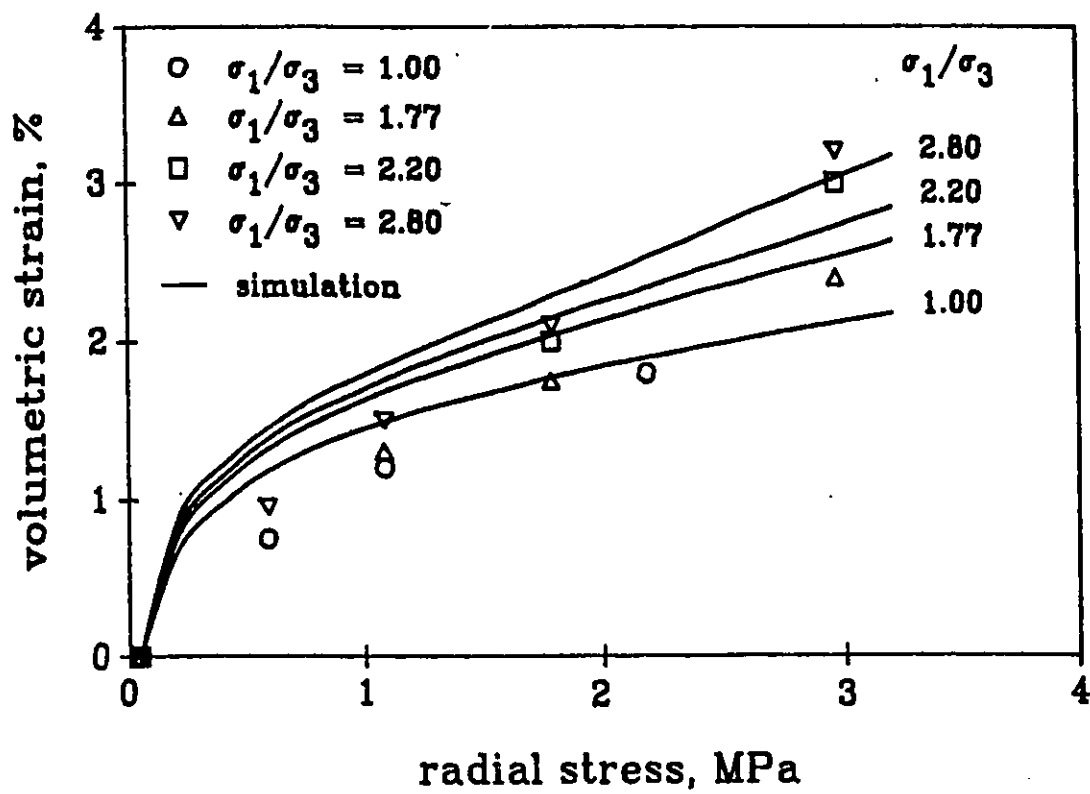


Fig. 5.6 Model simulation and experimental results of proportional loading test on dense Sacramento River sand, volumetric strain vs radial stress (Experimental results are from Lade, 1975)

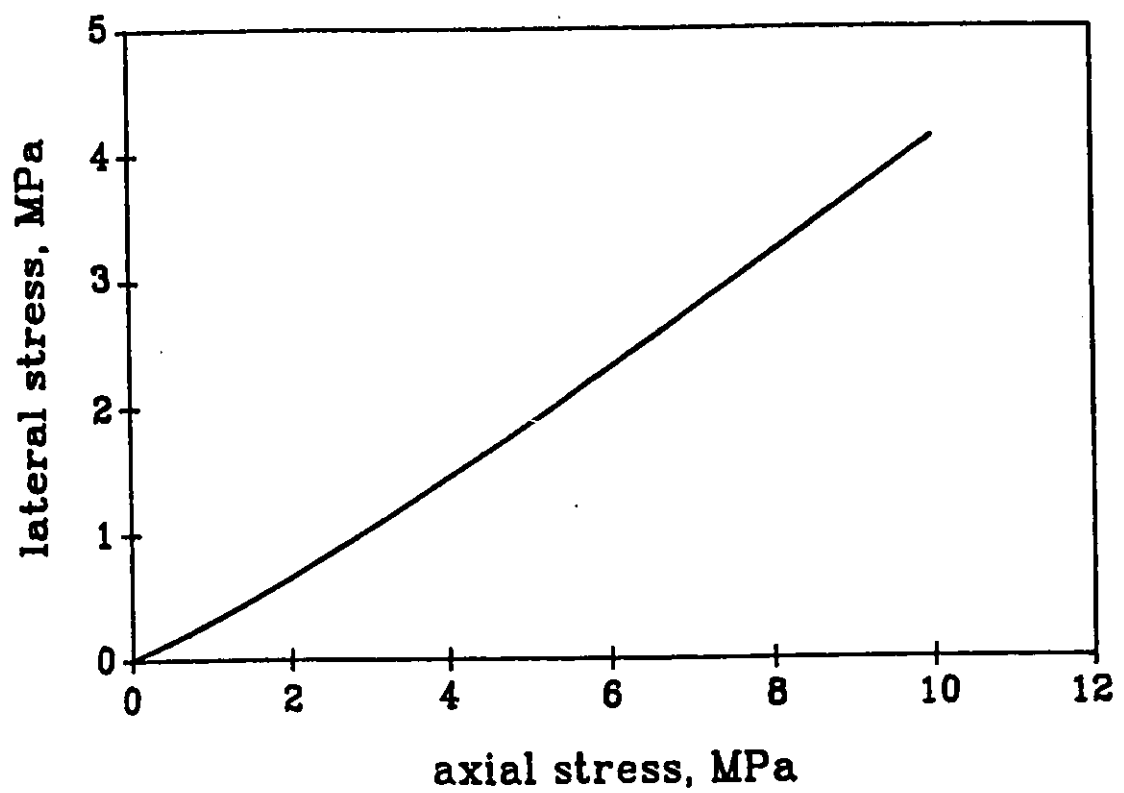


Fig. 5.7 Simulation of K_0 loading test on dense Sacramento River sand, lateral stress vs axial stress

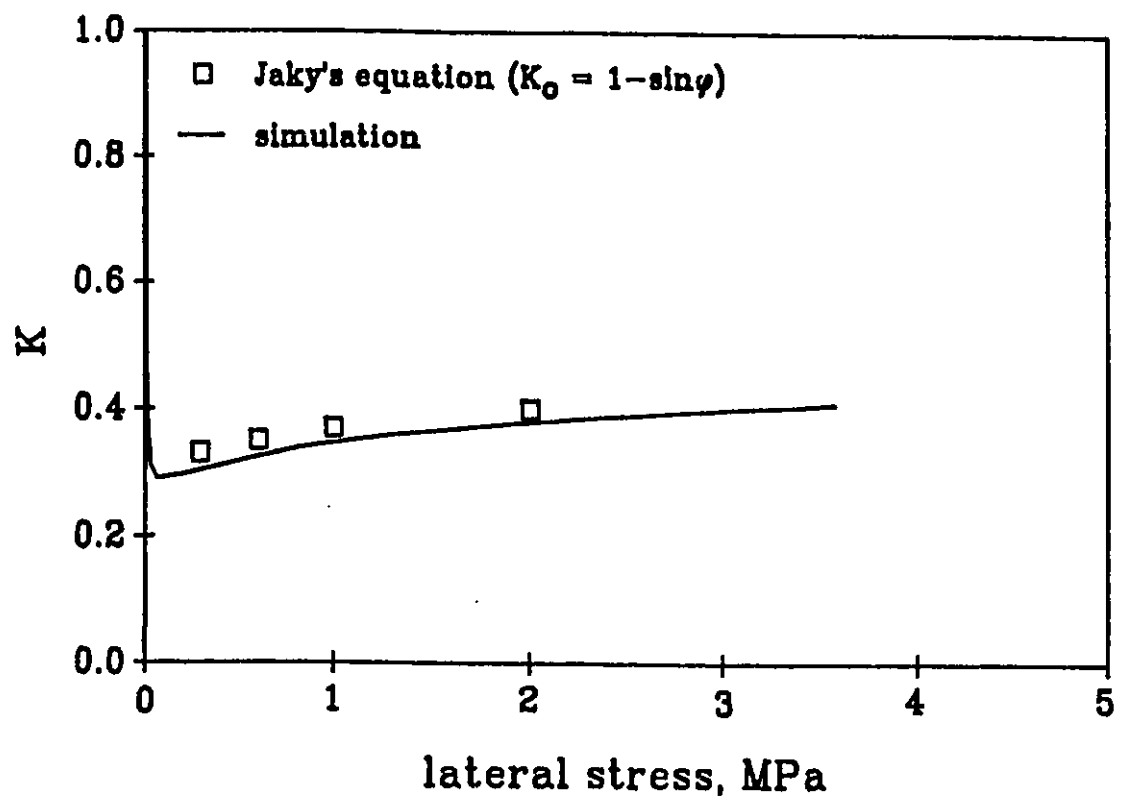


Fig. 5.8 Simulation of K_0 loading test on dense Sacramento River sand, K_0 versus lateral stress

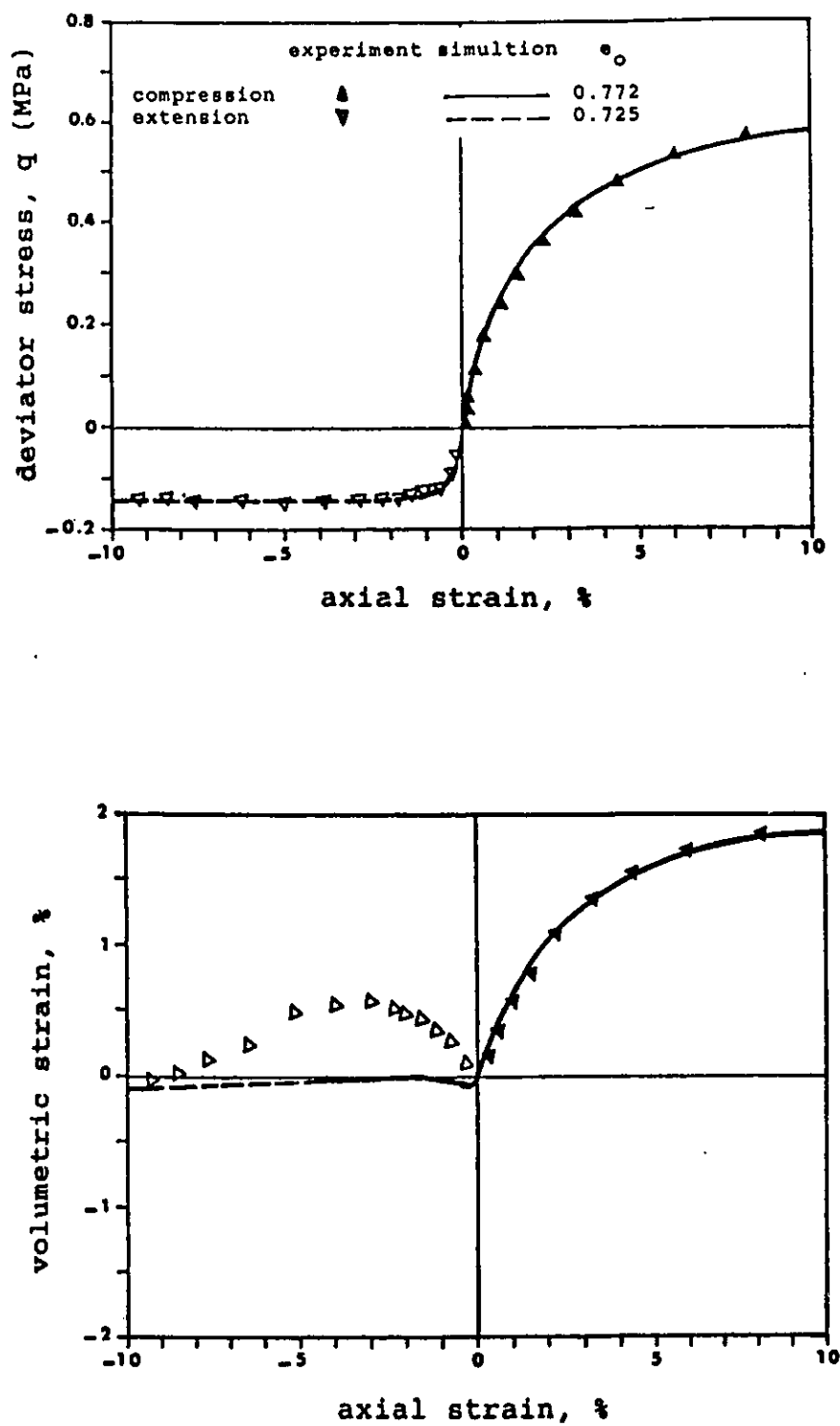


Fig. 5.9 Model simulation and experimental results of drained compression and extension tests at 200 kPa confining pressure on loose Fuji River sand
(Experimental results are from Tatsuoka and Ishihara, 1974)

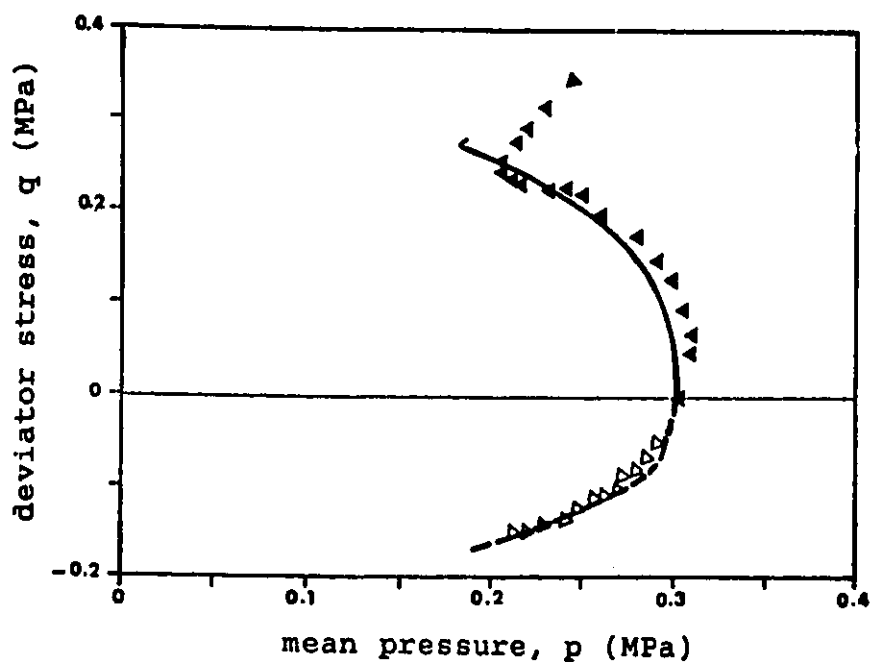
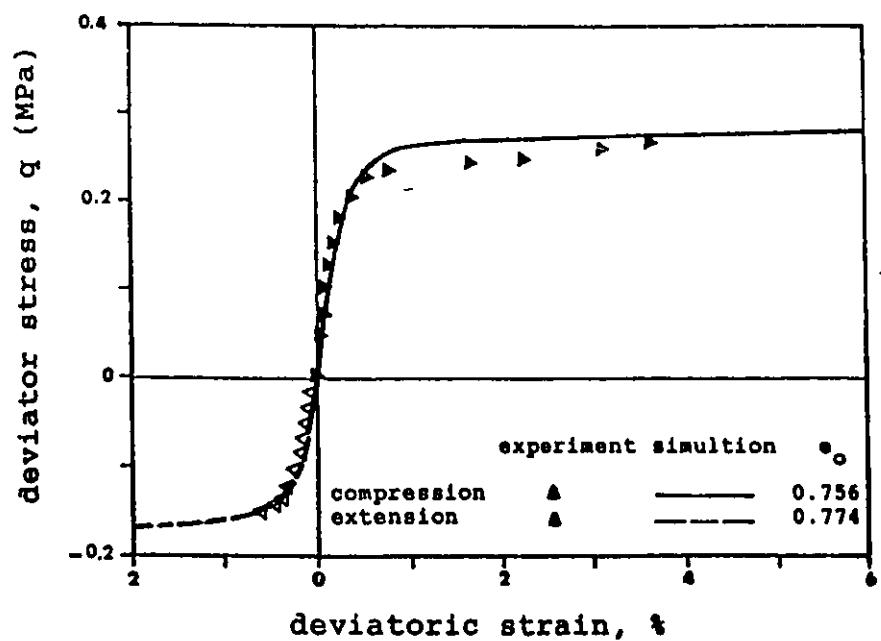


Fig. 5.10 Model simulation and experimental results of undrained compression and extension tests at 300 kPa confining pressure on loose Fuji River sand (Experimental results are from Ishihara et al., 1975)

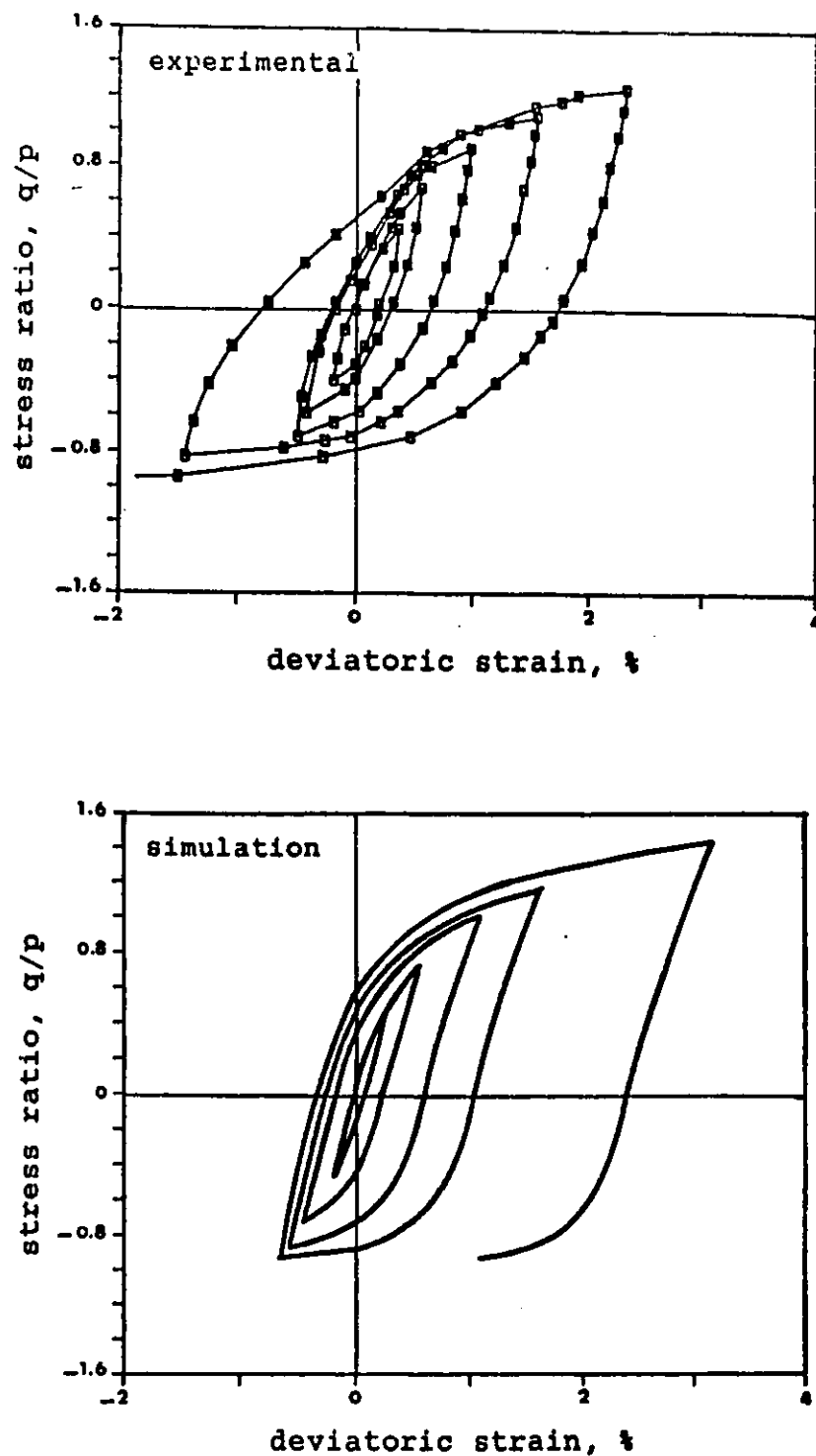


Fig. 5.11 Model simulation and experimental stress-strain curves of drained tests at 200 kPa confining pressure with cycles of increasing deviator stress on loose Fuji River sand (Experimental results are from Tatsuoka and Ishihara, 1974)

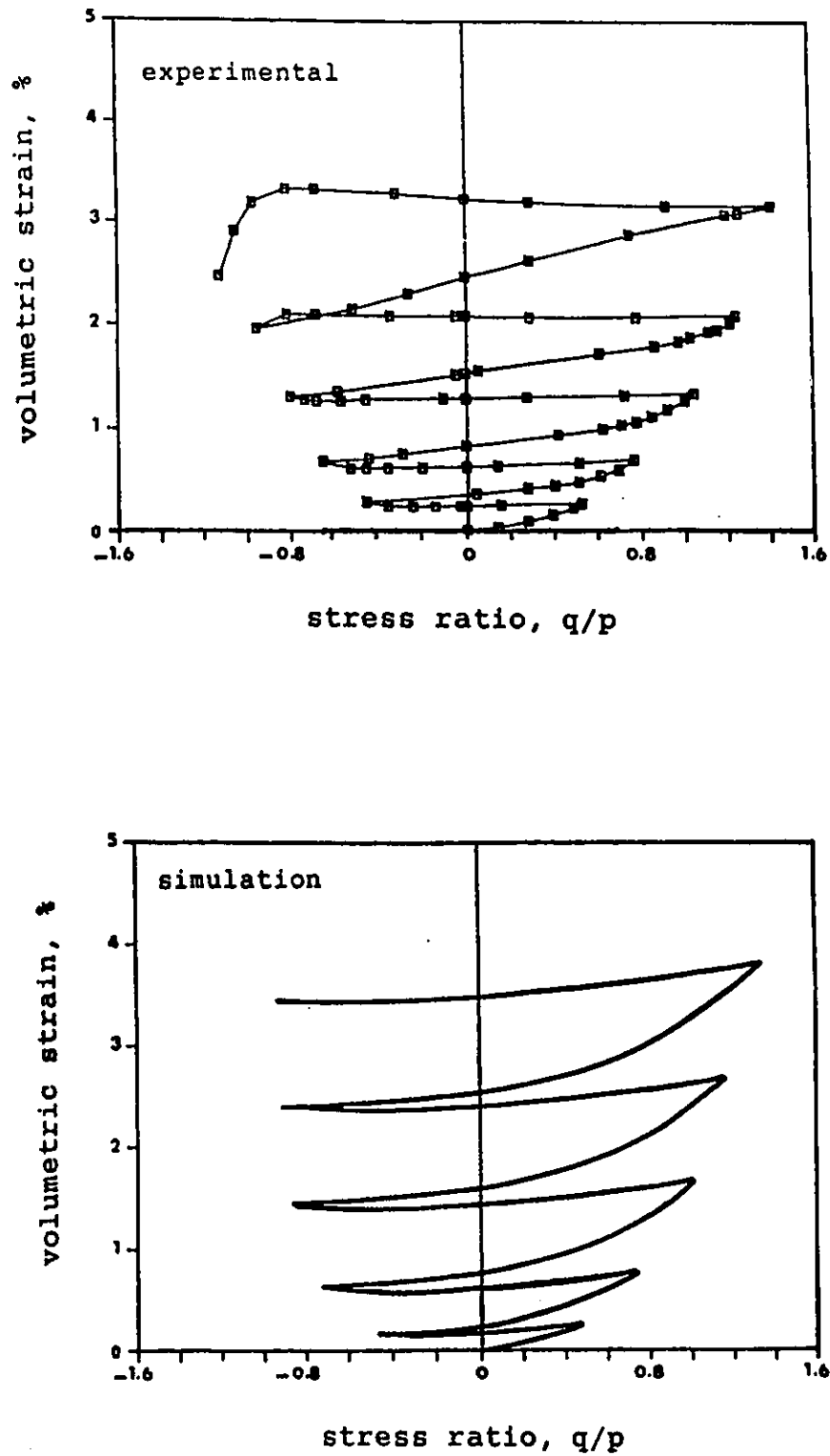


Fig. 5.12 Model simulation and experimental volumetric strain of the drained test shown in Fig. 5.11

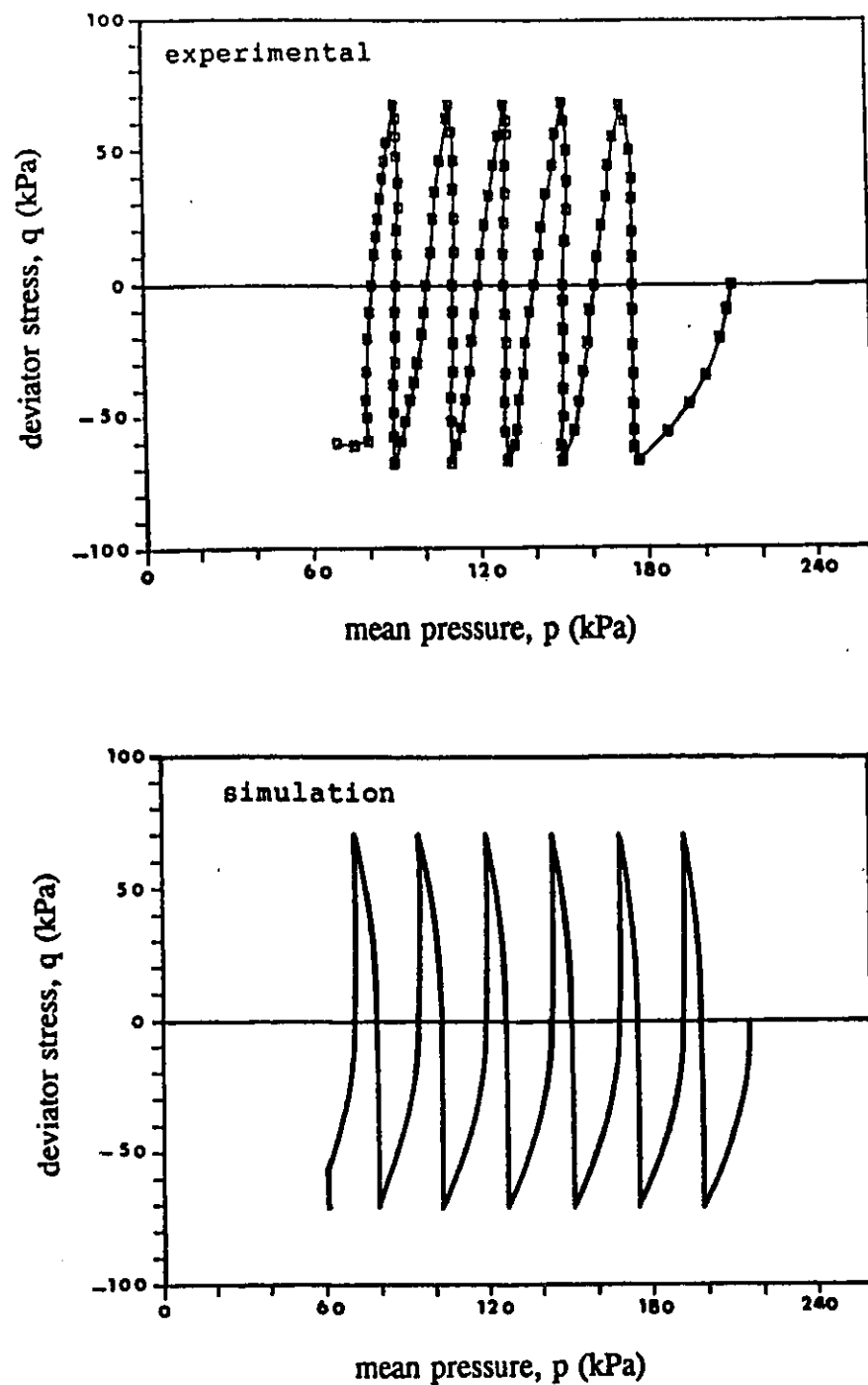


Fig. 5.13 Model simulation and experimental effective stress paths of the undrained test with stress cycles of constant amplitude on loose Fuji River sand (Experimental results are from Ishihara et al., 1975)

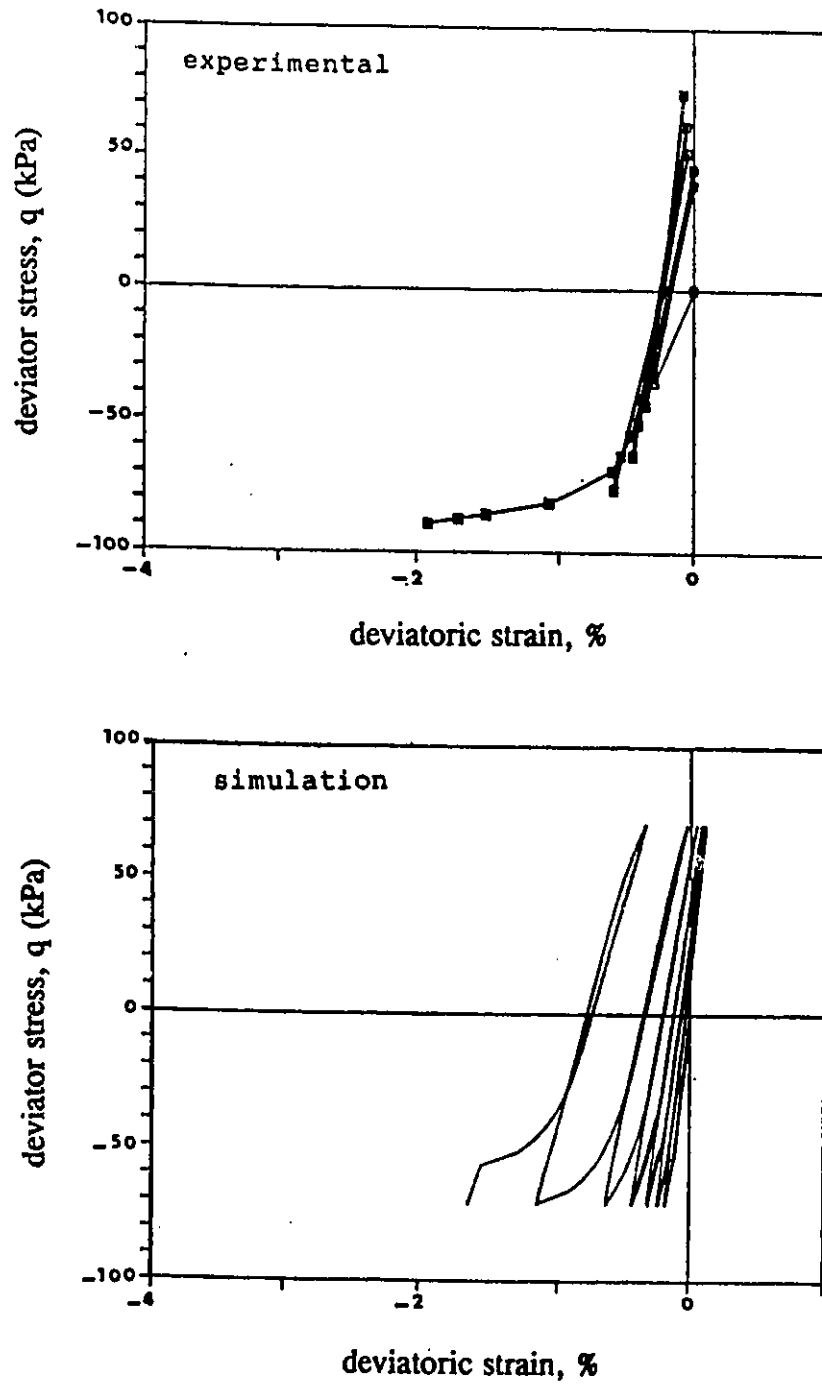


Fig. 5.14 Model simulation and experimental stress-strain curves of the undrained test shown in Fig. 5.13

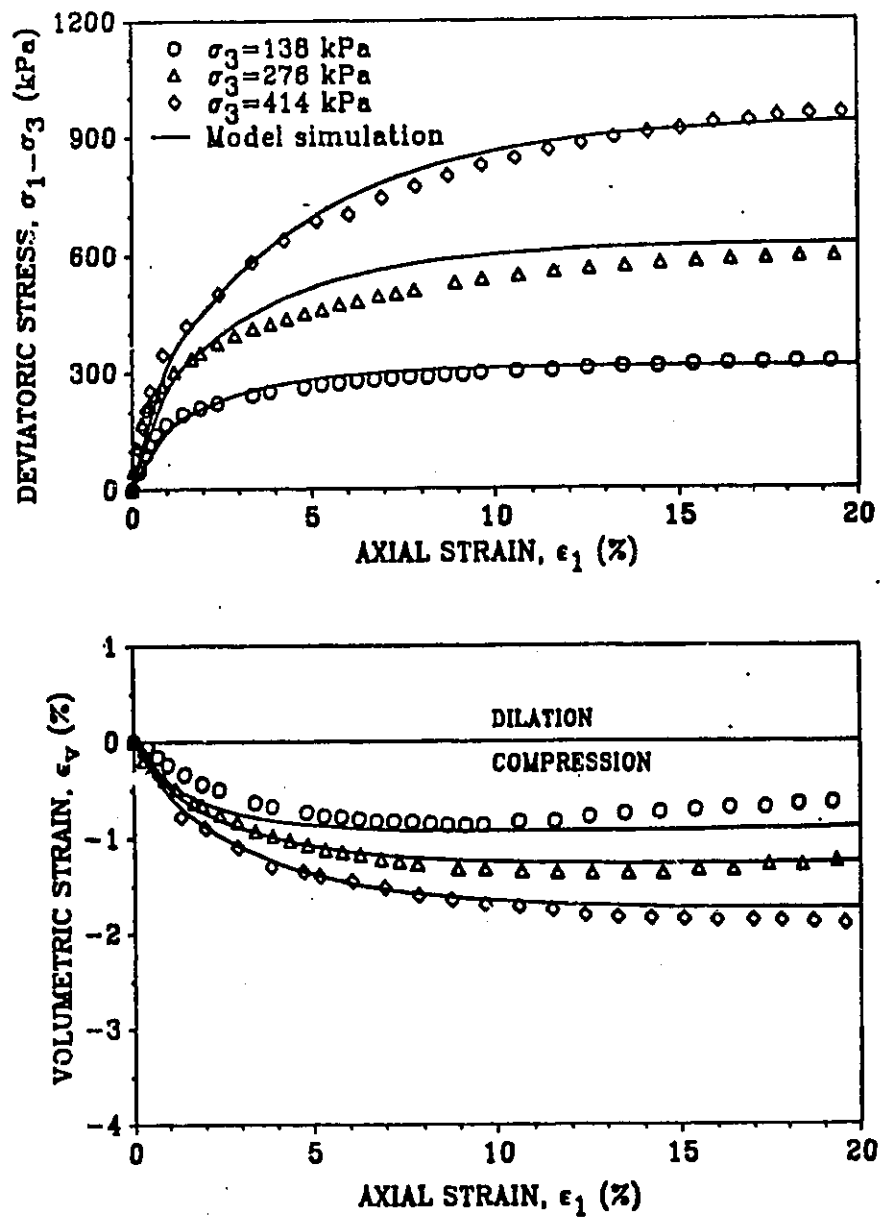


Fig. 5.15 Model simulation and experimental results of drained tests at different confining pressures on clean loose crushed quartz sand

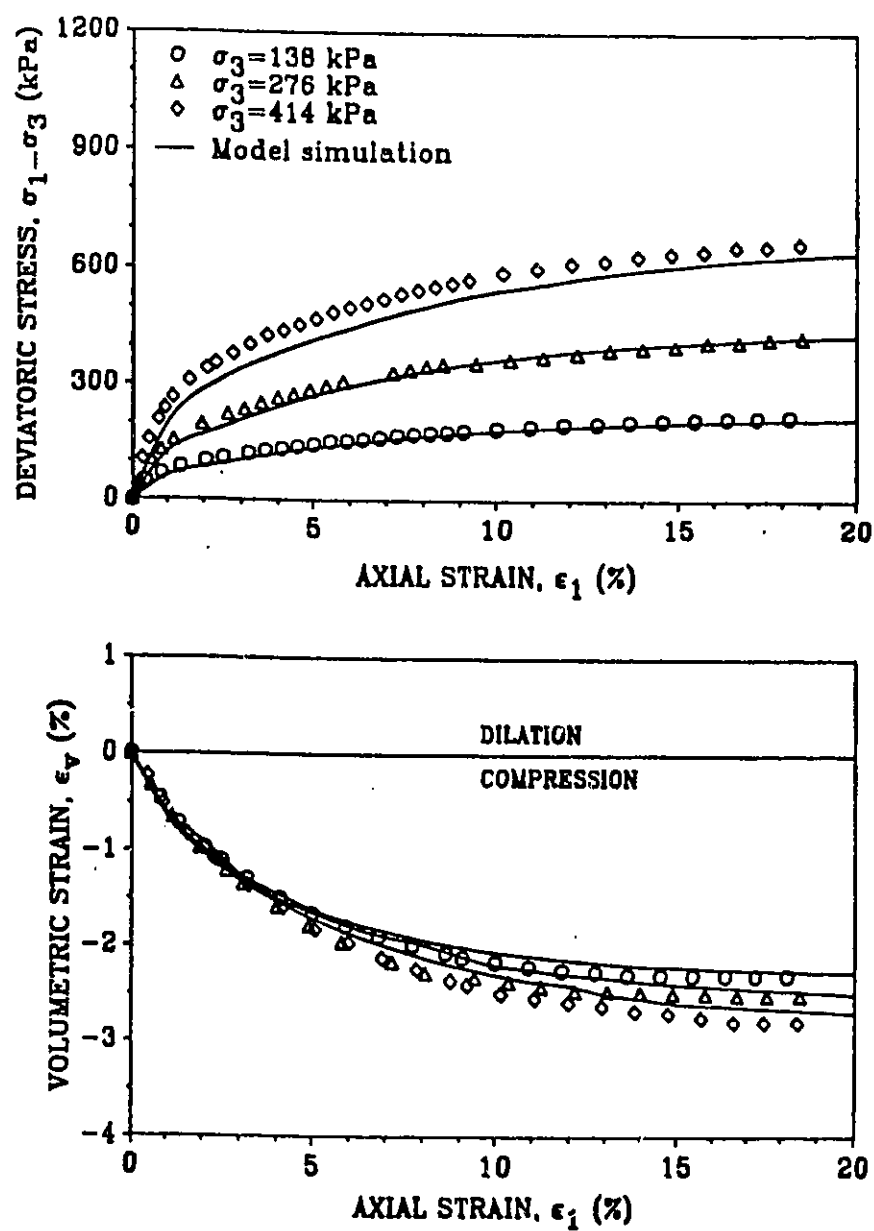


Fig. 5.16 Model simulation and experimental results of drained tests at different confining pressures on oil contaminated loose crushed quartz sand

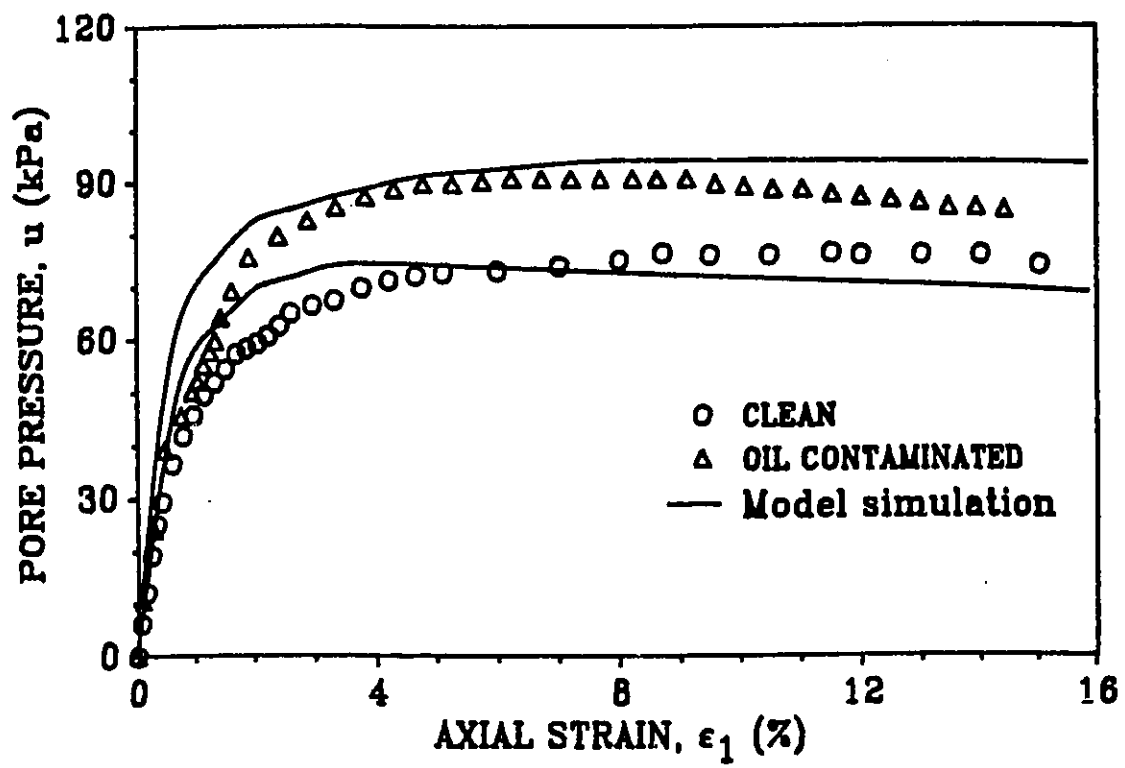


Fig. 5.17 Simulated and measured pore fluid pressure in undrained triaxial tests on clean and oil contaminated loose sand

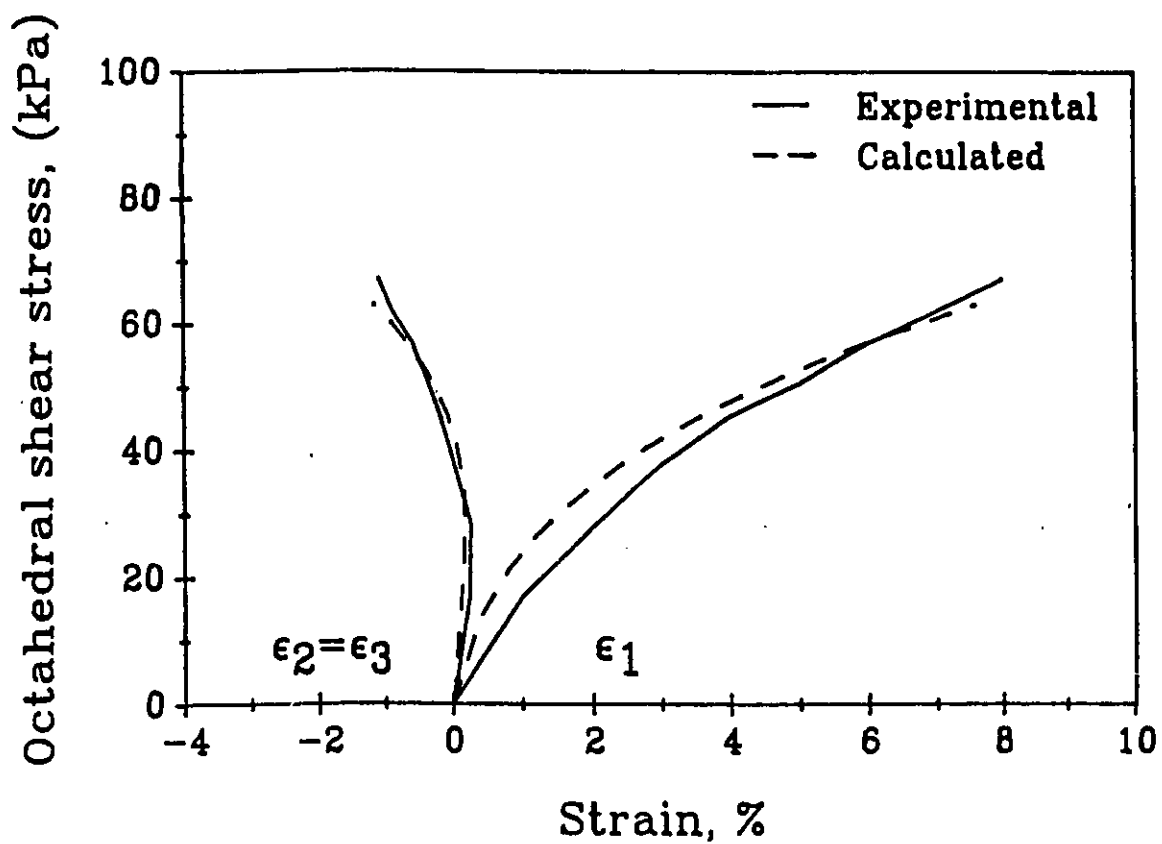


Fig. 5.18 Comparison of stress-strain response of conventional triaxial compression test (CTC10) of the artificial soil (Experimental results are from Desai and Siriwardane, 1984)

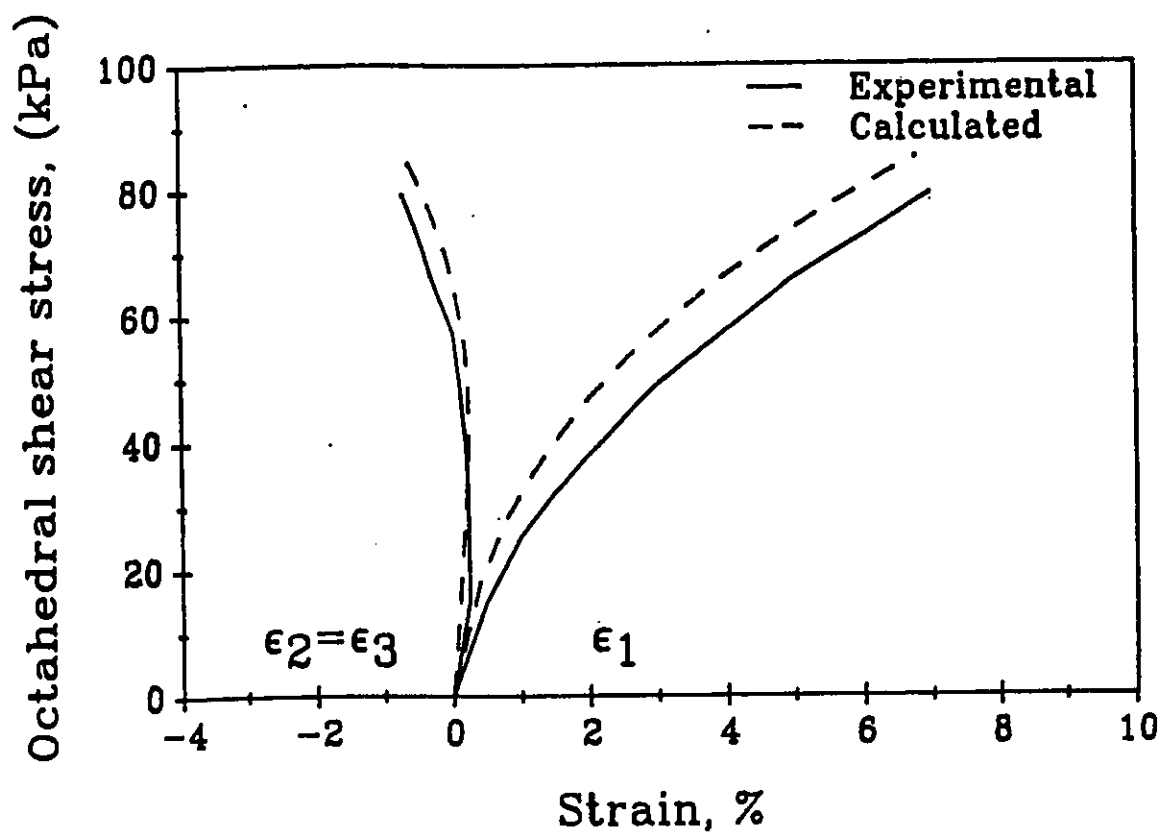


Fig. 5.19 Comparison of stress-strain response of conventional triaxial compression test (CTC15) of the artificial soil (Experimental results are from Desai and Siriwardane, 1984)

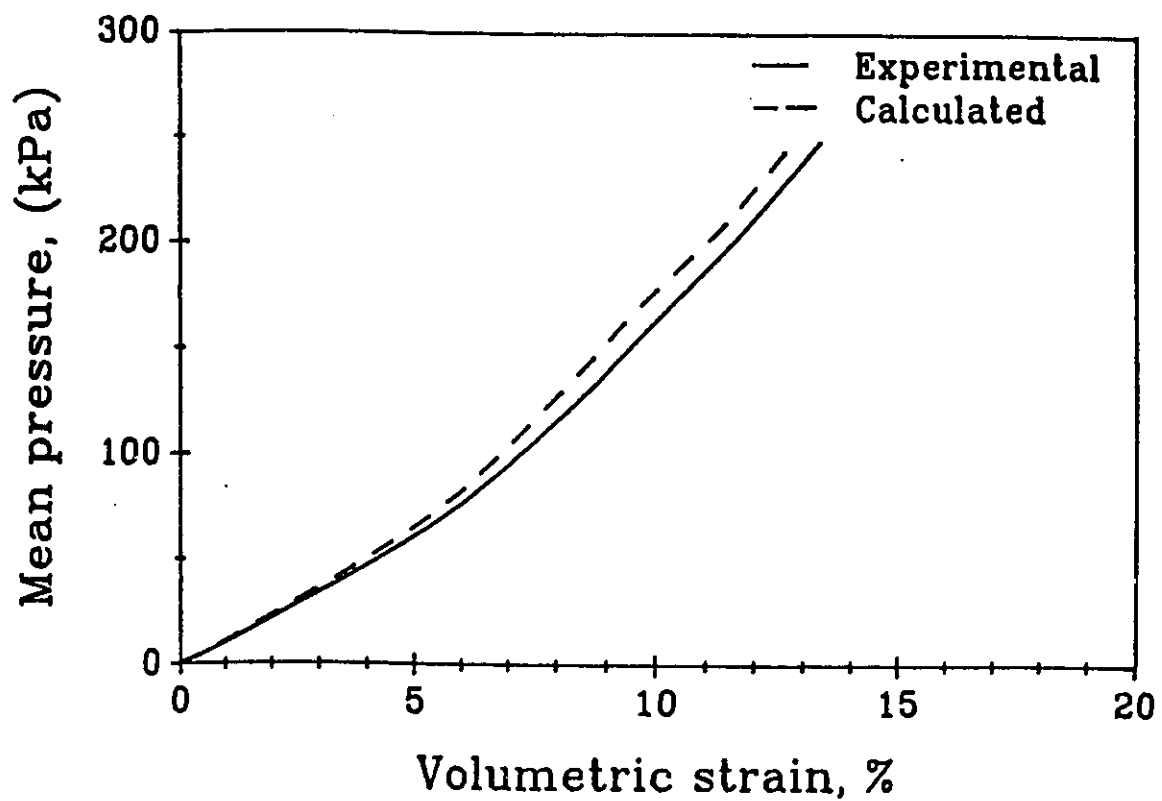


Fig. 5.20 Comparison of stress-strain response of isotropic compression test (HC) of the artificial soil
(Experimental results are from Desai and Siriwardane, 1984)

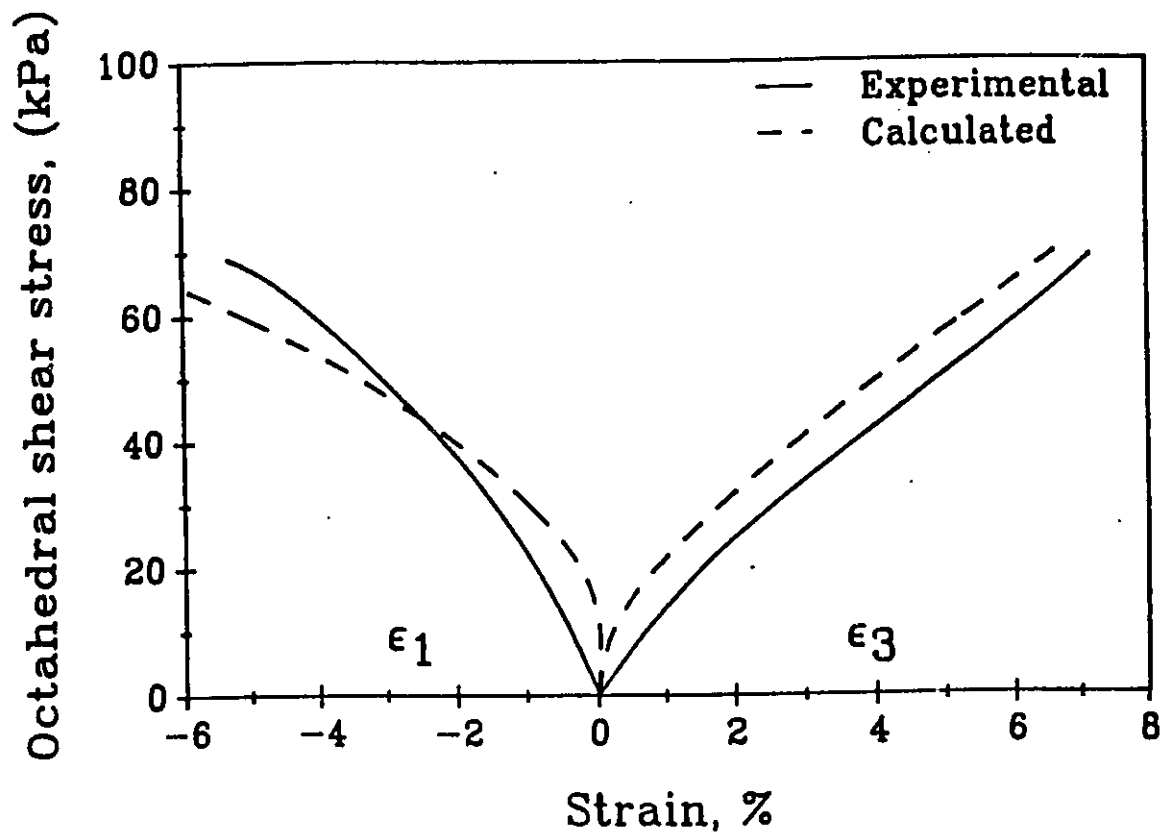


Fig. 5.21 Comparison of stress-strain response of simple shear test (SS10) of the artificial soil
(Experimental results are from Desai and Siriwardane, 1984)

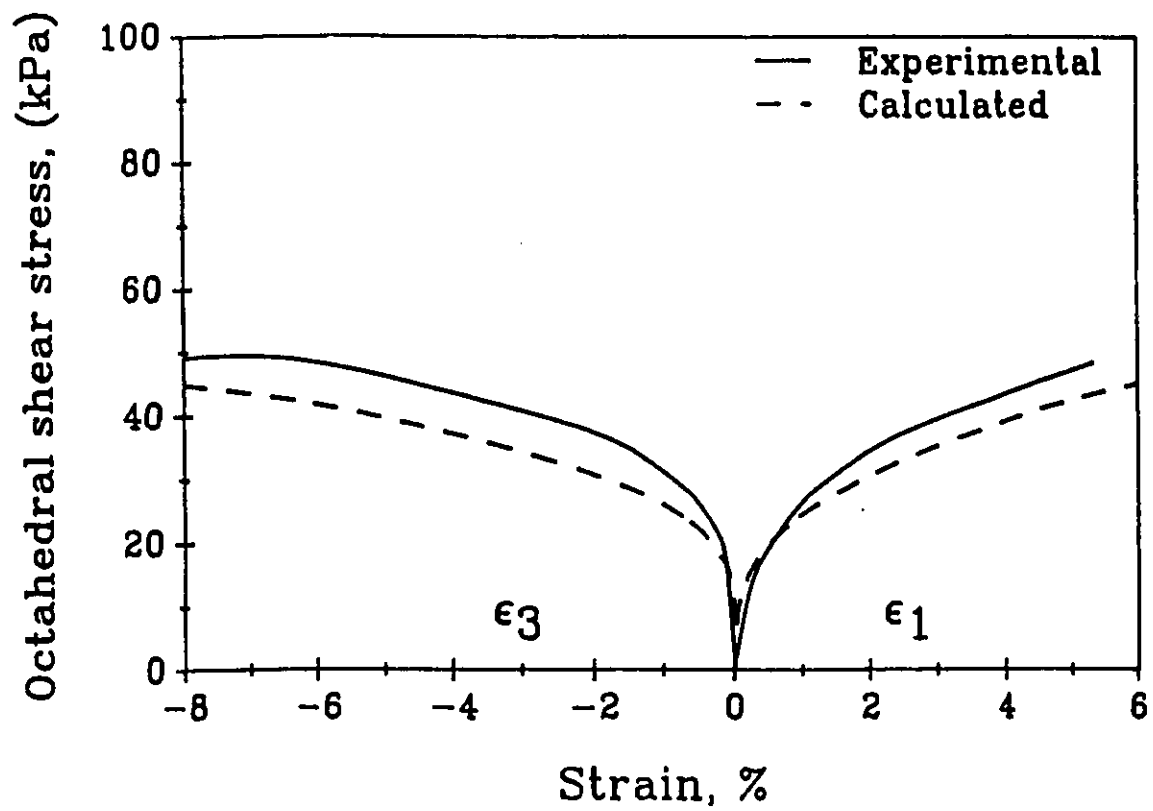


Fig. 5.22 Comparison of stress-strain response of conventional triaxial extension test (CTE10) of the artificial soil (Experimental results are from Desai and Siriwardane, 1984)

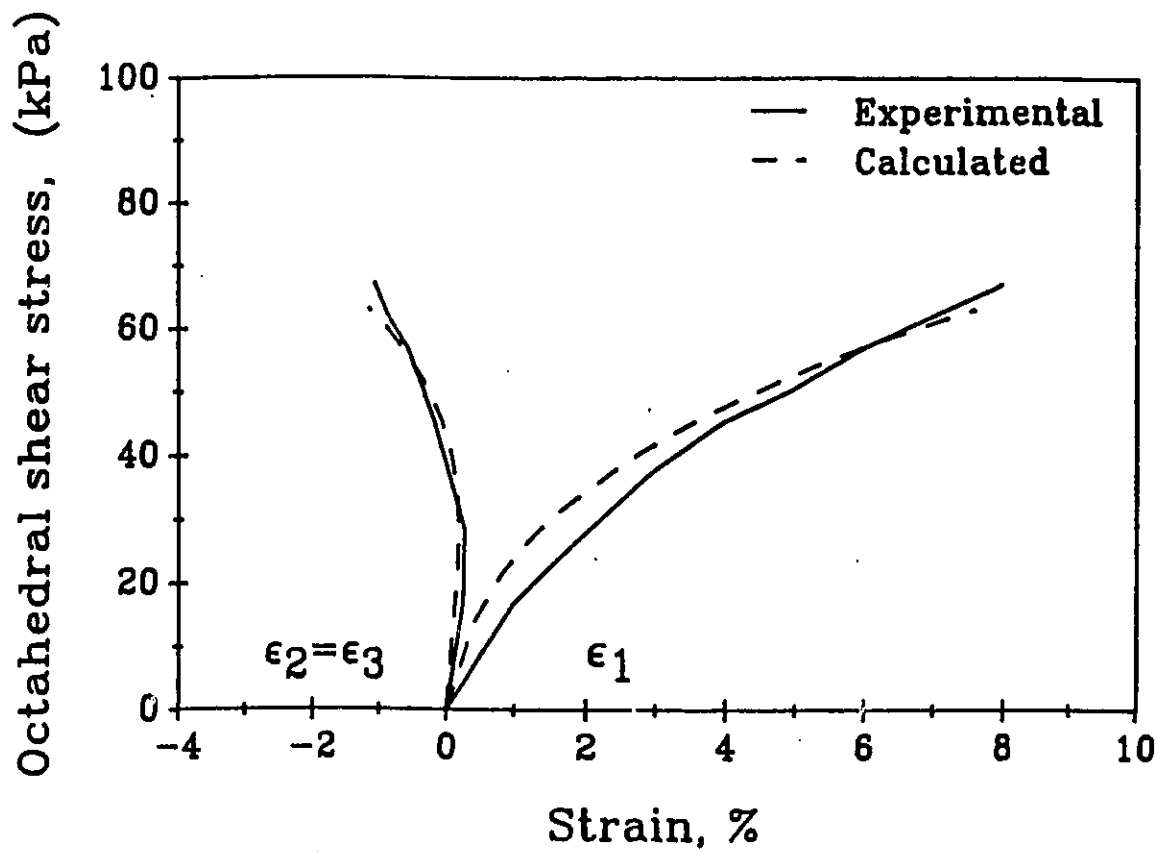


Fig. 5.23 Comparison of stress-strain response of triaxial compression test (TC10) of the artificial soil
(Experimental results are from Desai and Siriwardane, 1984)

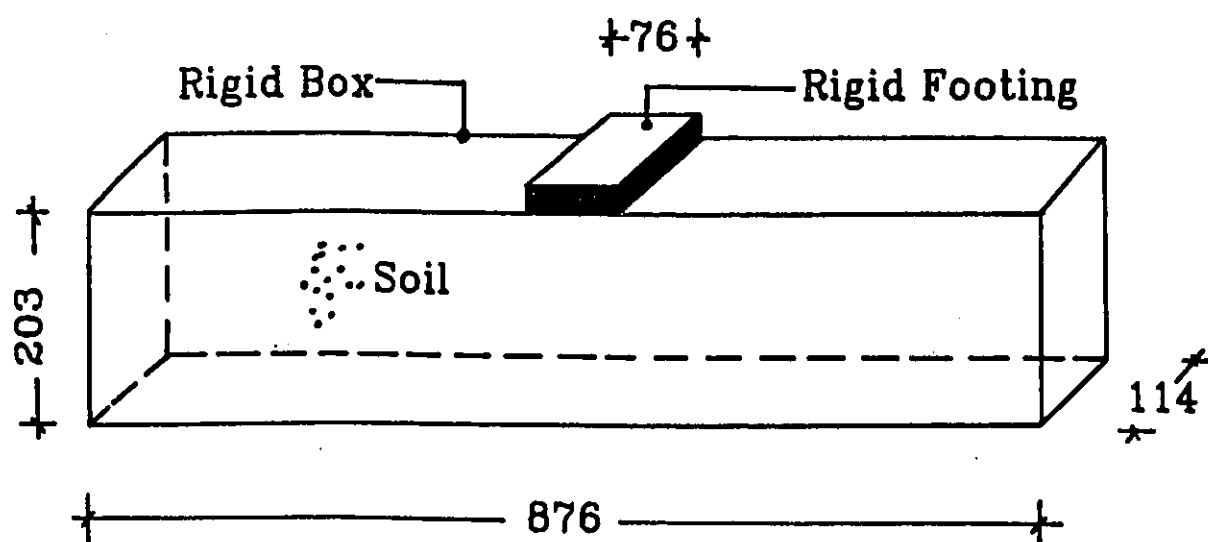


Fig. 6.1 Layout of the test box
(Modified after Desai and Siriwardane, 1984)

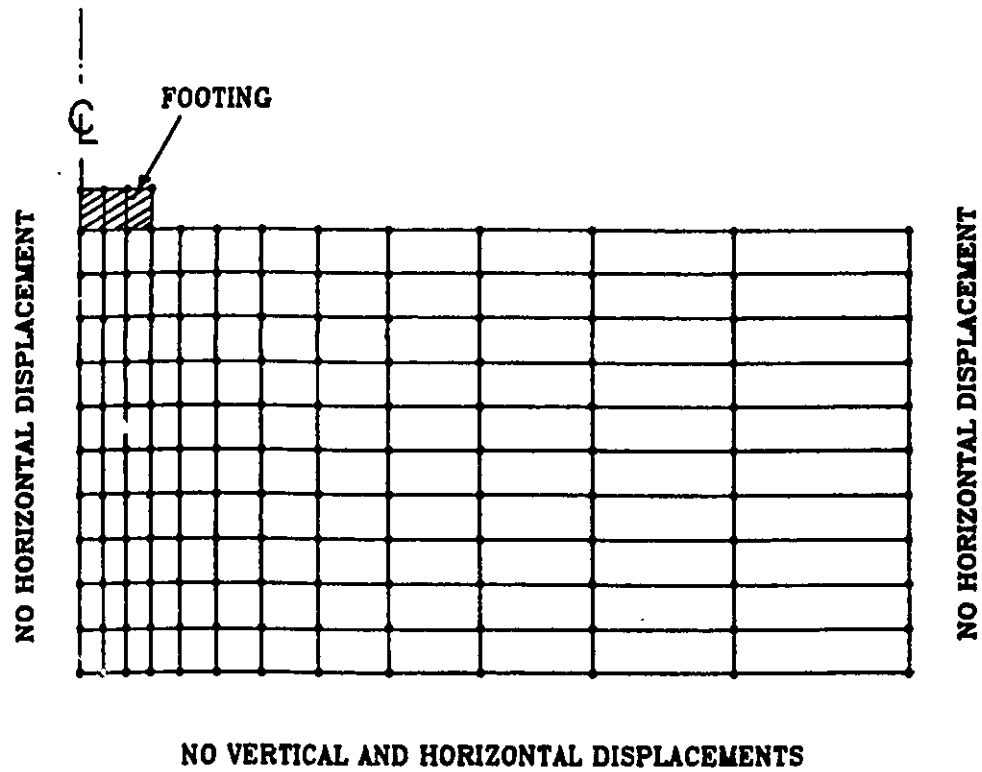


Fig. 6.2 Finite element mesh for the footing

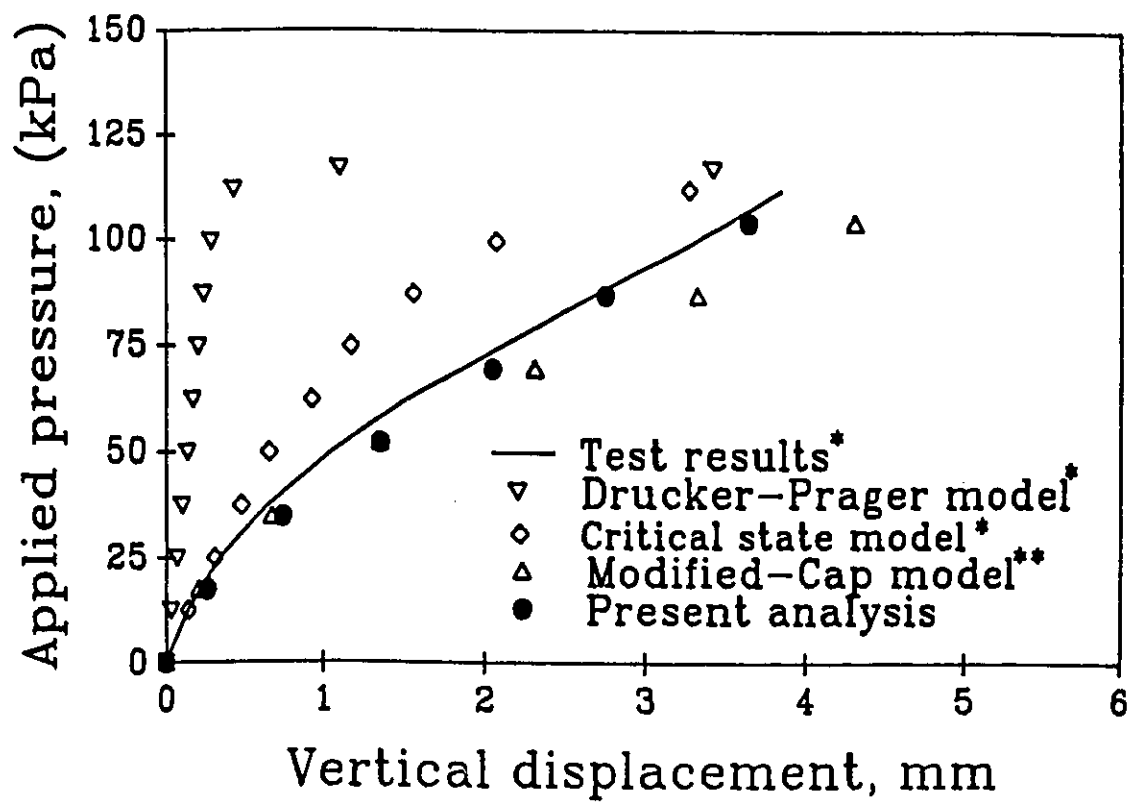
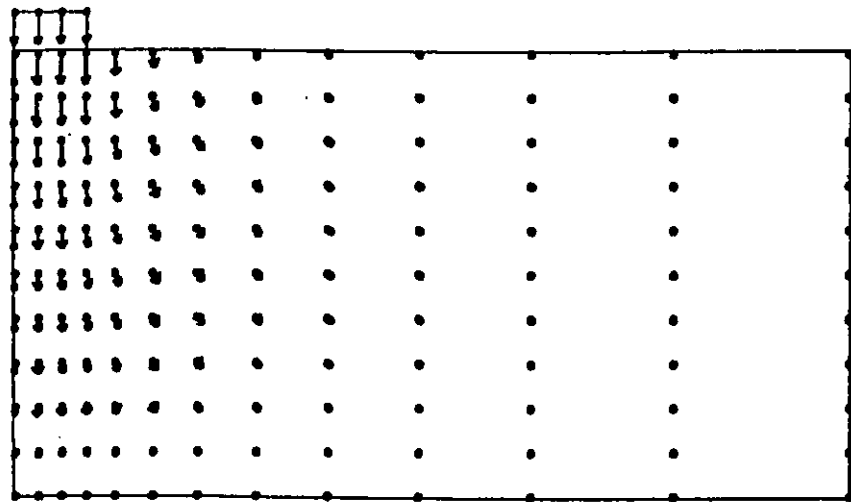


Fig. 6.3 Comparisons of calculations using various models
(Experimental results and predictions of other
models are from Desai and Siriwardane, 1984)



(displacements are multiplied by three)

Fig. 6.4 Computed displacement patterns

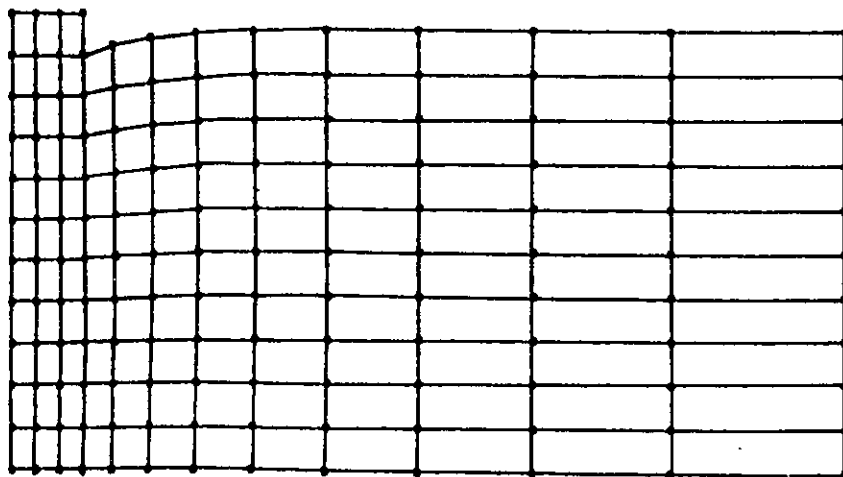


Fig. 6.5 Deformed finite element mesh

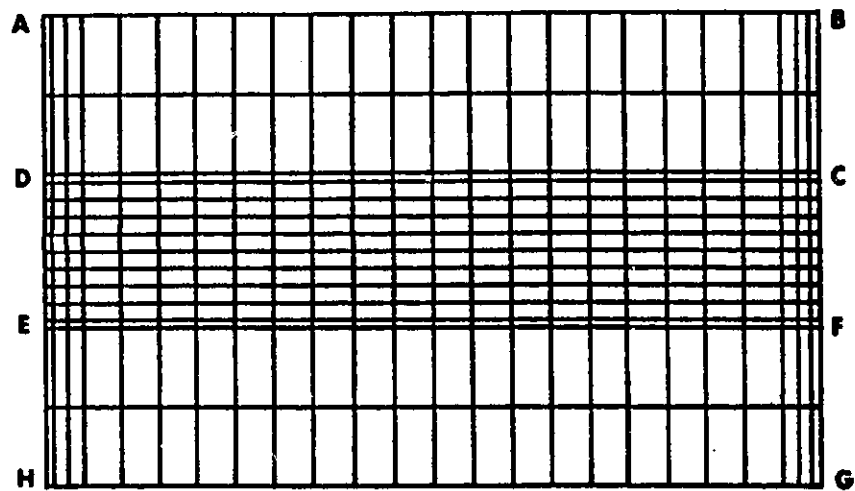


Fig. 6.6 Finite element mesh for the simple shear device

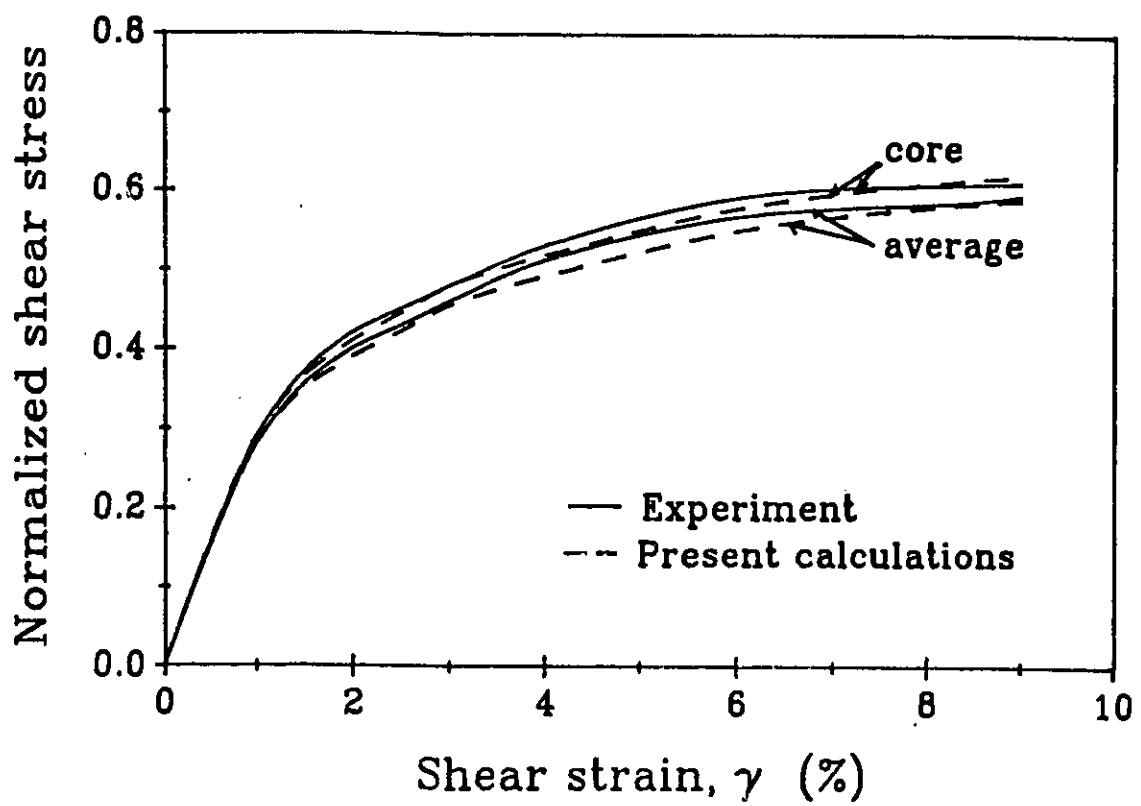


Fig. 6.7 Comparisons of normalized shear stress-strain response of Leighton Buzzard sand
(Experimental results are from Budhu and Britto, 1987)

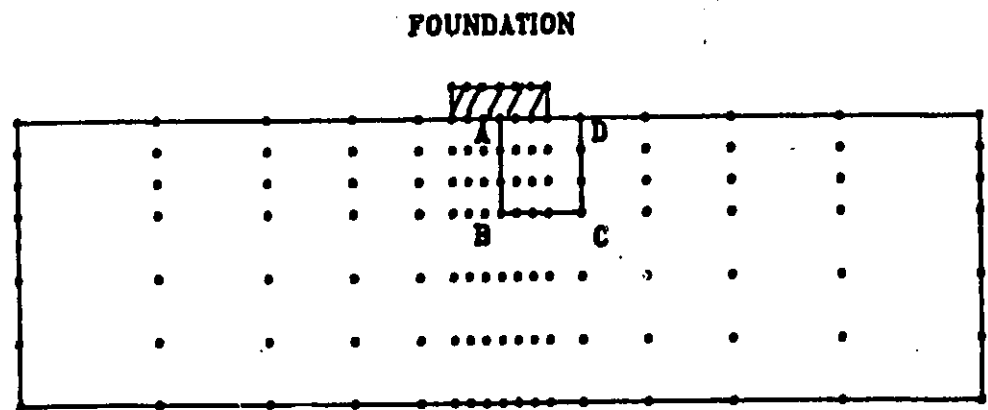


Fig. 6.8 Finite element mesh

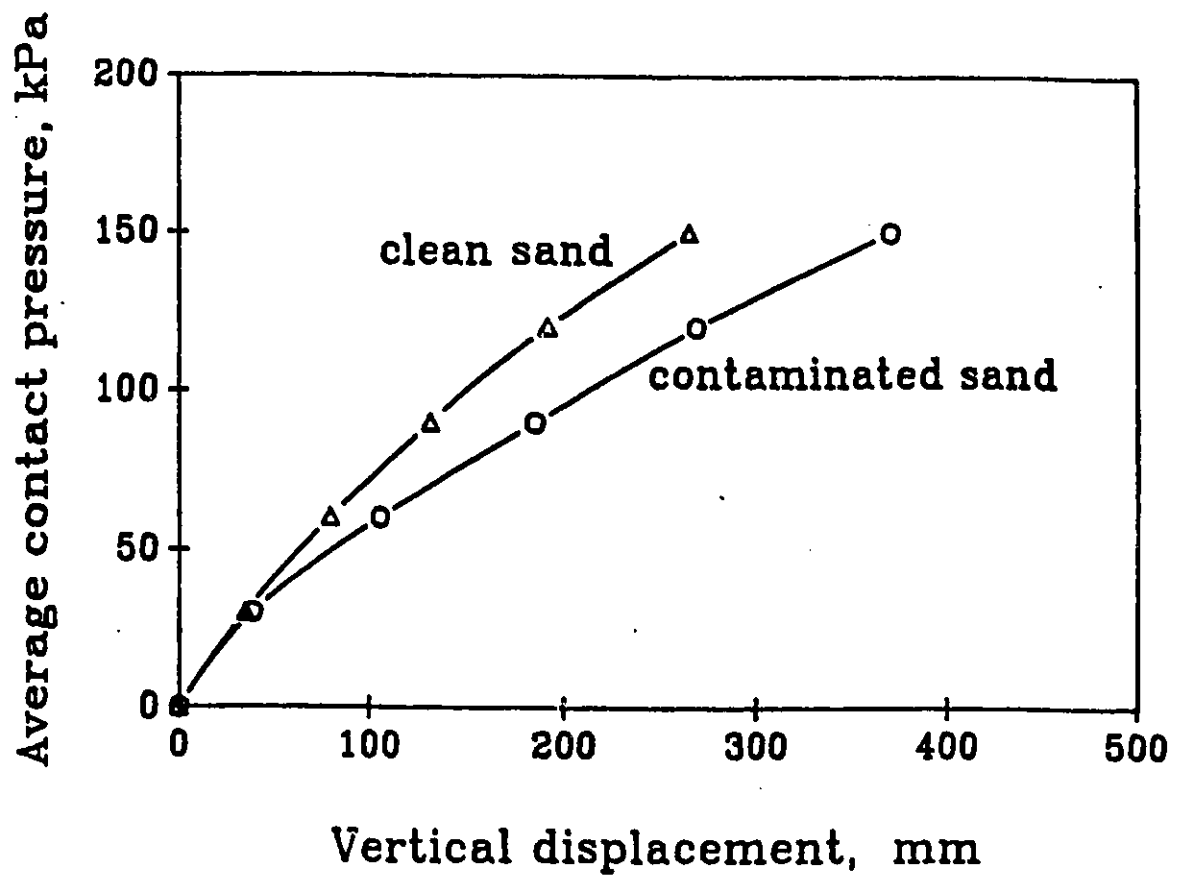


Fig. 6.9 Load-displacement behavior of a rigid foundation on clean and oil contaminated sands

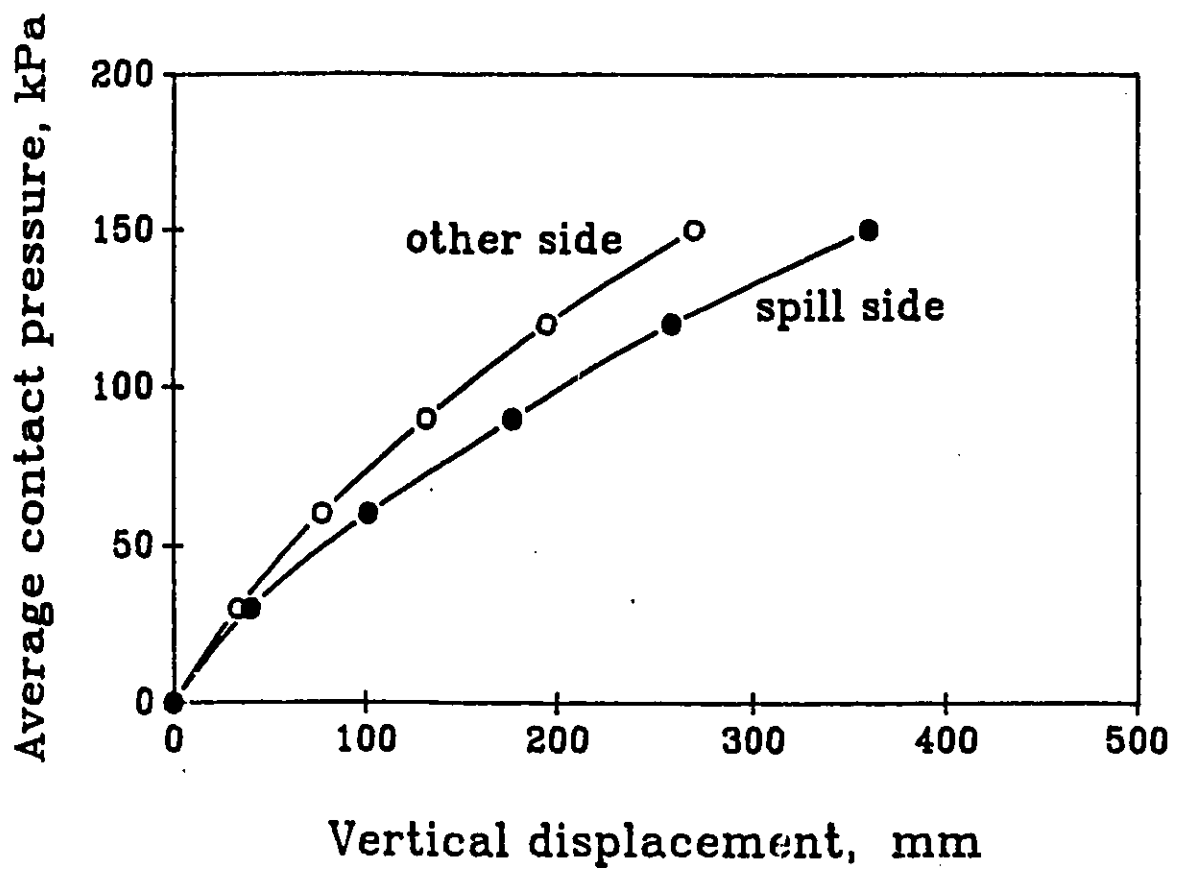


Fig. 6.10 Vertical load versus vertical displacement curves for two corners of a rigid foundation when a section of foundation soil is contaminated

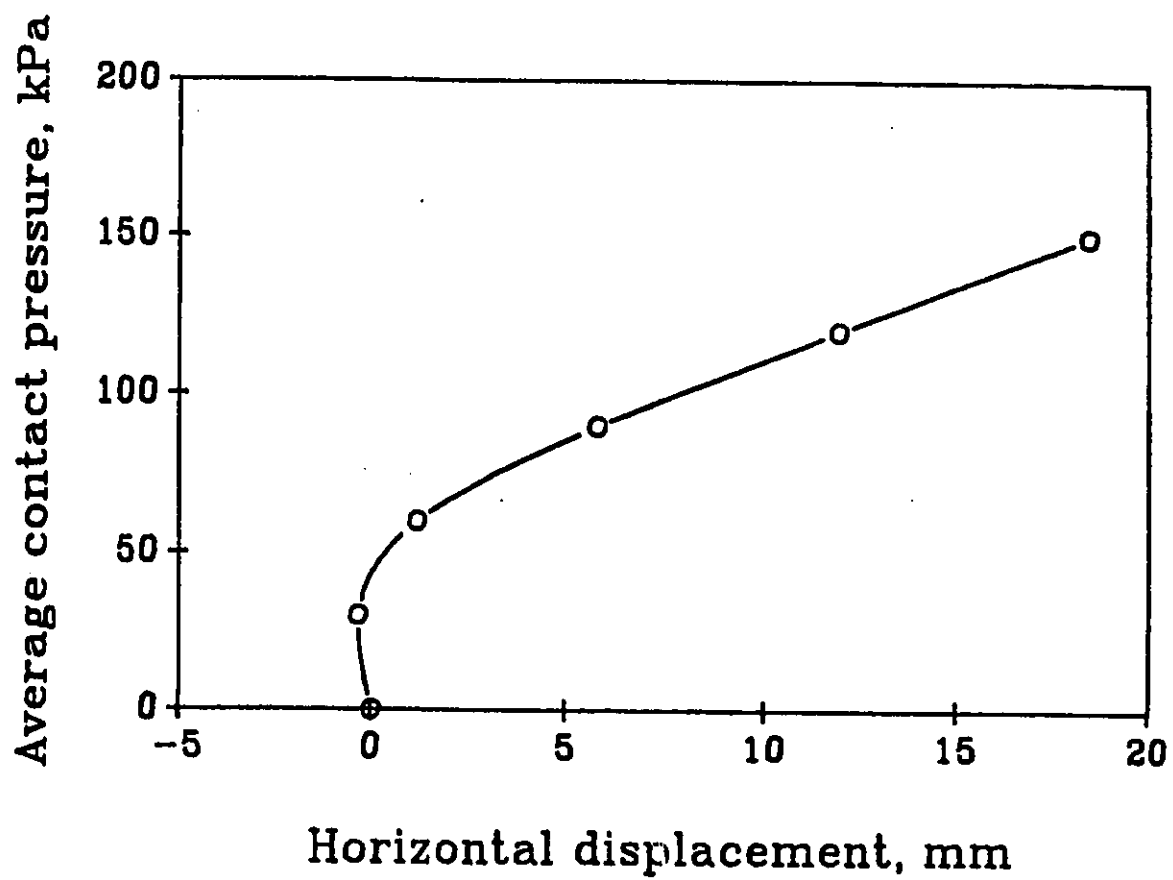


Fig. 6.11 Horizontal displacements of foundation center when a section of foundation soils is contaminated

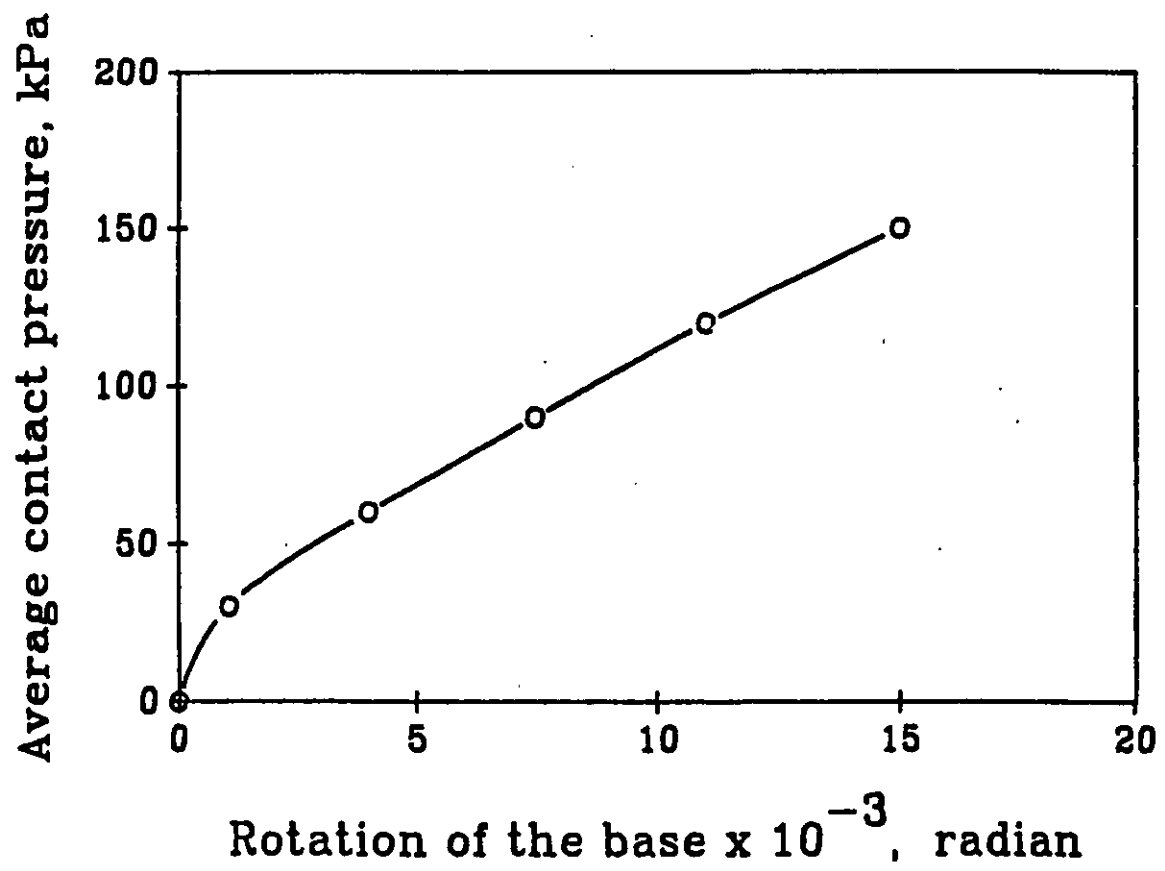


Fig. 6.12 Rotation of foundation base when a section of foundation soil is contaminated

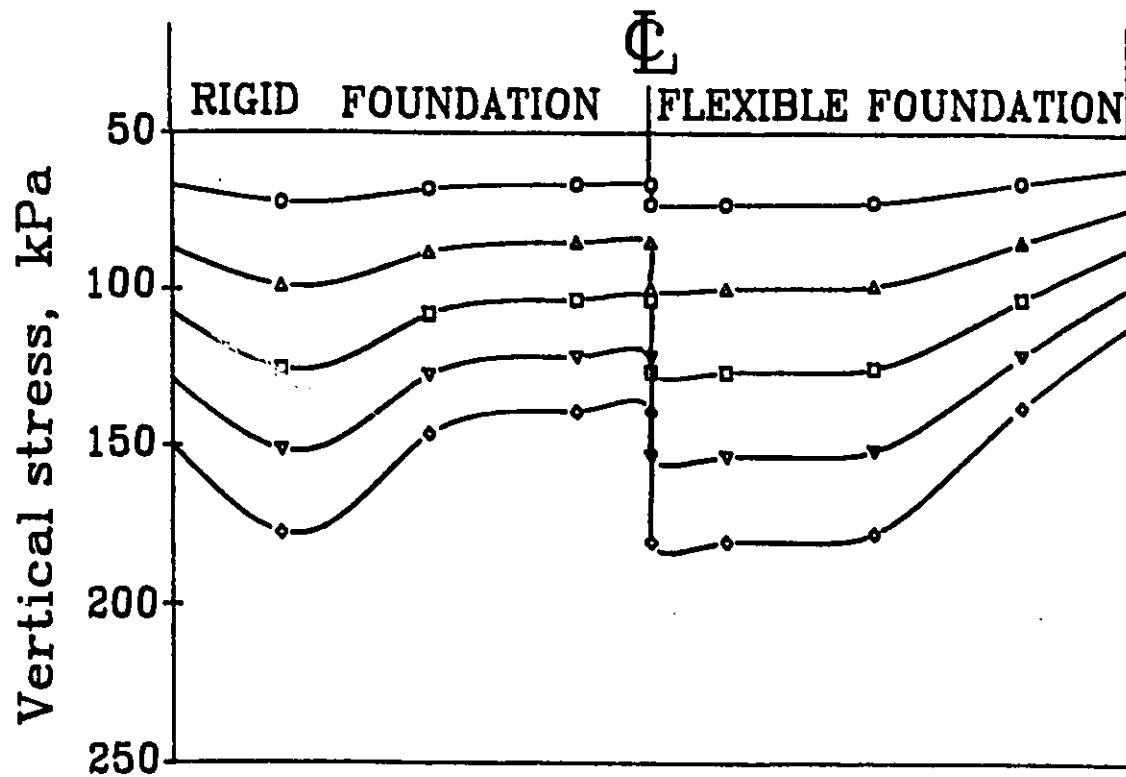


Fig. 6.13 Stress distributions in clean sand below a rigid foundation and a flexible foundation

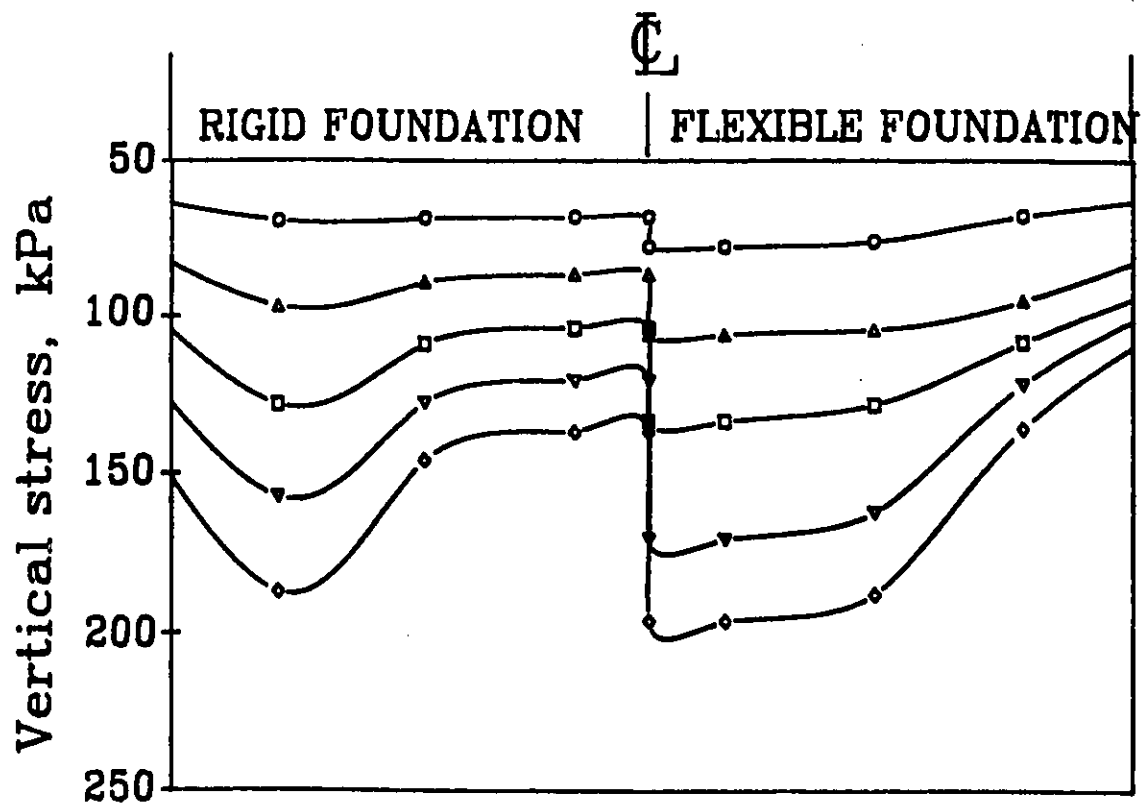


Fig. 6.14 Stress distributions in oil contaminated sand below a rigid foundation and a flexible foundation

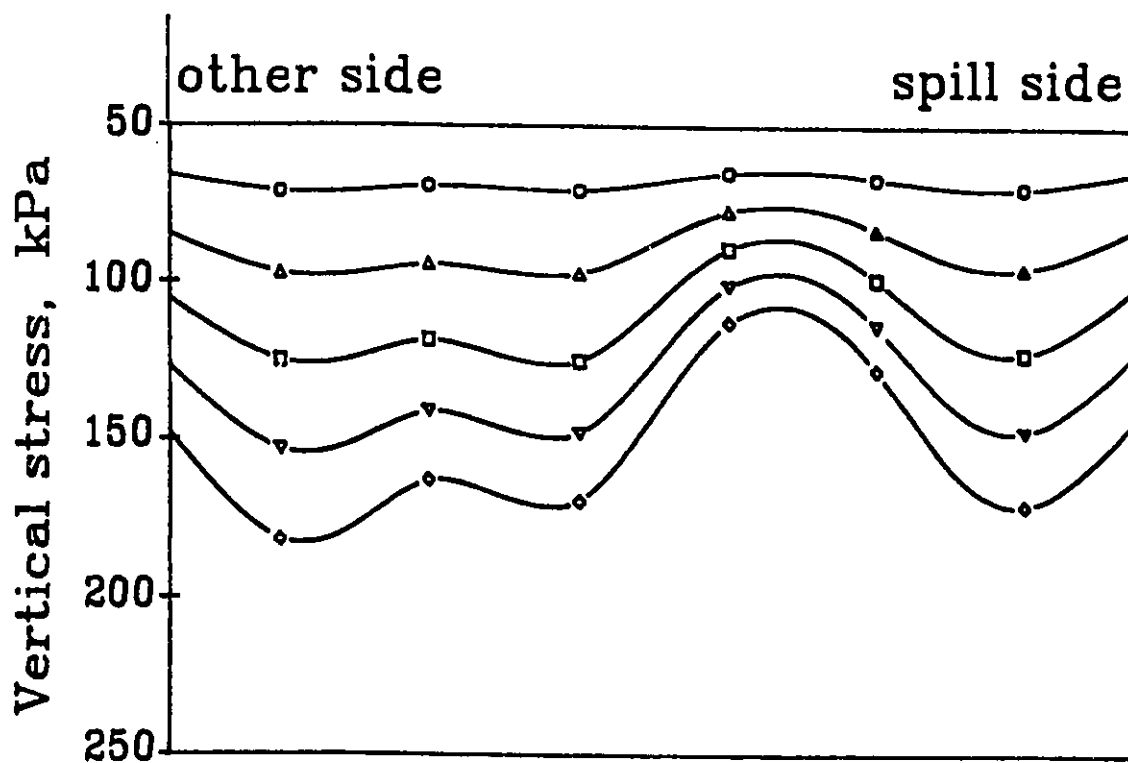


Fig. 6.15 Stress distribution in oil contaminated sand below a rigid foundation when a section of foundation soils is contaminated

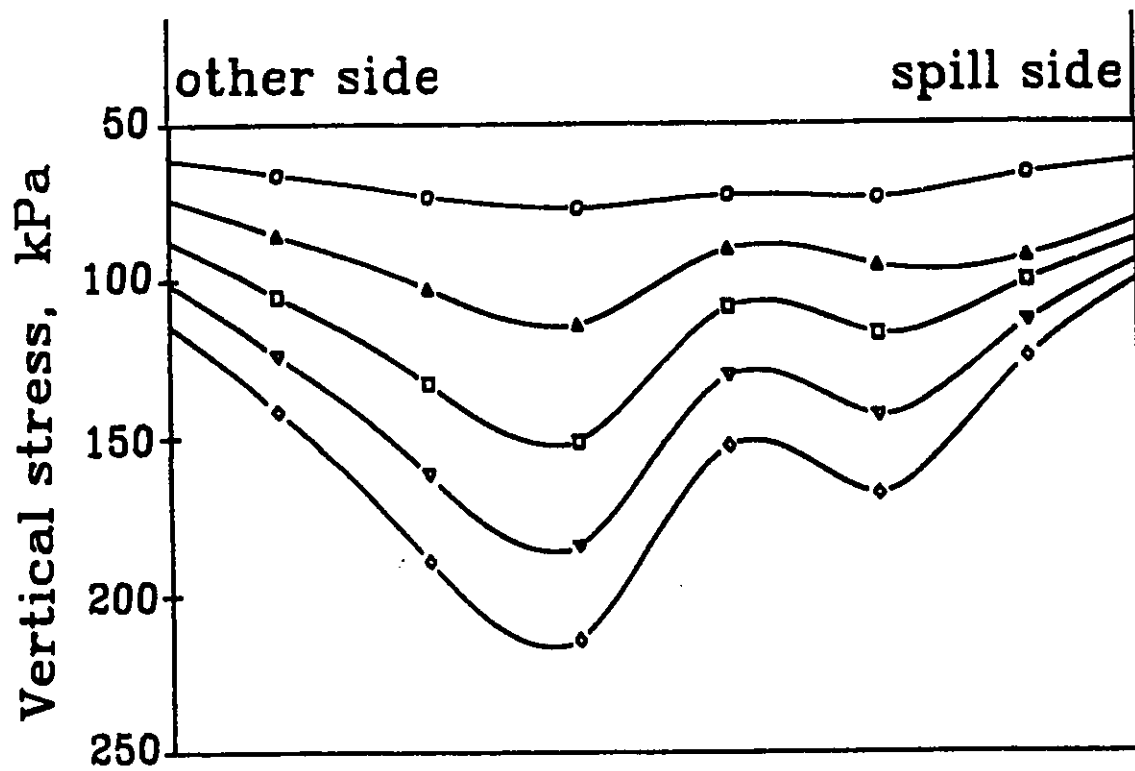
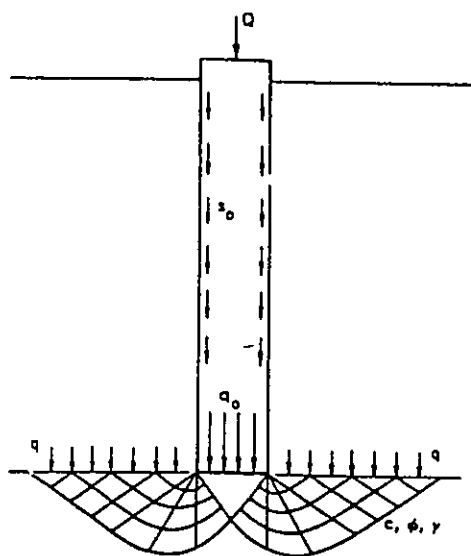
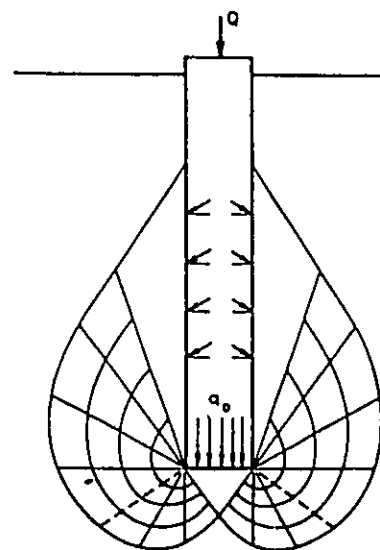


Fig. 6.16 Stress distribution in oil contaminated sand below a flexible foundation when a section of foundation soil is contaminated



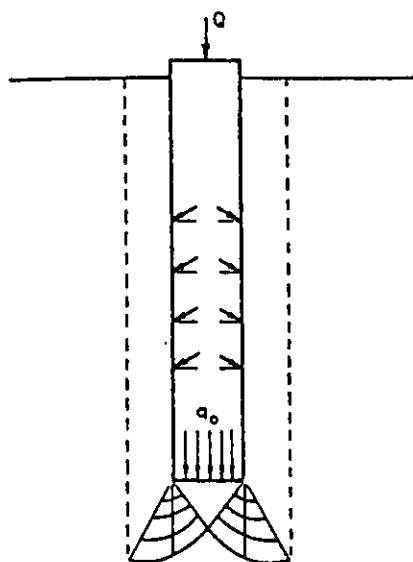
(a)

Prandtl (1921)
Reissner (1924)
Caquot (1934)
Buisman (1935)
Terzaghi (1943)



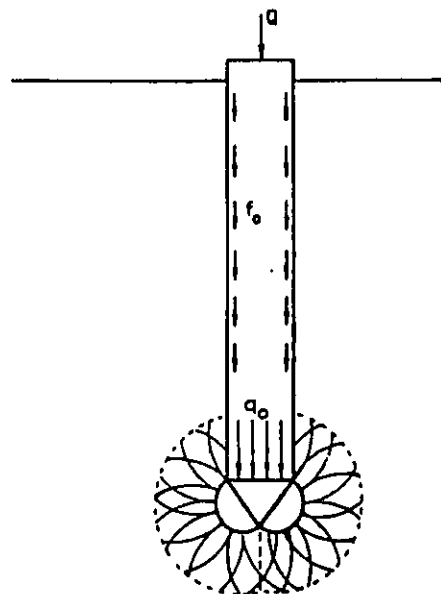
(b)

De Beer (1945)
Jáky (1948)
Meyerhof (1951)



(c)

Berezantsev &
Yaroshenko (1962)
Vesić (1963)



(d)

Bishop, Hill &
Mott (1945)
Skempton, Yassin
& Gibson (1953)

Fig. 7.1 Different assumed failure patterns under deep foundations
(after Vesic, 1967)

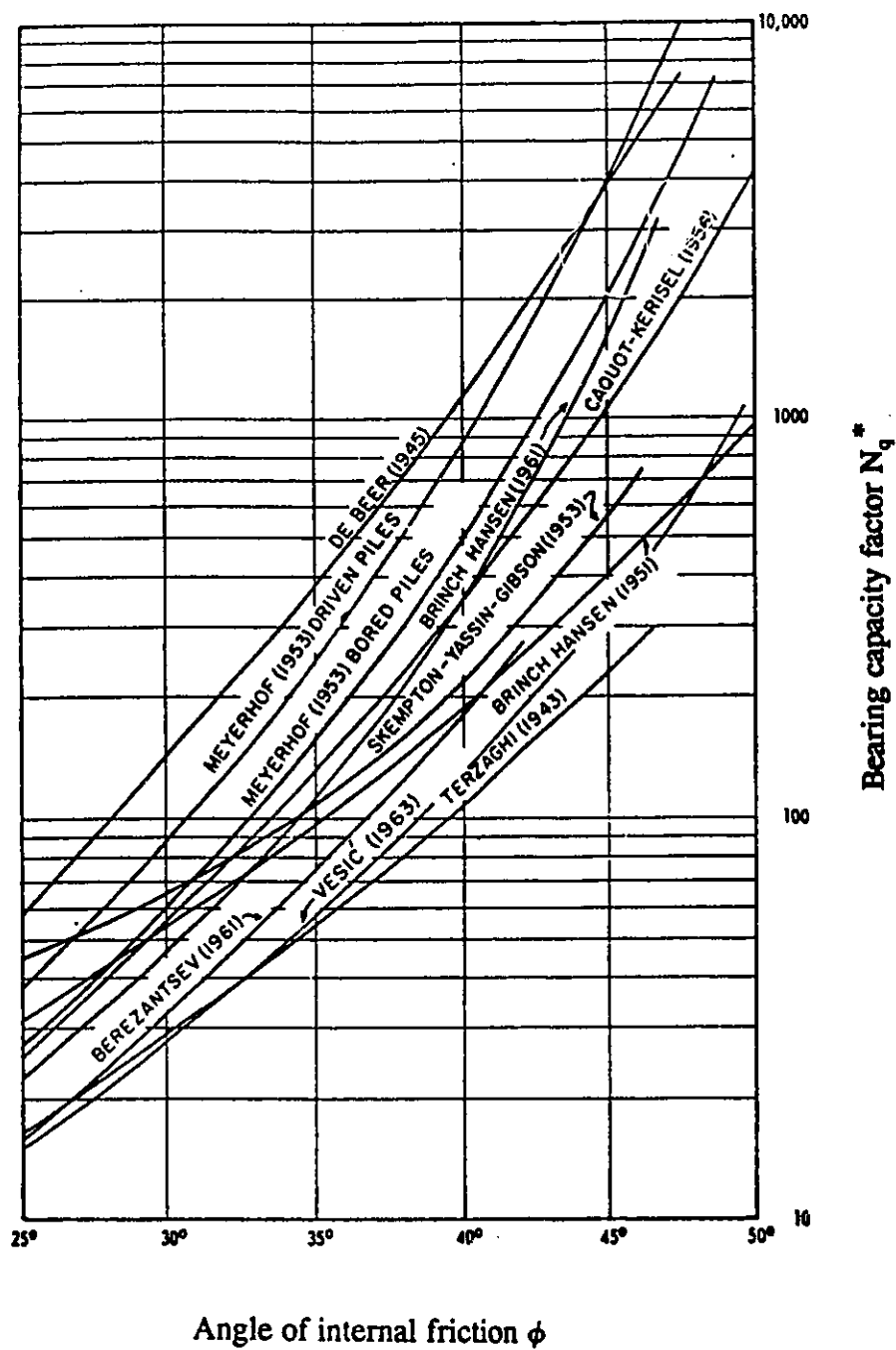


Fig. 7.2 Bearing capacity factors for circular deep foundations
(after Vesic, 1977)

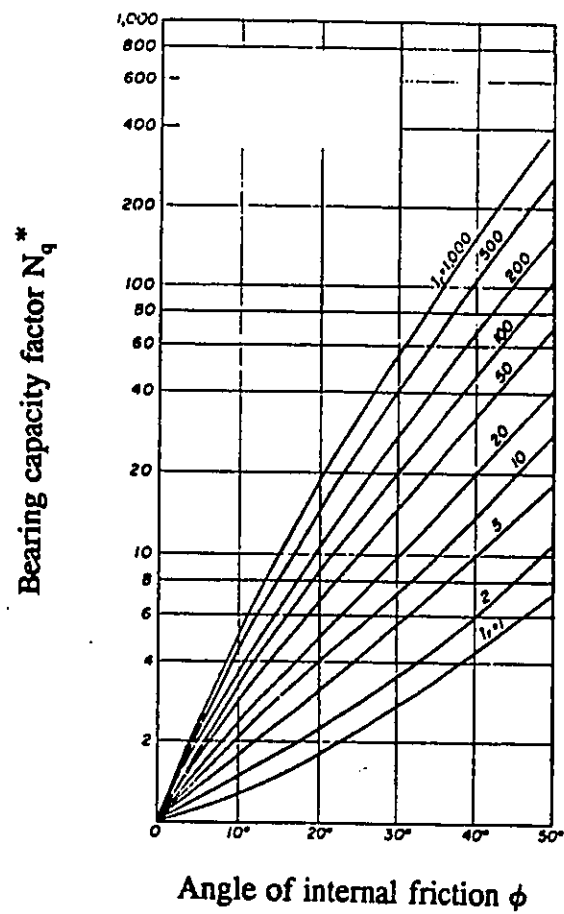


Fig. 7.3 Bearing capacity factor N_q^* for a deep foundation in compressible solid (after Vesic, 1977)

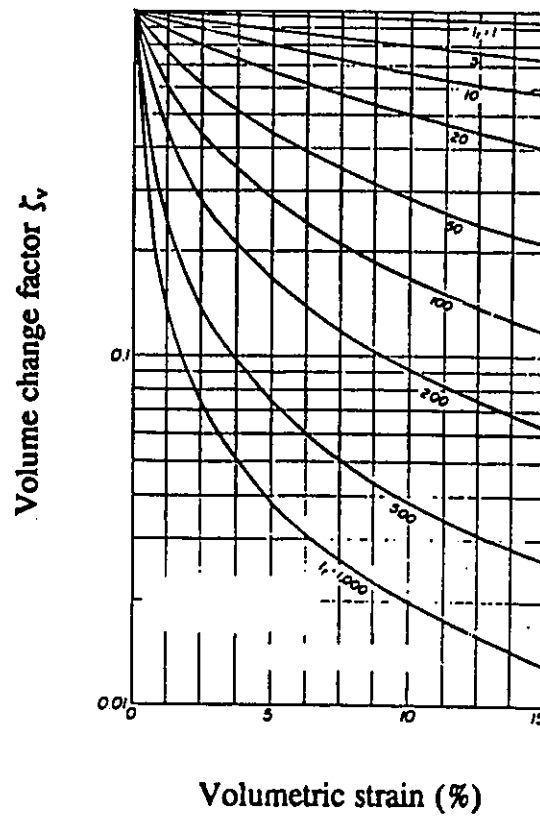


Fig. 7.4 Volume change factors for expansion of a spherical cavity (after Vesic, 1977)

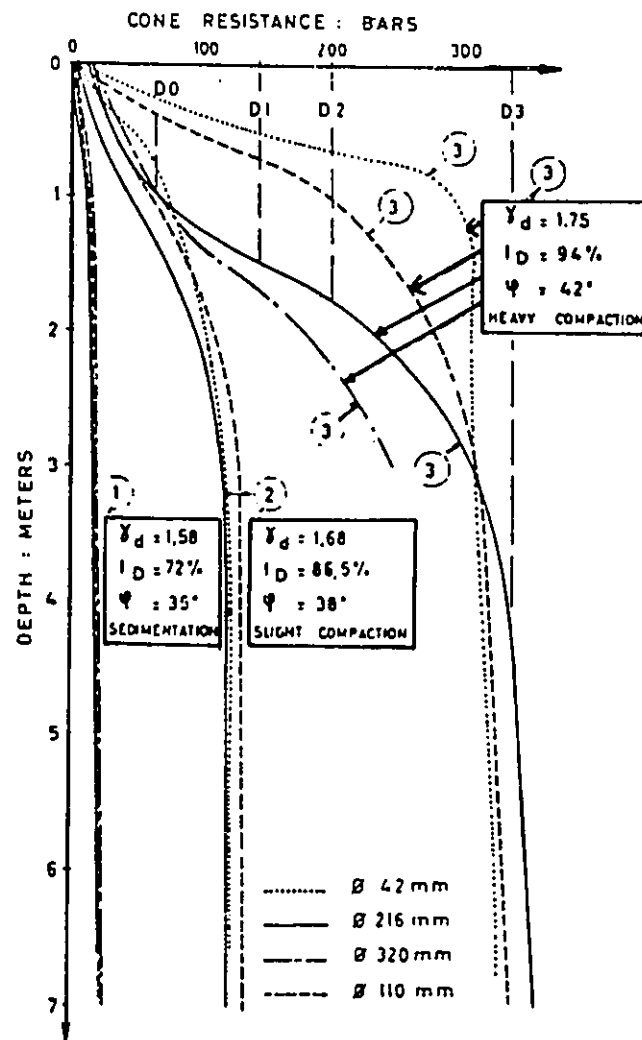


Fig. 7.5 Variation of penetrometer point stress with depth
(after Kerisel, 1964)

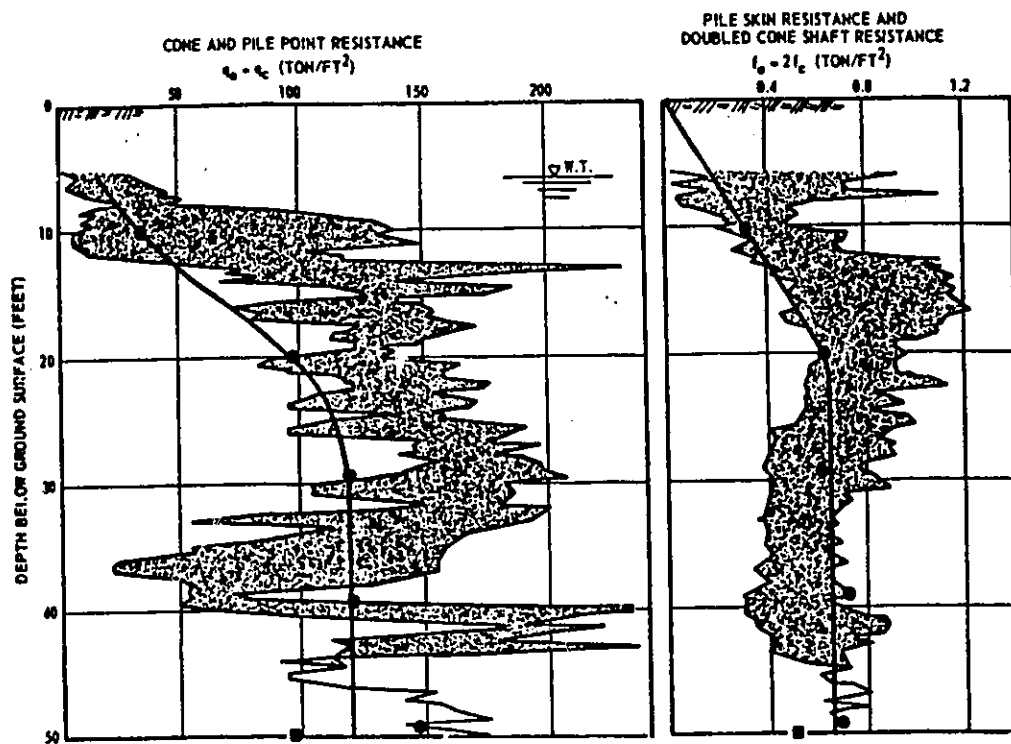


Fig. 7.6 Variation of toe and shaft resistances with depth
(after Vesic, 1970)

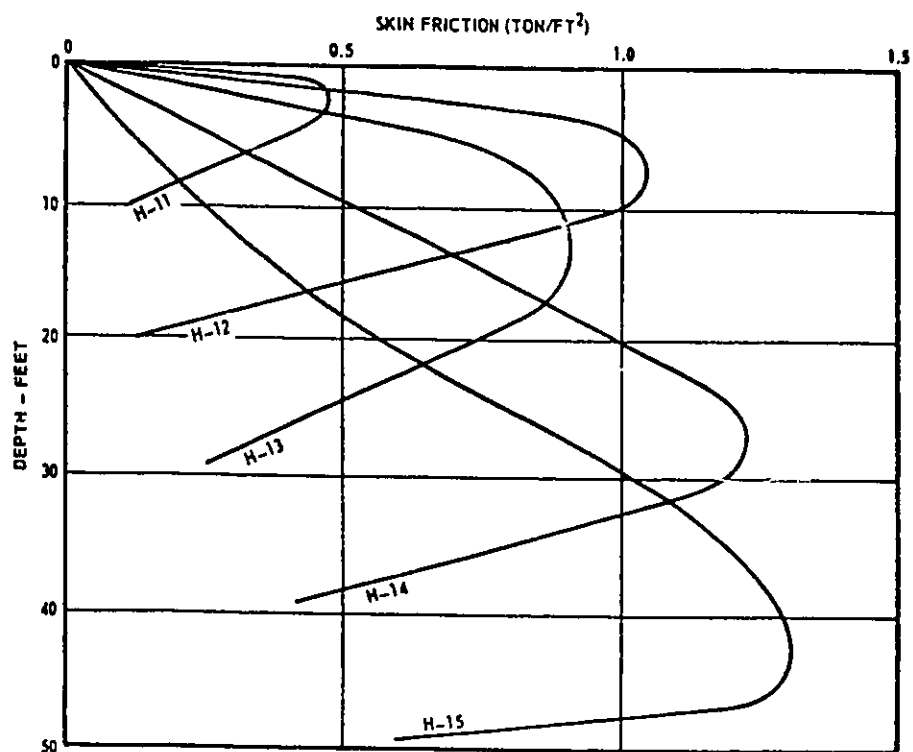


Fig. 7.7 Distribution of shaft resistance along pile shafts
(after Vesic, 1970)

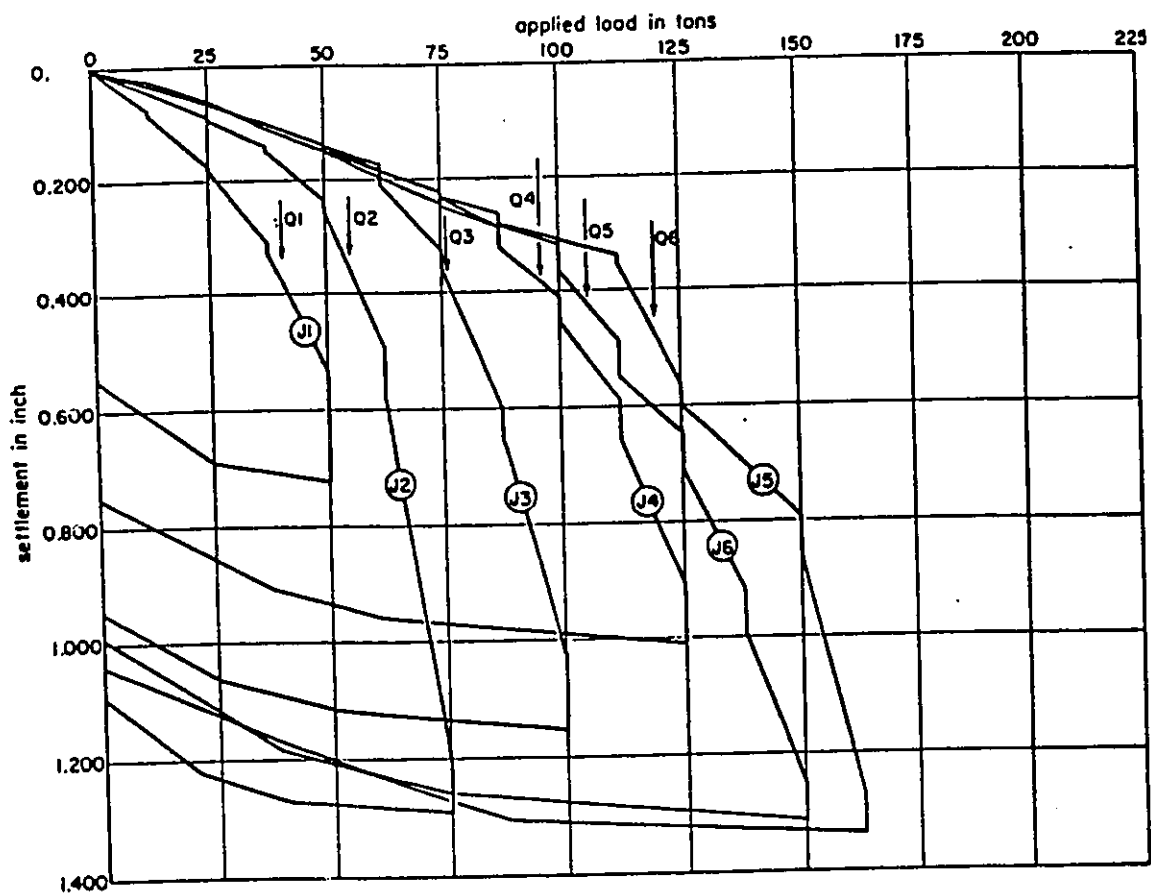


Fig. 7.8 Load-movement curves for precast concrete pile
(after Tavenas, 1971)

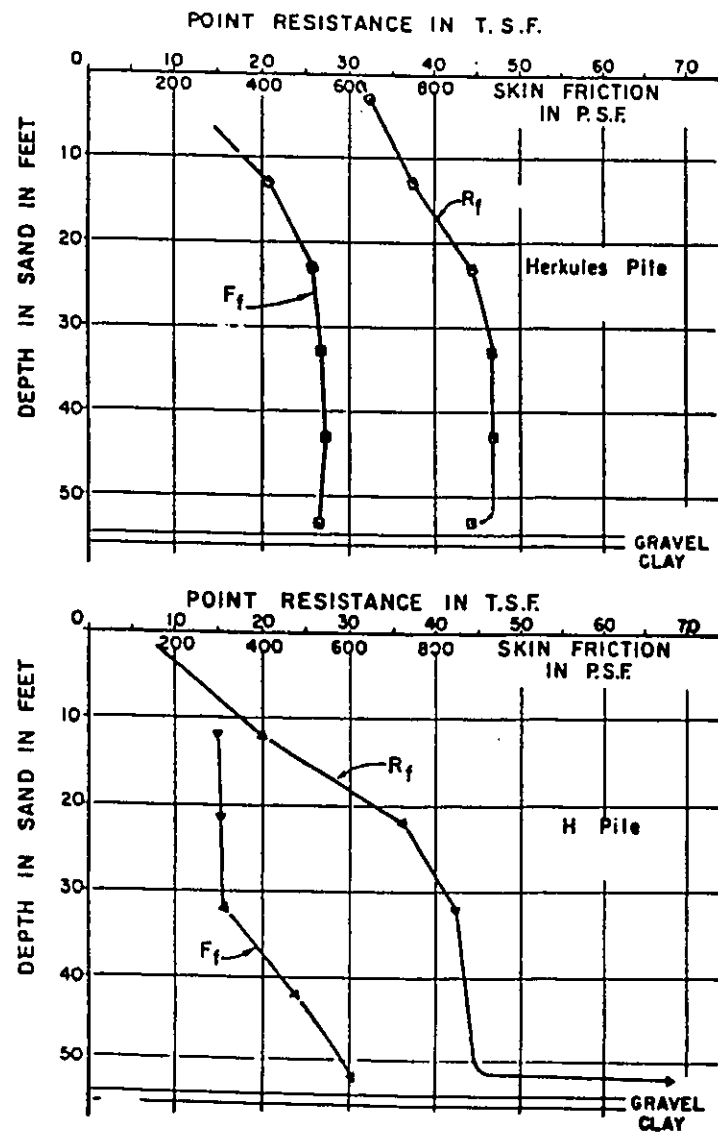


Fig. 7.9 Variation of the ultimate toe resistance and shaft resistance with depth for precast concrete and H-piles (after Tavenas, 1971)

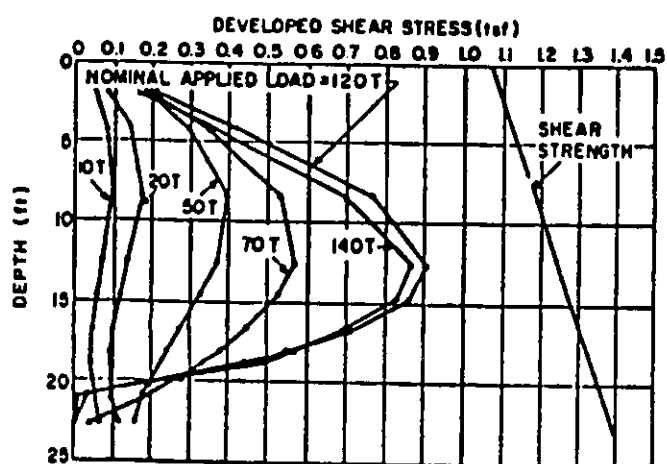


Fig. 7.10 Distribution of developed shaft resistance for Test S1
(after Reese et al., 1976)

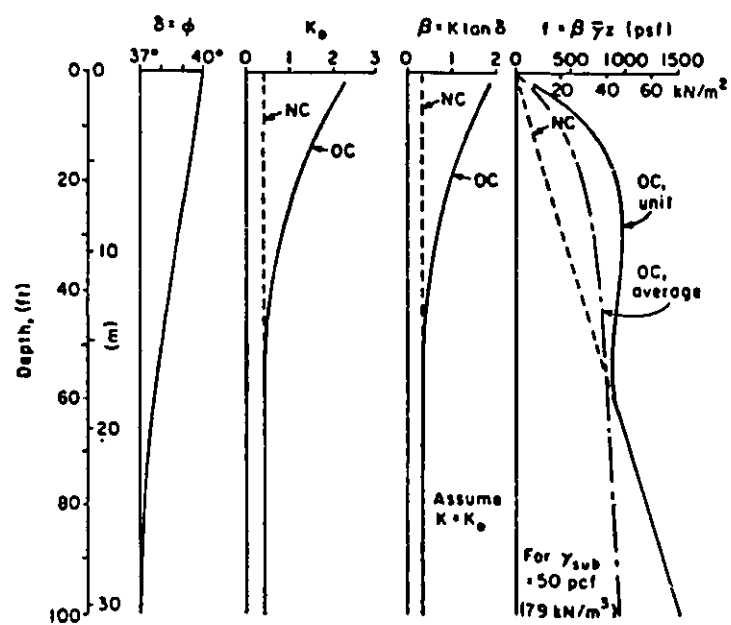


Fig. 7.11 Example illustrating principles of shaft resistance in sand
(after Kulhawy, 1984)

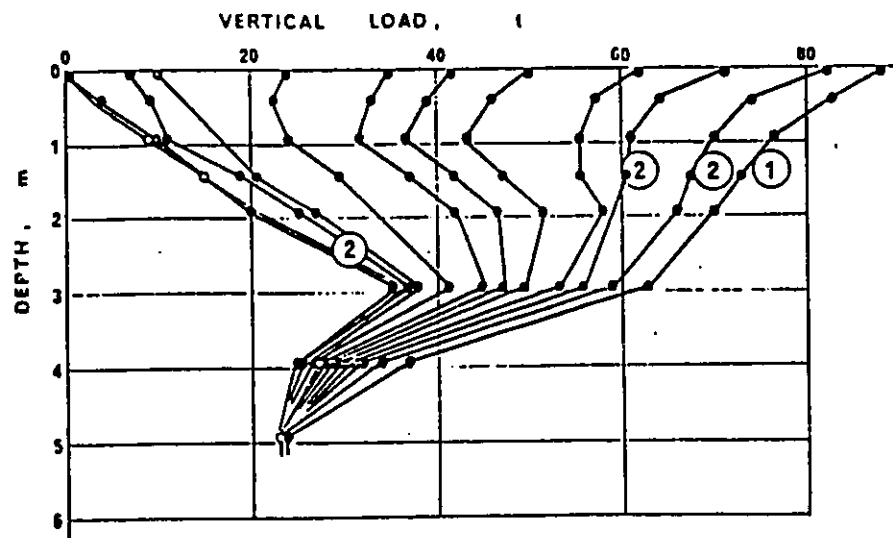


Fig. 7.12 Vertical loads transmitted at several levels in caisson steel pile jacked into hard clay
(after Kerisel and Adam, 1969)

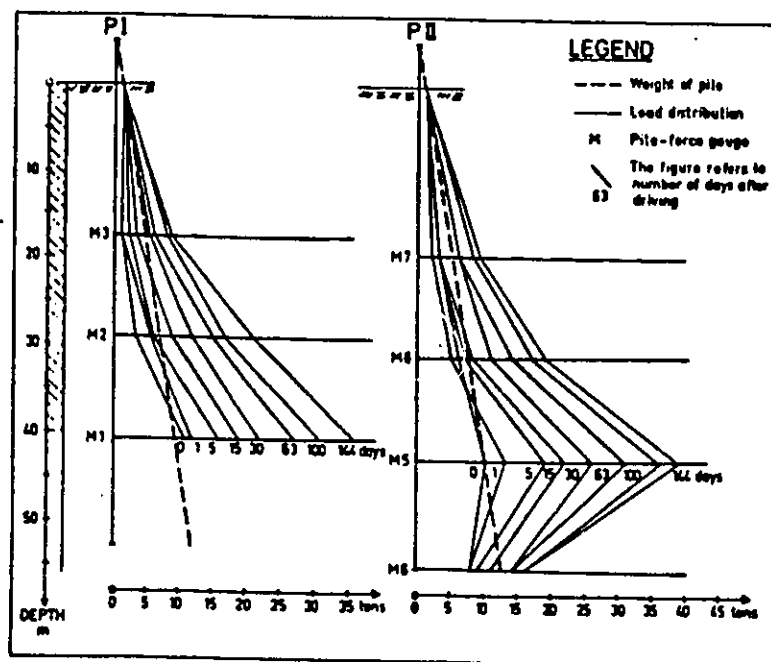


Fig. 7.13 Vertical distribution of load in the piles at different times after driving
(after Fellenius, 1972)

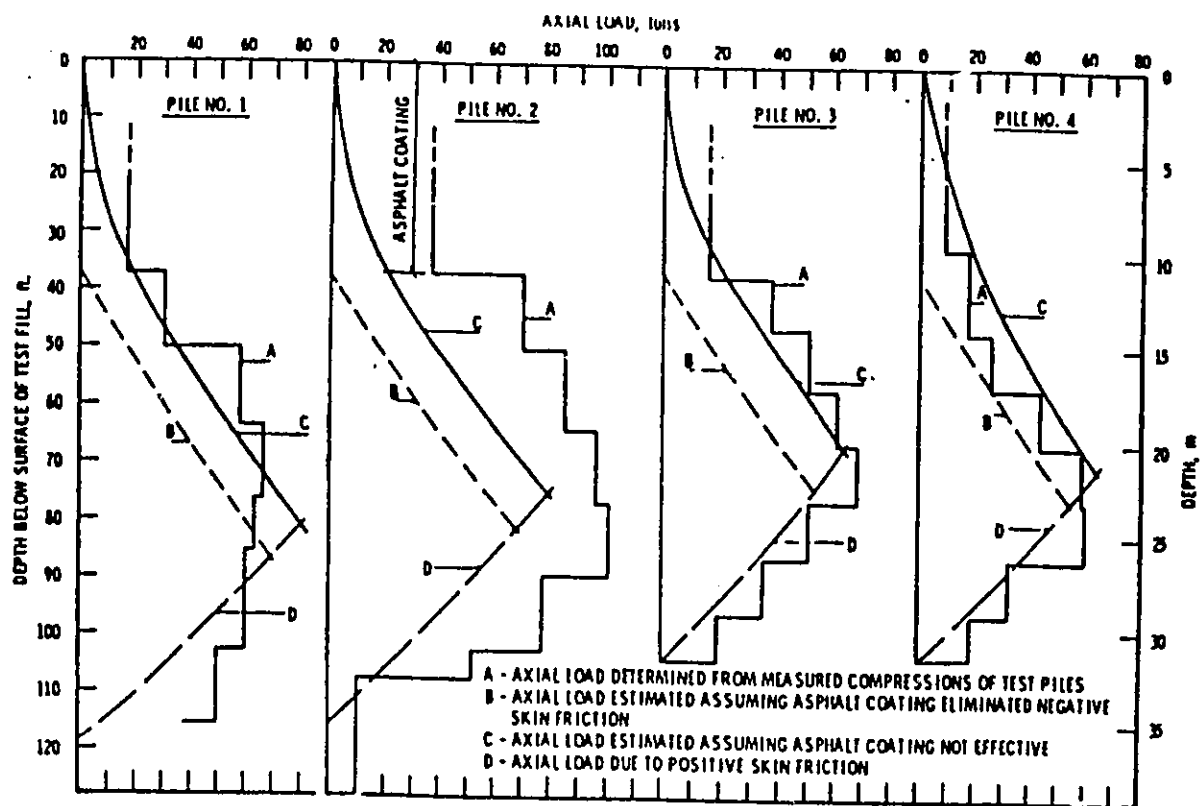


Fig. 7.14 Distribution of axial load in test piles
 (after Bozozuk et al., 1979)

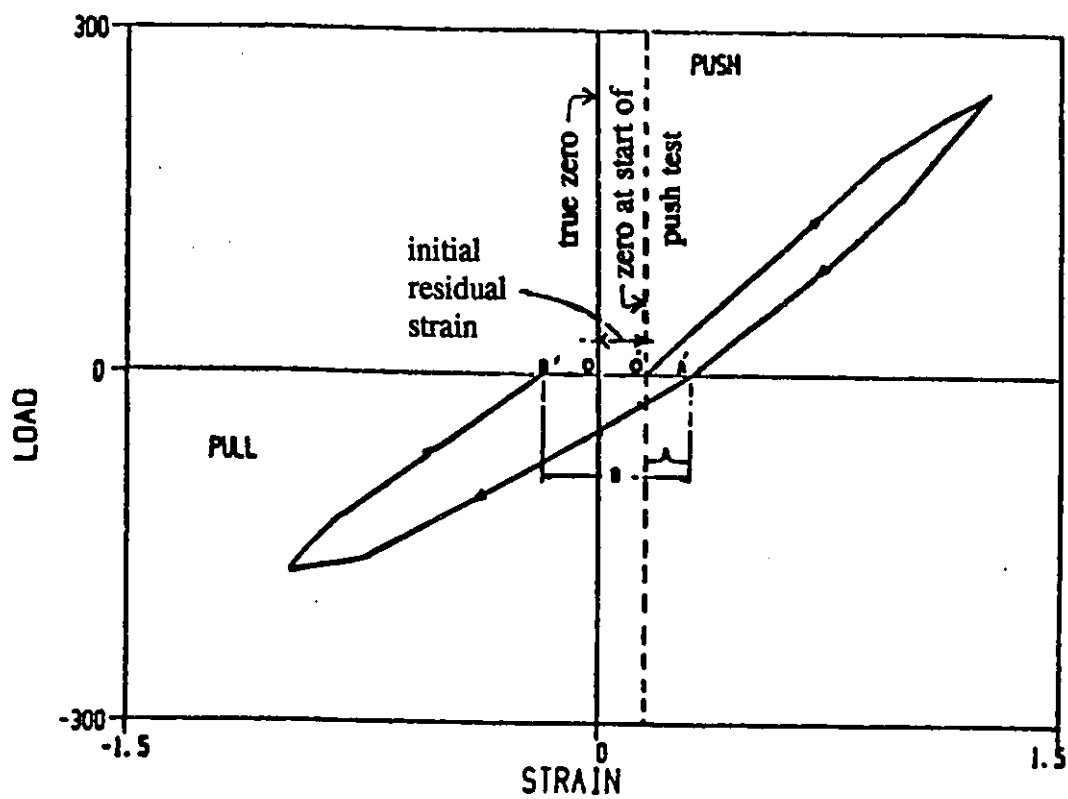


Fig. 7.15 The principle of estimating residual compression by means of comparing compression strain measured from a push test with elongation strain measured in a pull test on the same pile (after Fellenius, 1990)

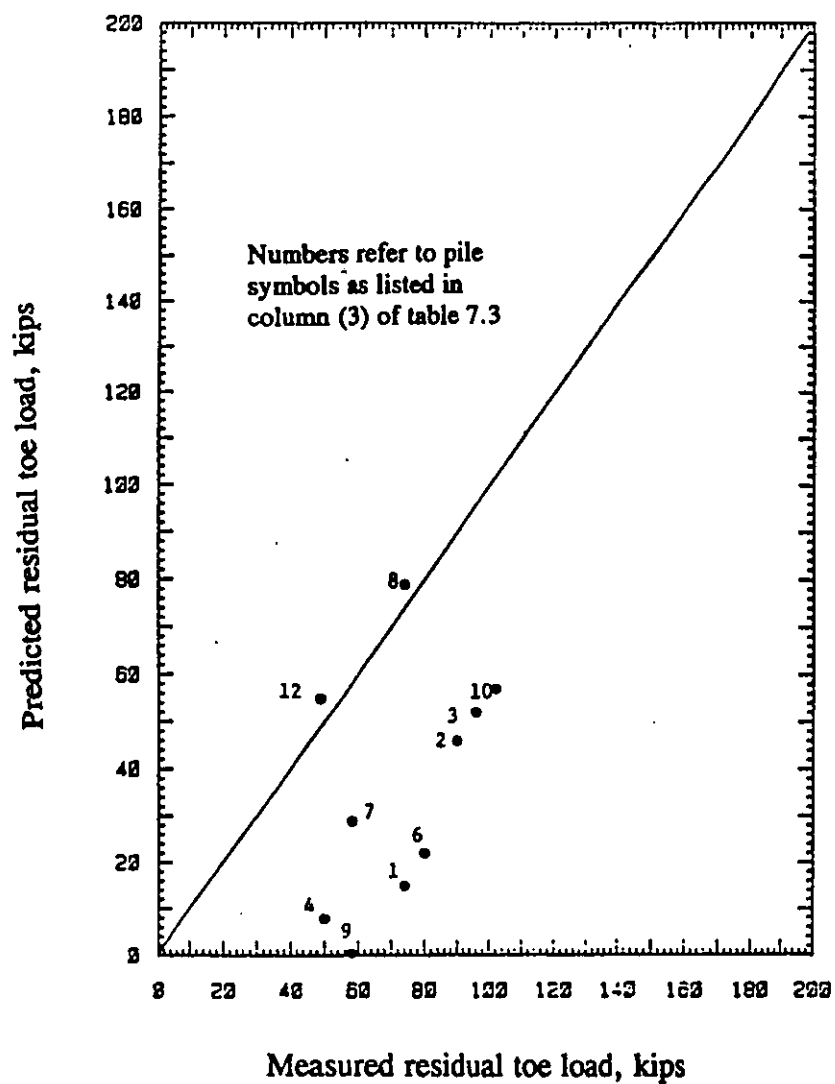


Fig. 7.16 Comparison of predicted versus measured residual toe load using blowcount correlation (after Briaud and Tucker, 1984)

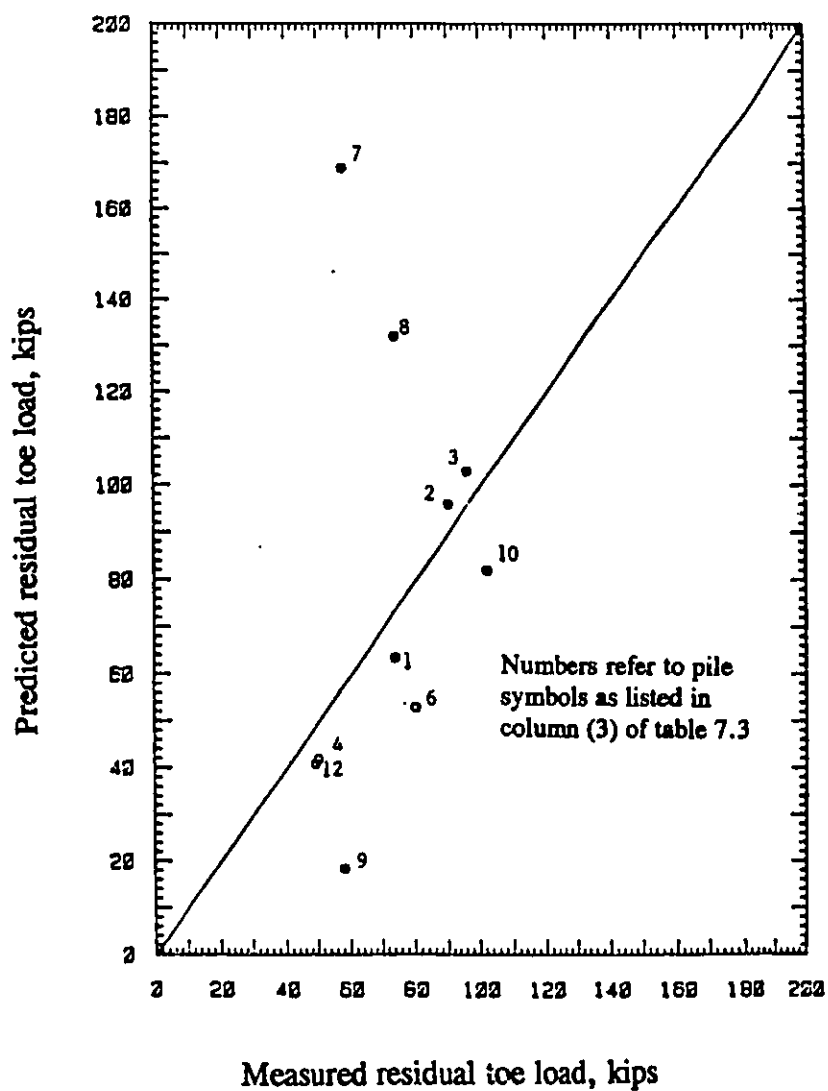


Fig. 7.17 Comparison of predicted versus measured residual toe load using ultimate resistance correlation (after Briaud and Tucker, 1984)

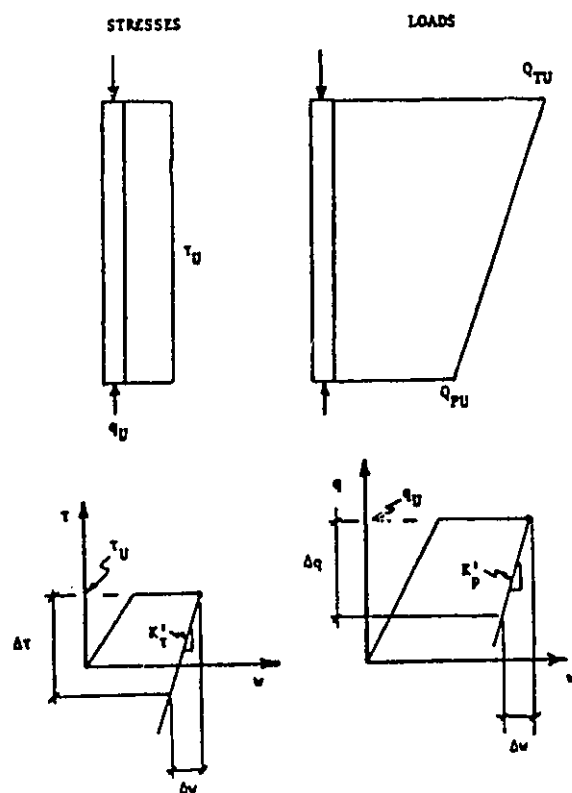


Fig. 7.18 Unloading process for residual loads
(after Briaud et al., 1984)

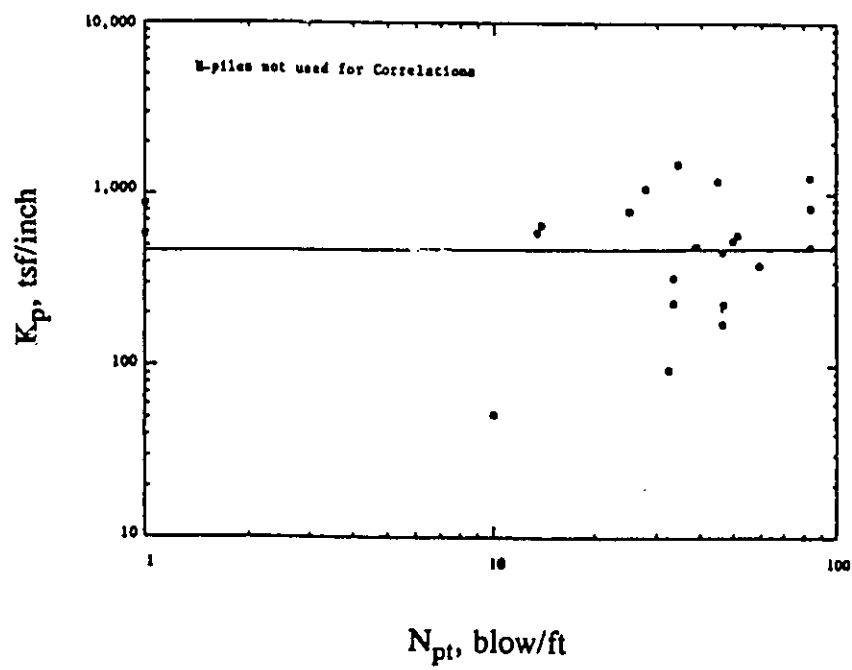


Fig. 7.19 Log (K_p) versus Log (N_{pt})
(after Briaud et al., 1984)

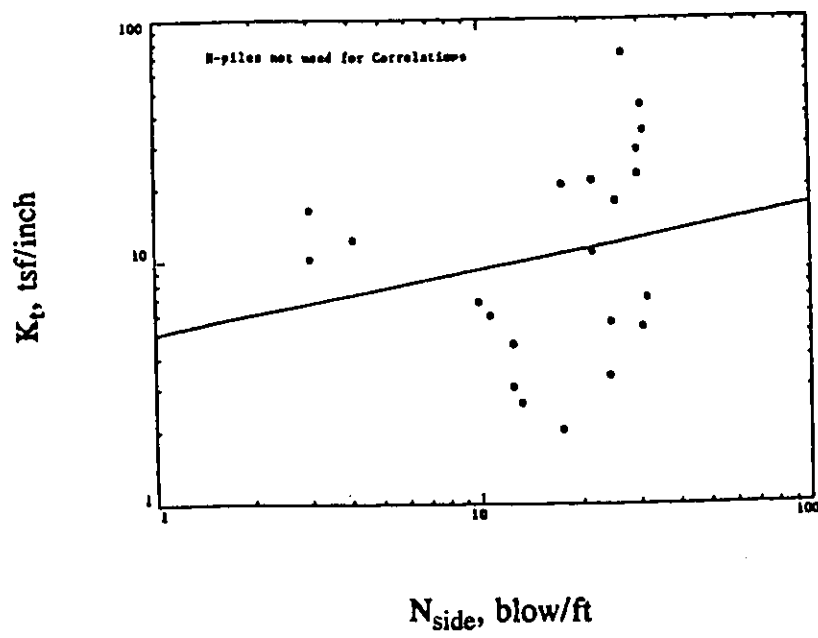


Fig. 7.20 Log (K_t) versus Log (N_{side})
(after Briaud et al., 1984)

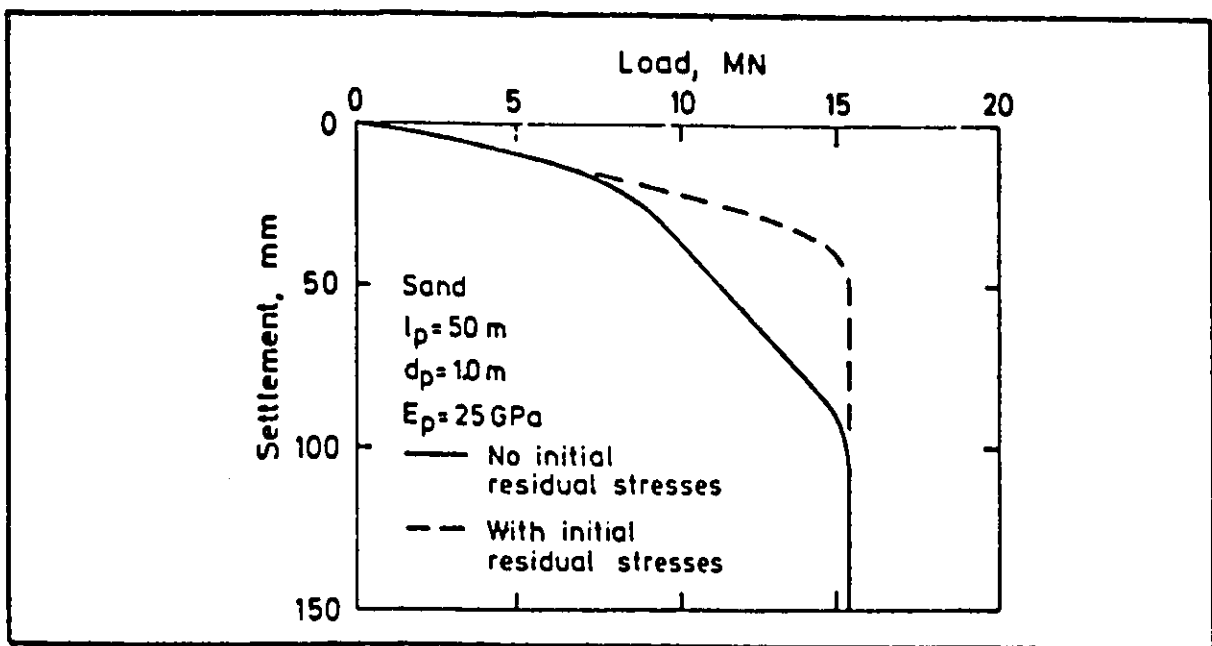


Fig. 7.21 Influence of residual loads on calculated pile head stiffness during compression loading (after Poulos, 1987)

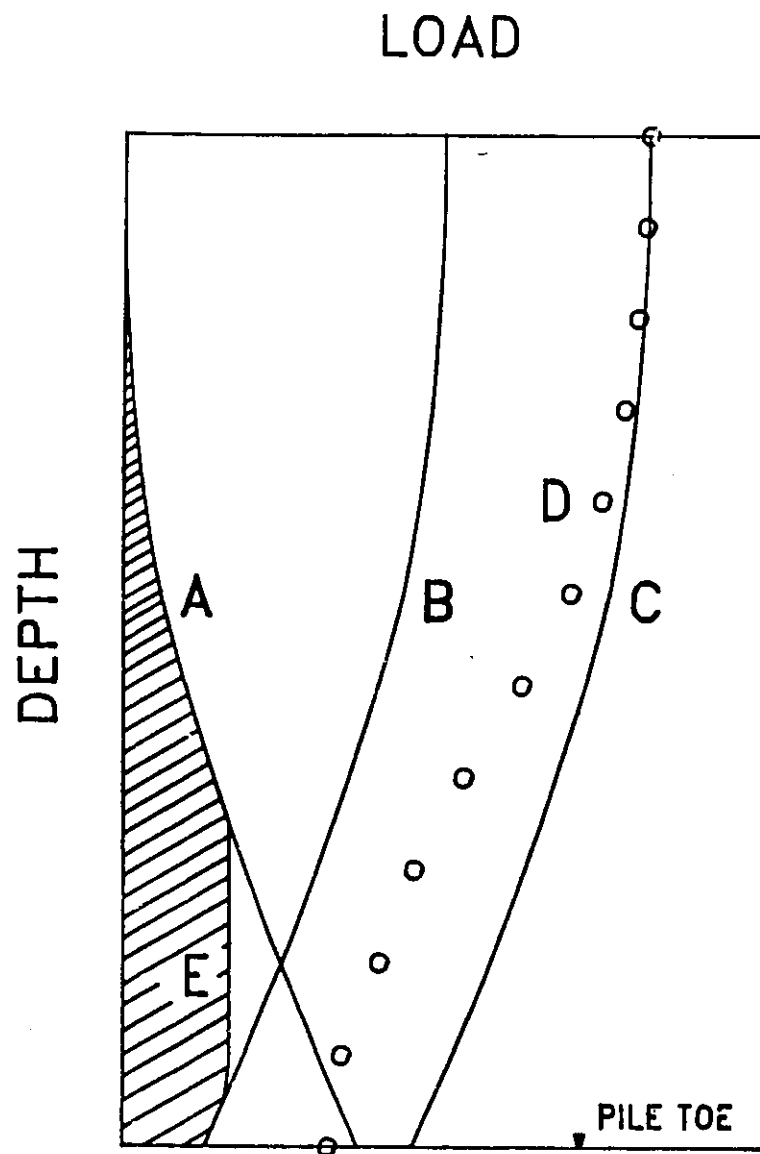


Fig. 7.22 Principle of determining residual load as to distribution and magnitude in relation to loads induced during the static test (after Altzée et al., 1990)

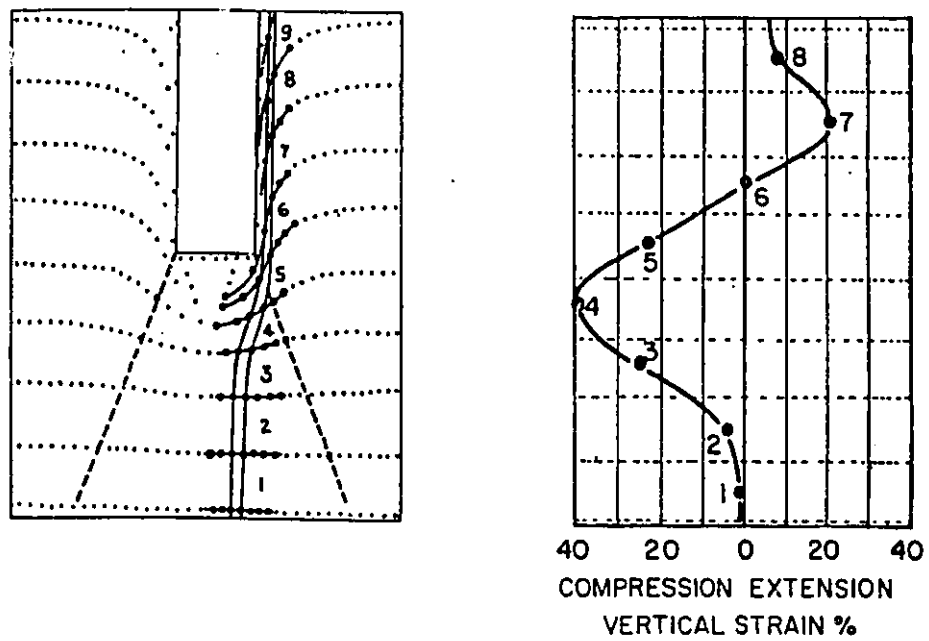


Fig. 7.23 Displacement and strains around a deep foundation in sand
(after Robinsky and Morrison, 1964)

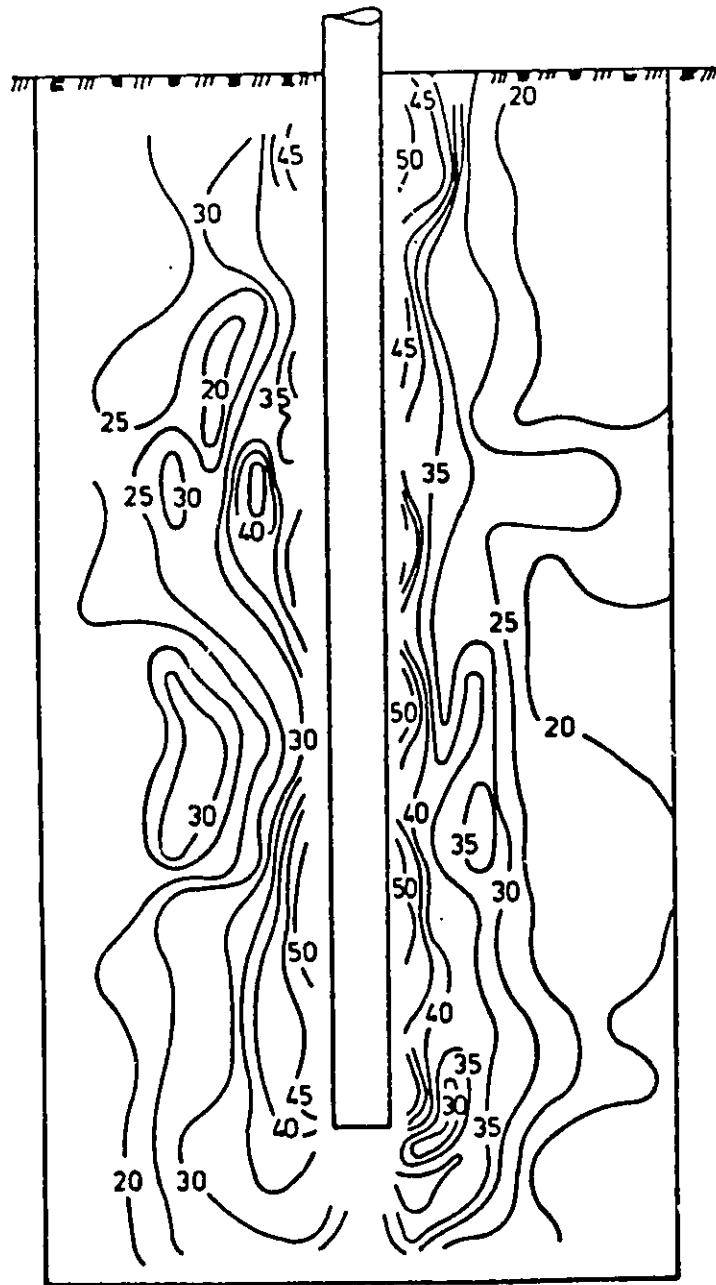


Fig. 7.24 Distribution of relative density after pile driving
(after Robinsky and Morrison, 1964)

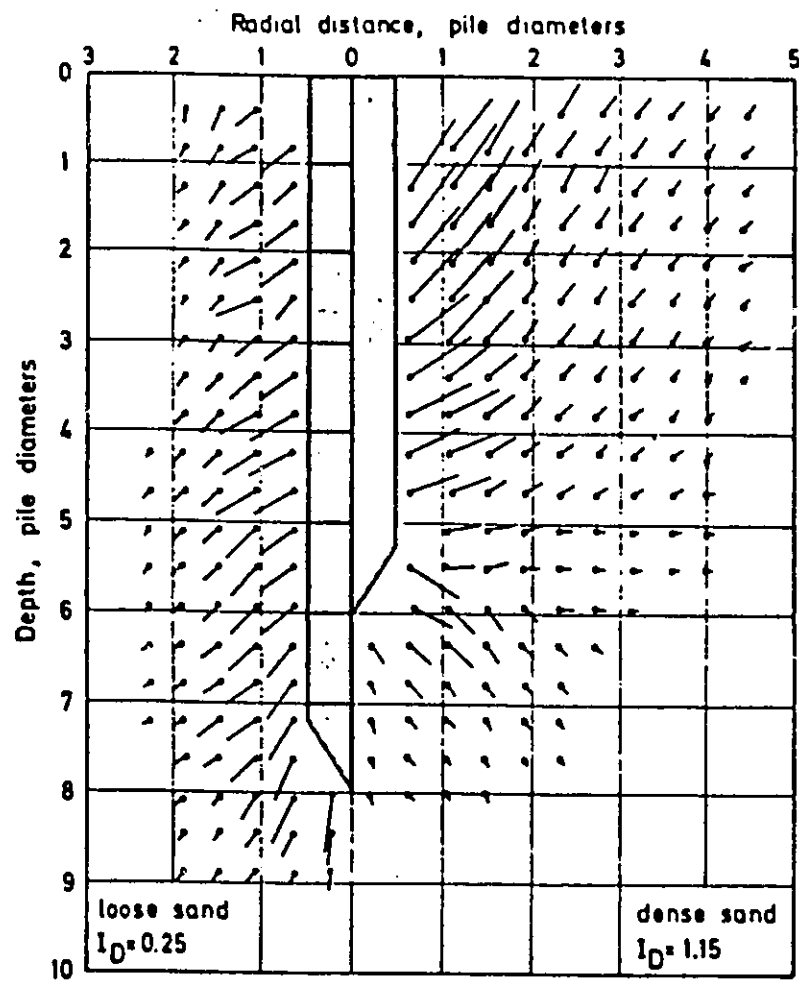


Fig. 7.25 Soil movements due to penetrating cone
(after Davidson et al., 1981)

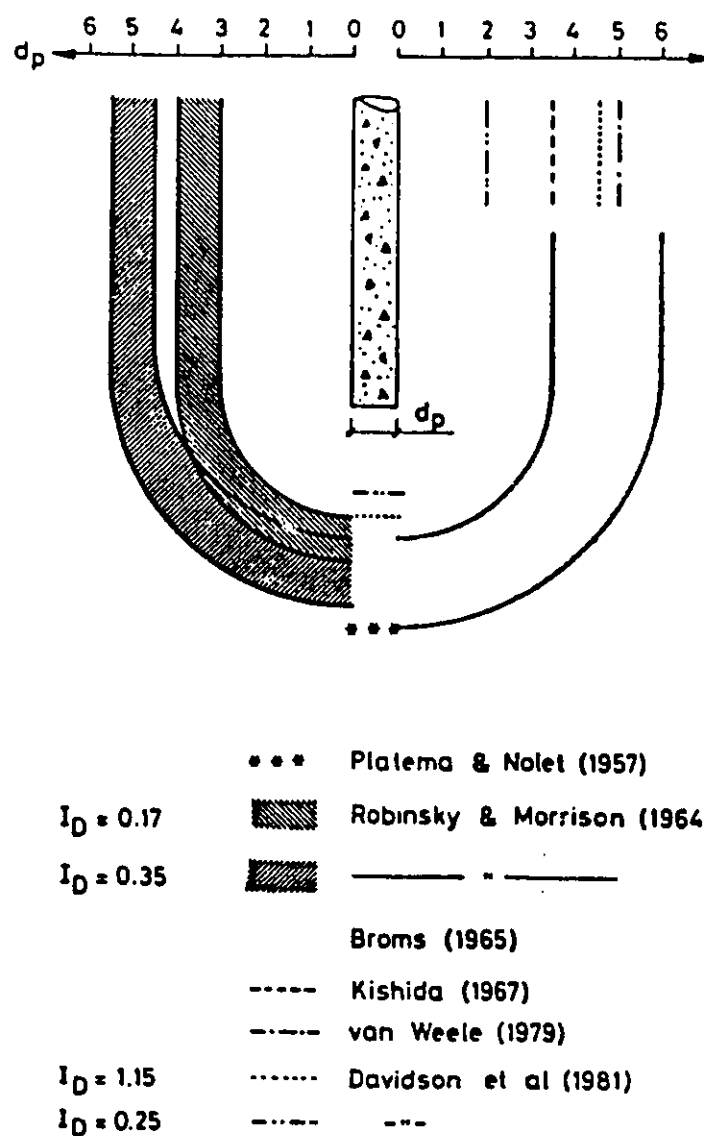


Fig. 7.26 Extension of the zone of soil affected by driving,
 a summary of different investigations
 (after Ekstrom, 1989)

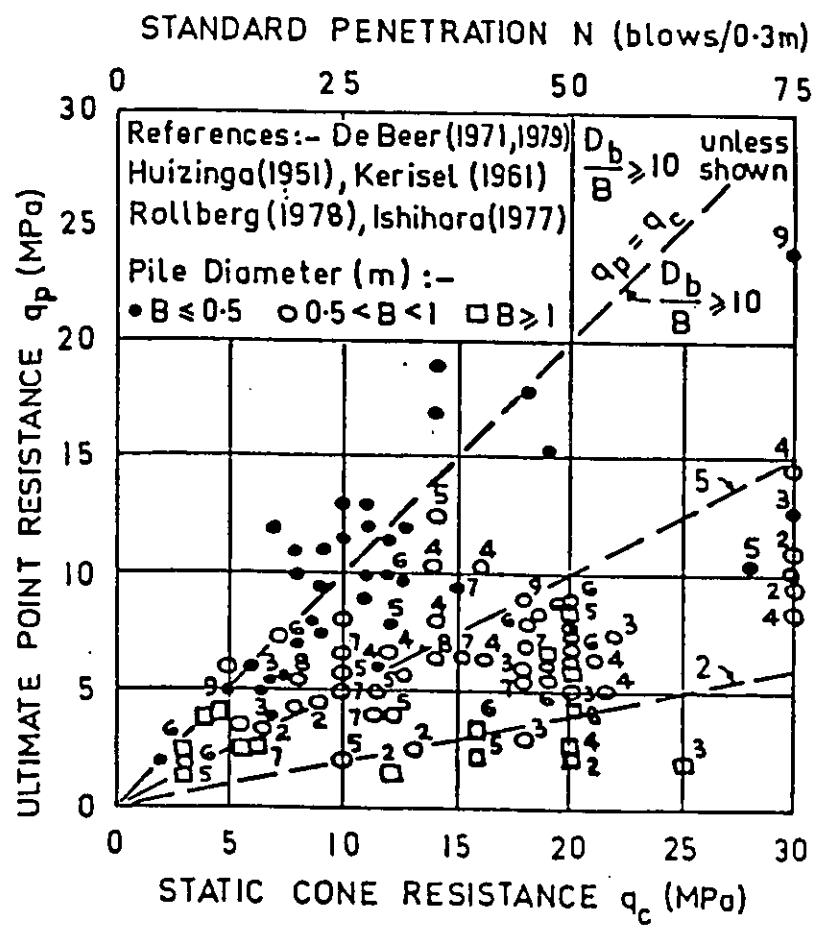


Fig. 7.27 Relation between ultimate toe resistance of driven piles and static cone and Standard Penetration resistance in sand (after Meyerhof, 1983)

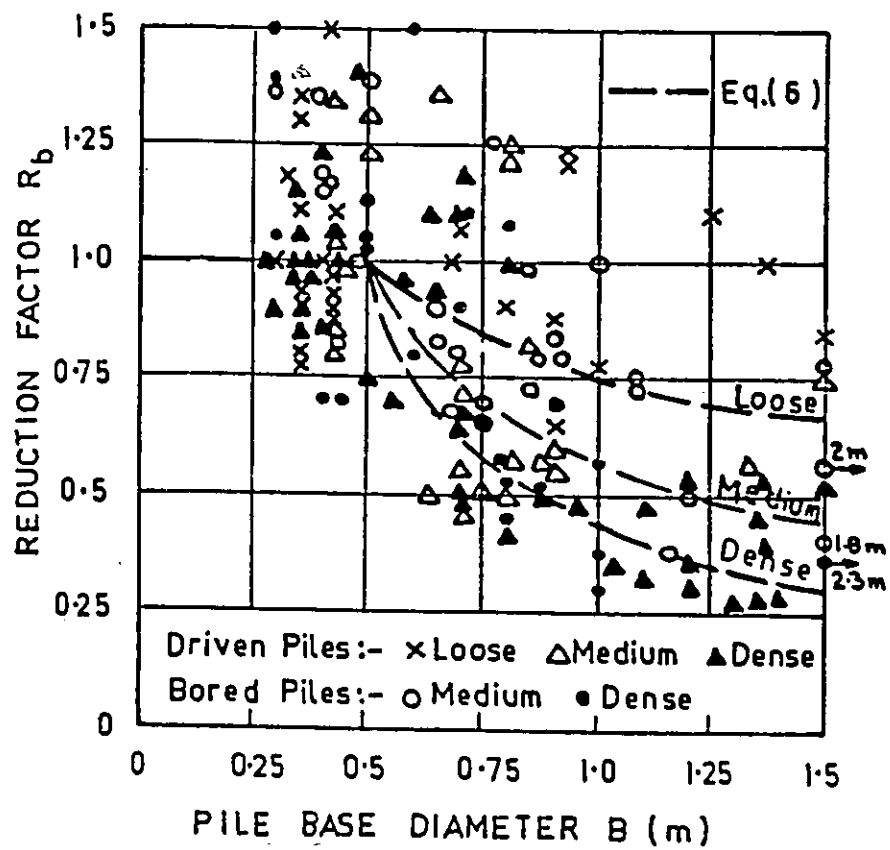


Fig. 7.28 Empirical reduction factor for ultimate toe resistance of large diameter piles in sand (after Meyerhof, 1983)

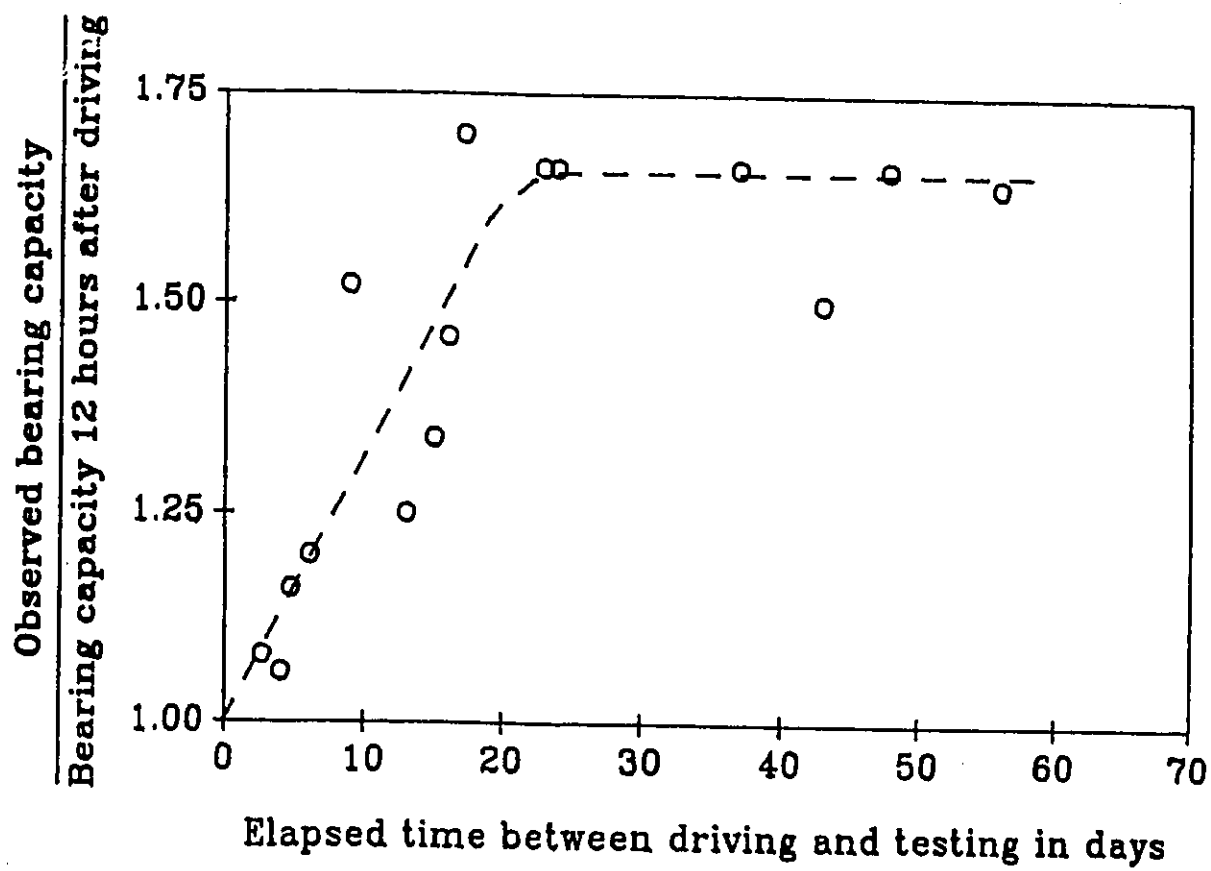


Fig. 7.29 Variation of bearing capacity with time at St-Charles River
(after Tavenas, 1971)

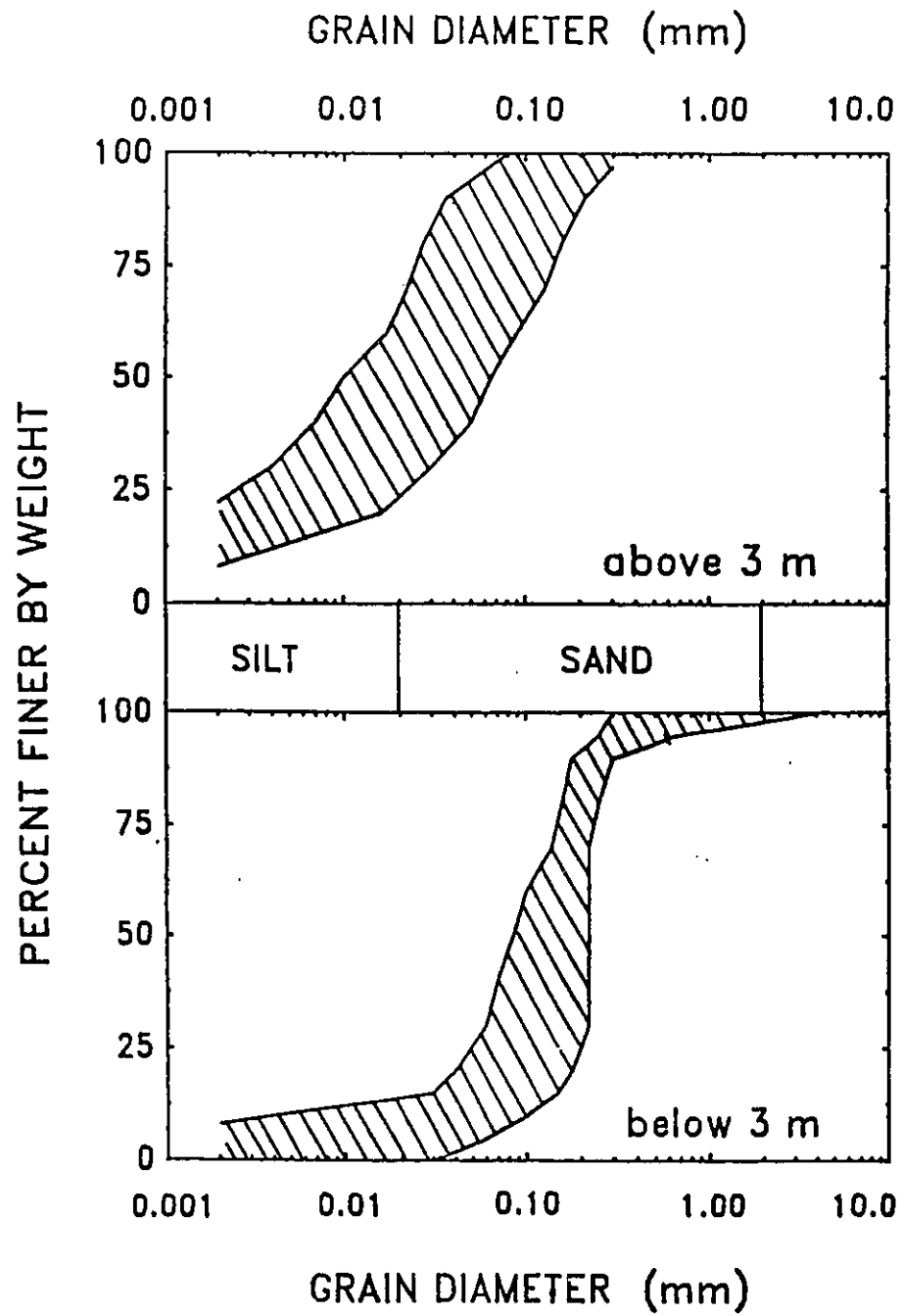


Fig. 8.1 Particle size distribution envelopes of the two soil layers.

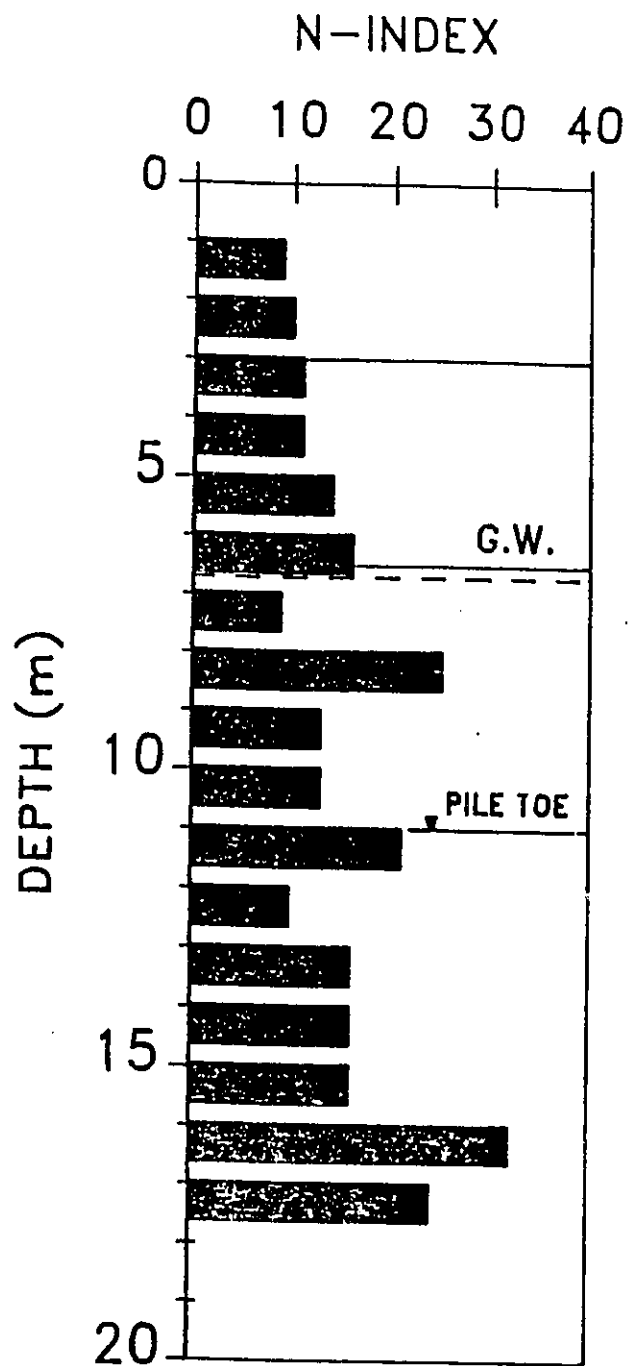


Fig. 8.2 Standard penetration test profile from a borehole located 7.0 m away from the test pile

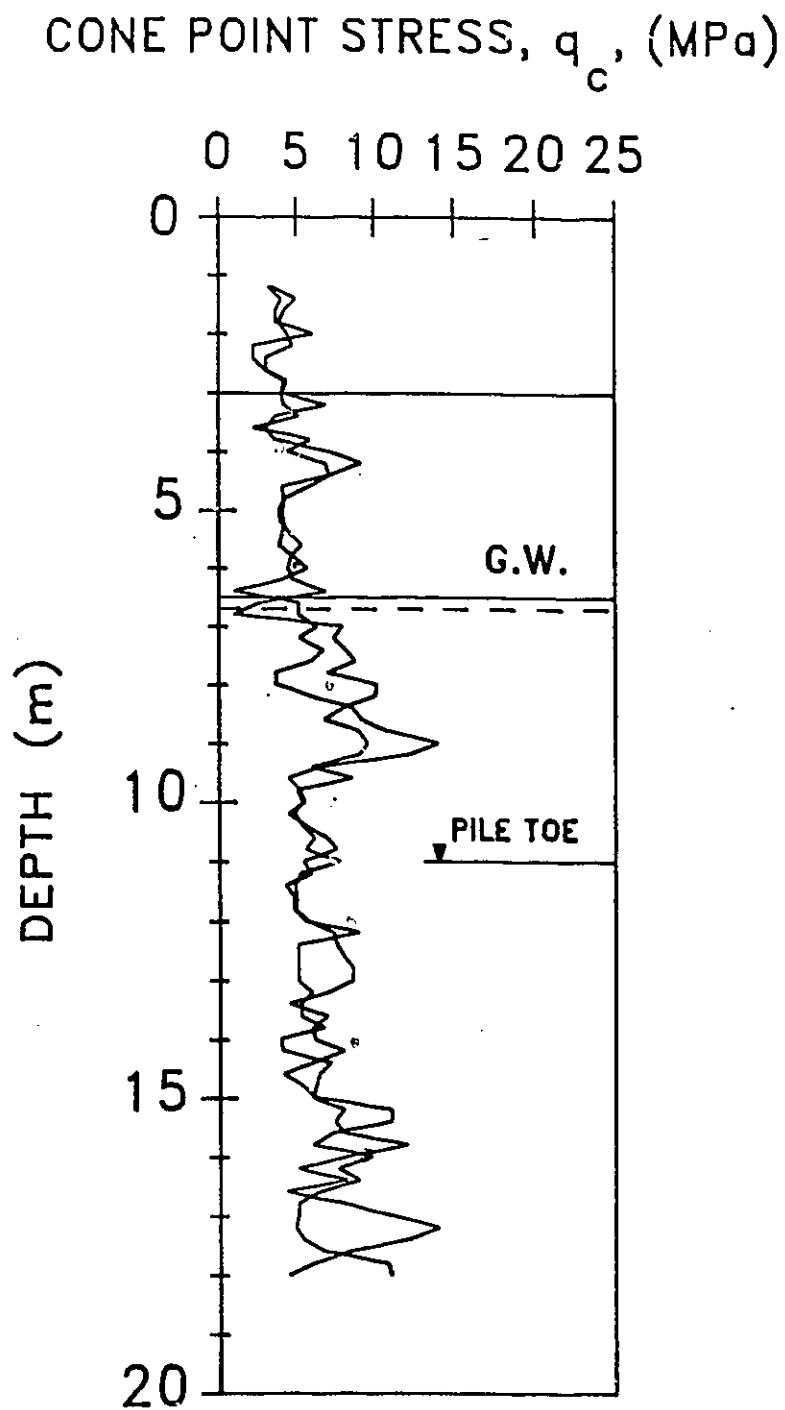


Fig. 8.3 Results from two static cone penetrometer tests 2.5 m away from the test pile
a: Cone point resistance

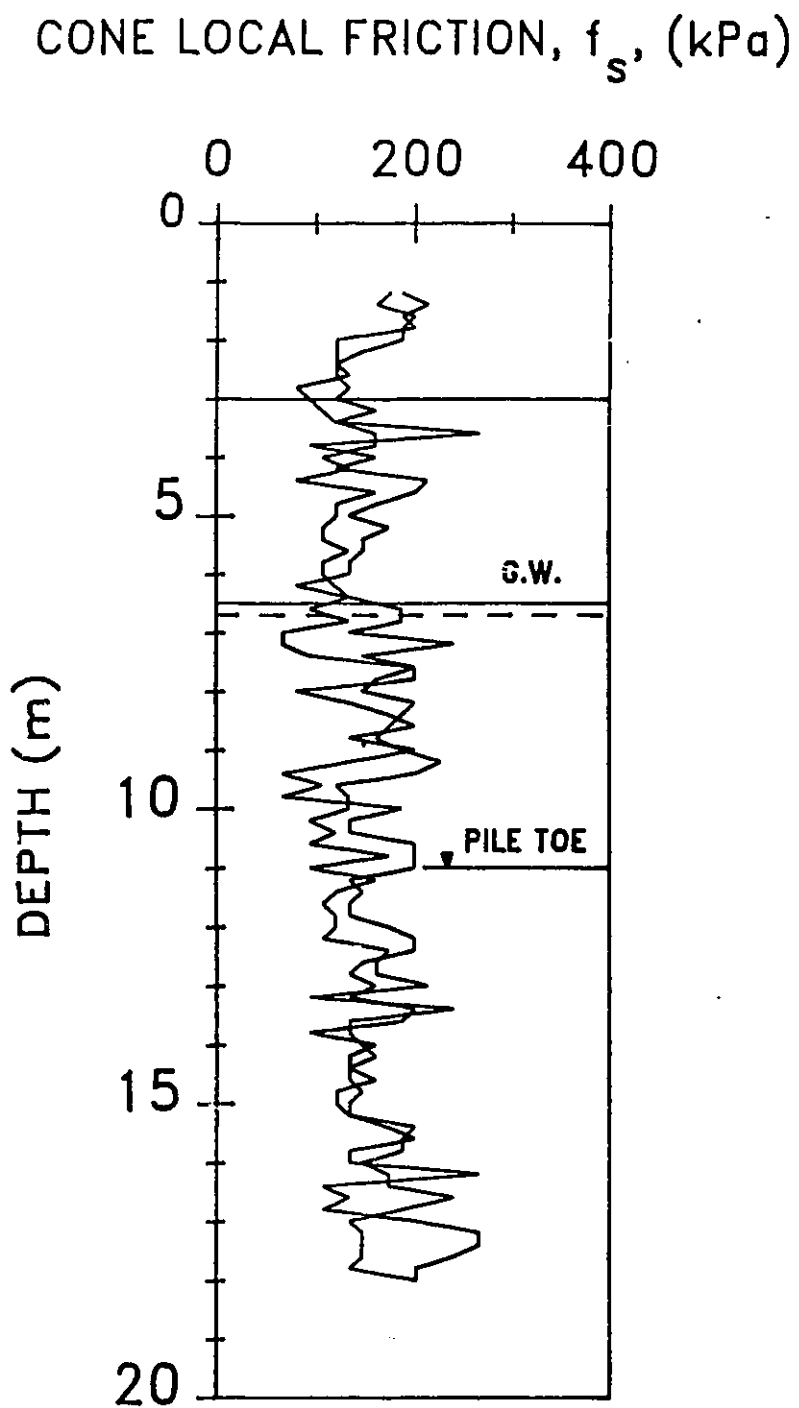


Fig. 8.3 Results from two static cone penetrometer tests 2.5 m away from the test pile
b: Cone local friction

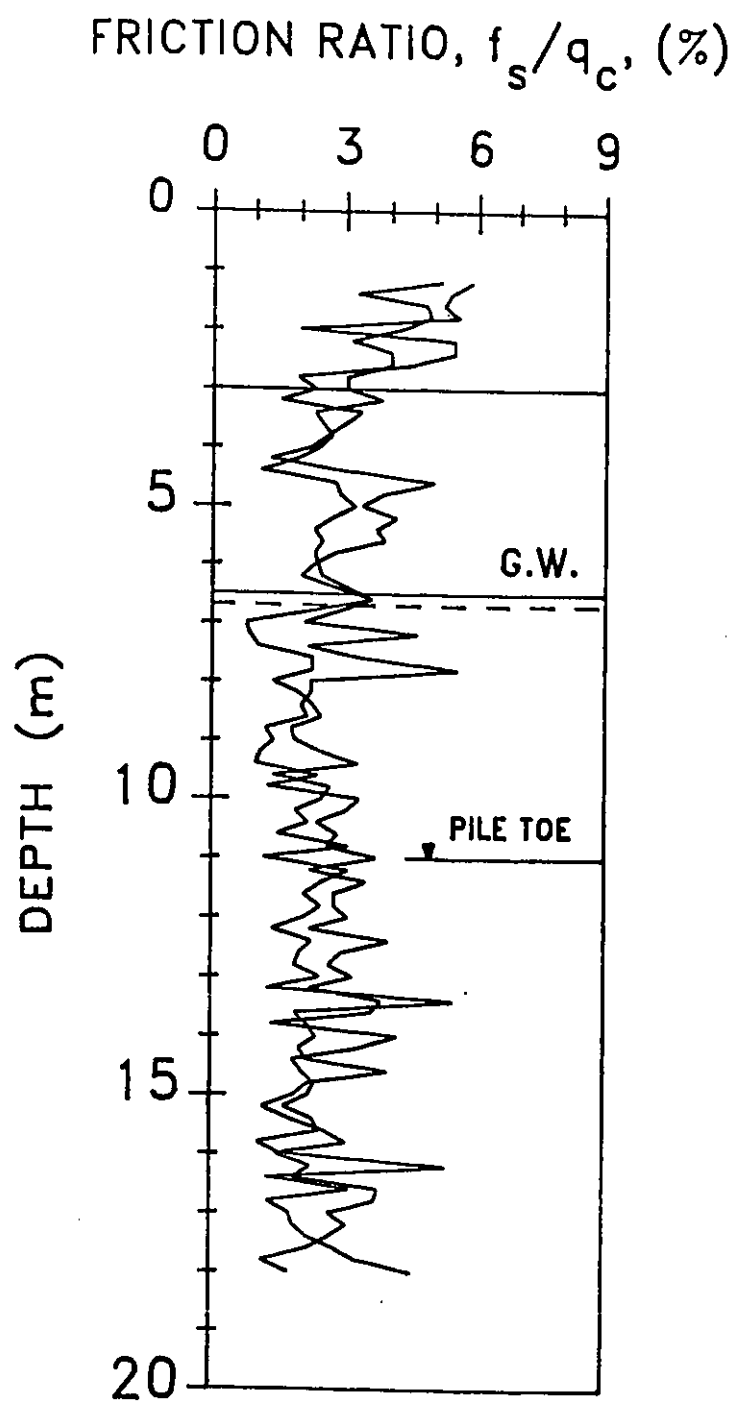


Fig. 8.3 Results from two static cone penetrometer tests 2.5 m away from the test pile
c: Cone friction ratio

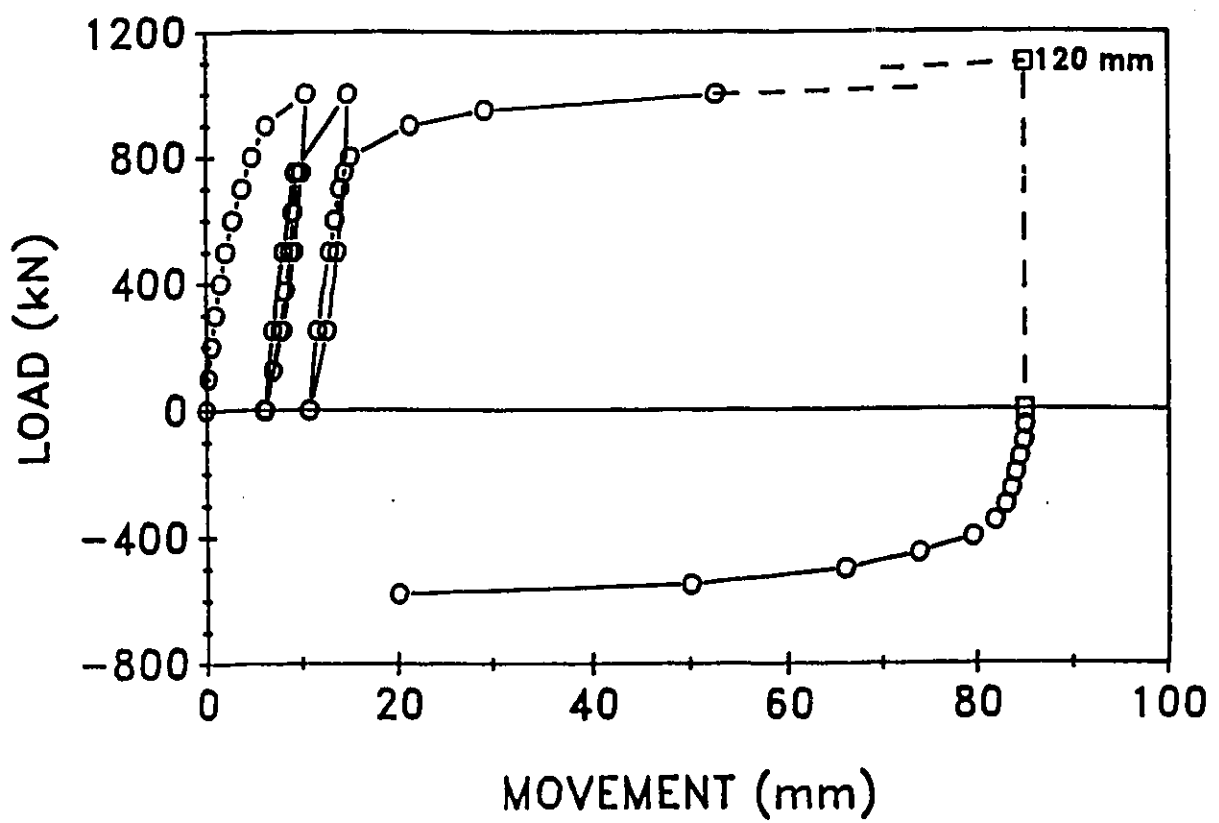


Fig. 8.4 Pile head load-movement diagram

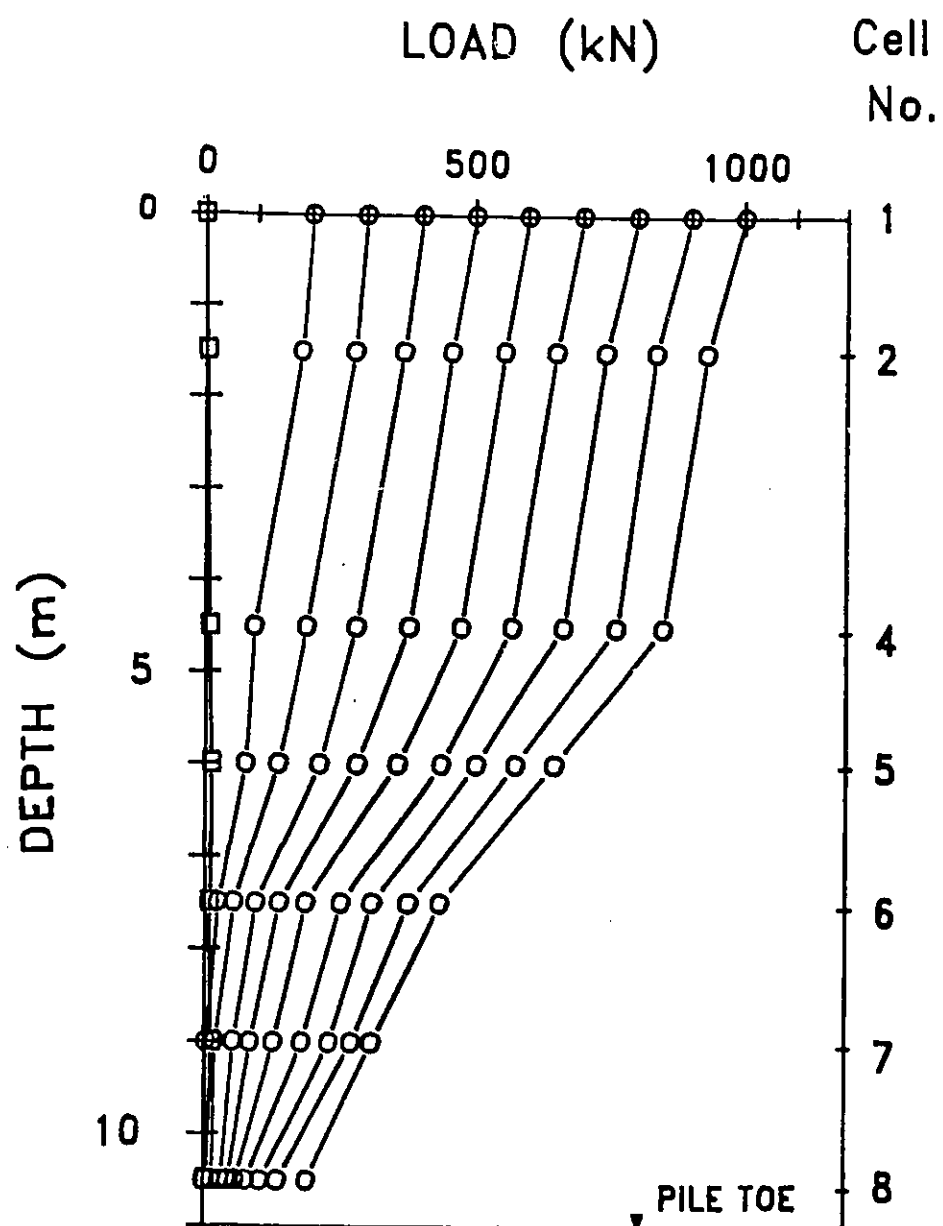


Fig. 8.5 Load distribution in the pile as measured in Test I
(square symbol is measured load remaining after full unloading)

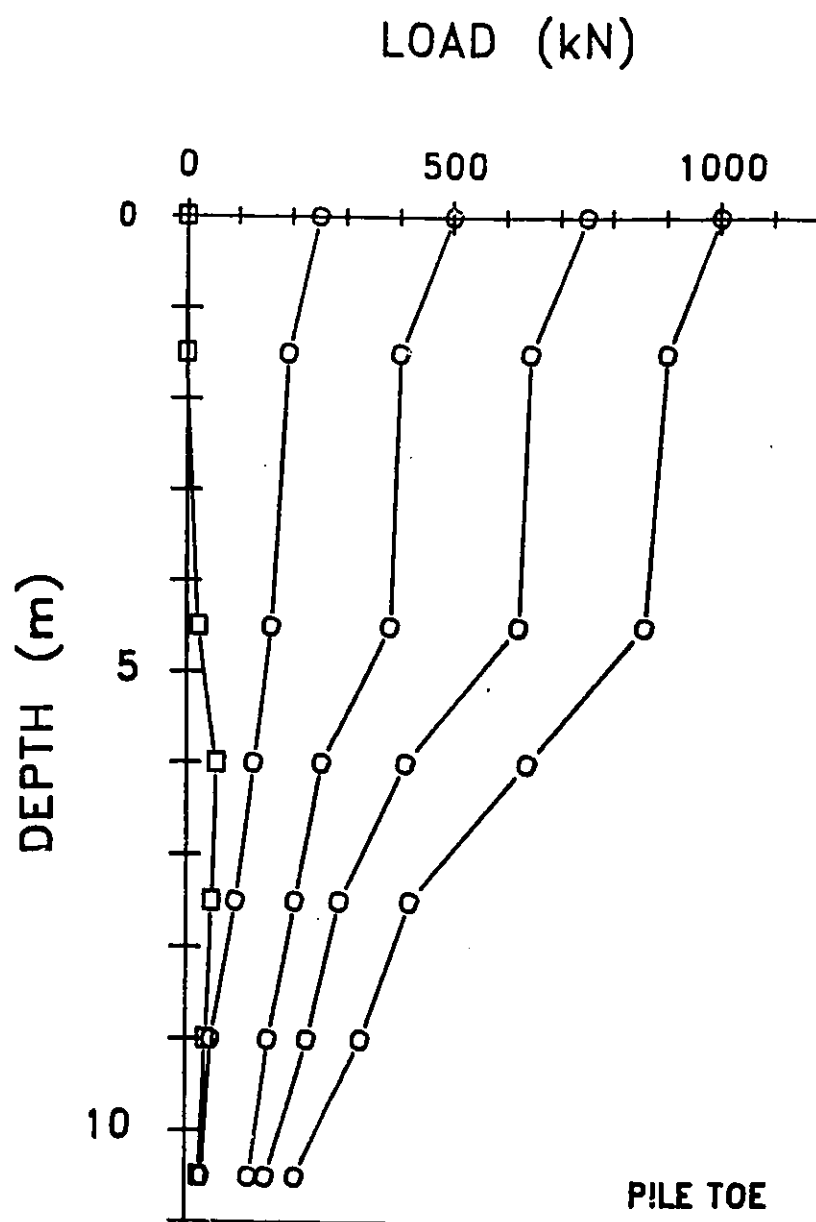


Fig. 8.6 Load distribution in the pile as measured in Test II (square symbol is measured load remaining after full unloading)

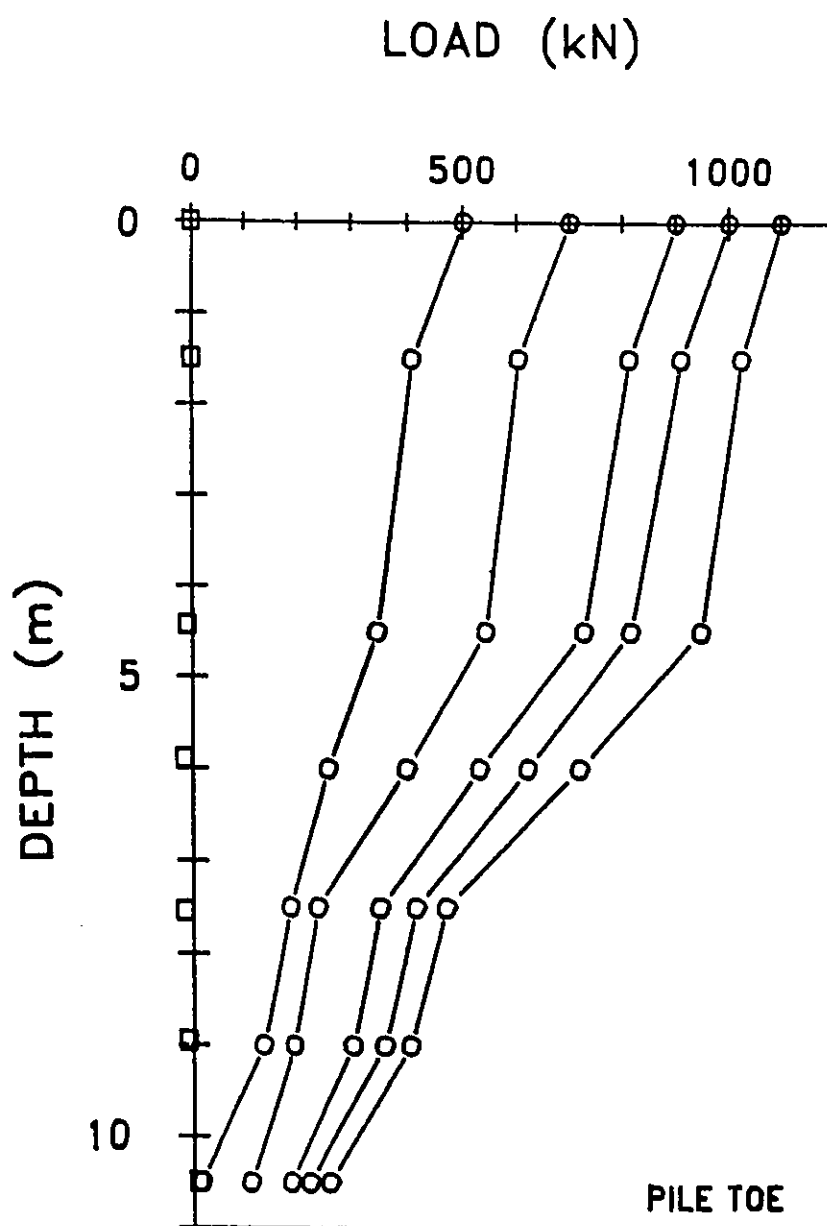


Fig. 8.7 Load distribution in the pile as measured in Test III (square symbol is measured load remaining after full unloading)

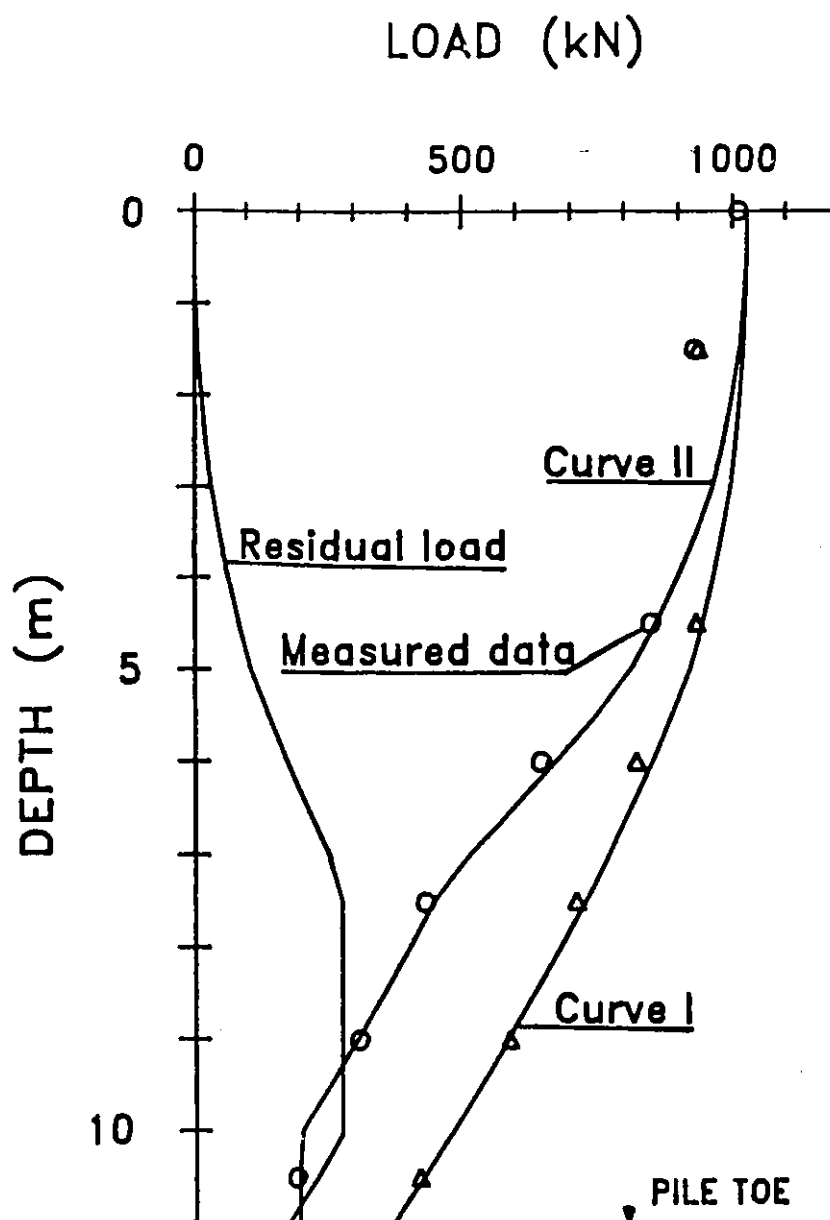


Fig. 8.8 Load distributions in the pile during test I showing the residual load, the measured load, the calculated true load (Curve I), and the calculated measured load (Curve II)

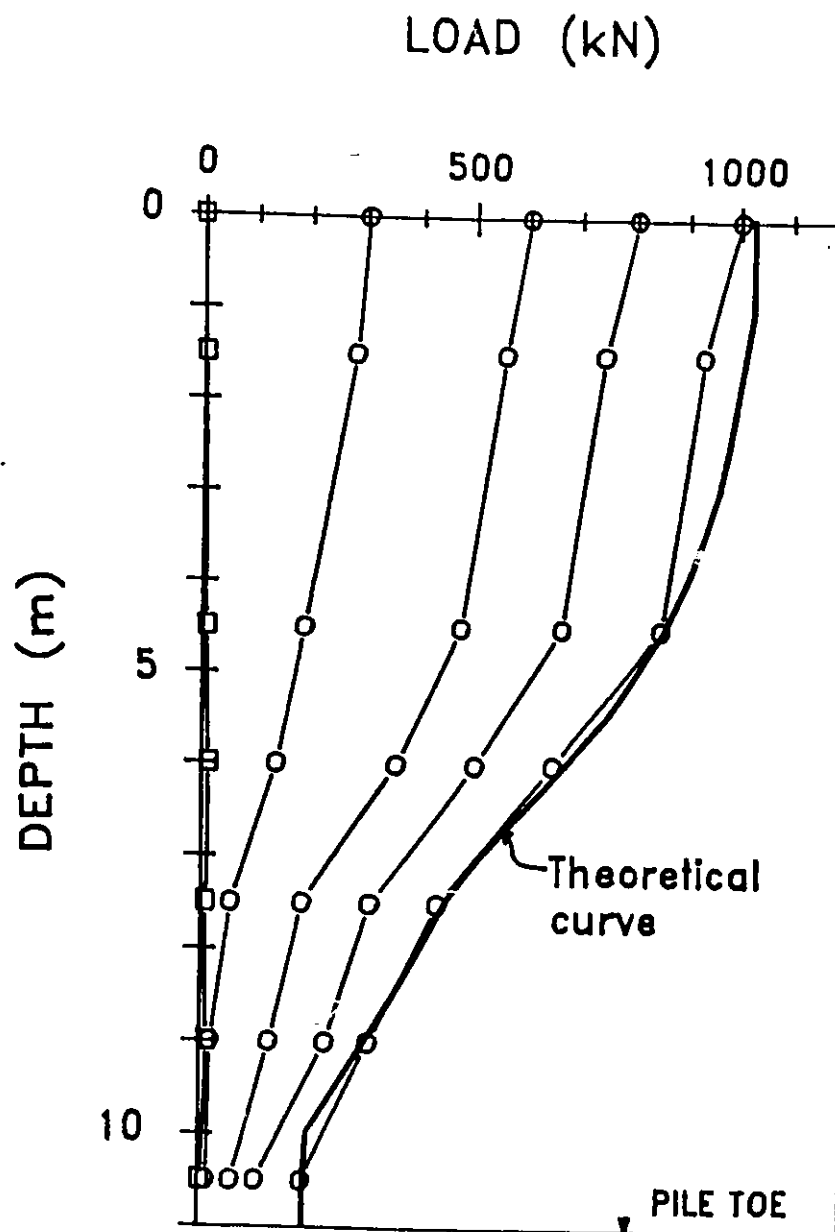


Fig. 8.9 Test I load distribution,
 (a) without the correction for residual load (square symbol
 is measured load remaining after full unloading)

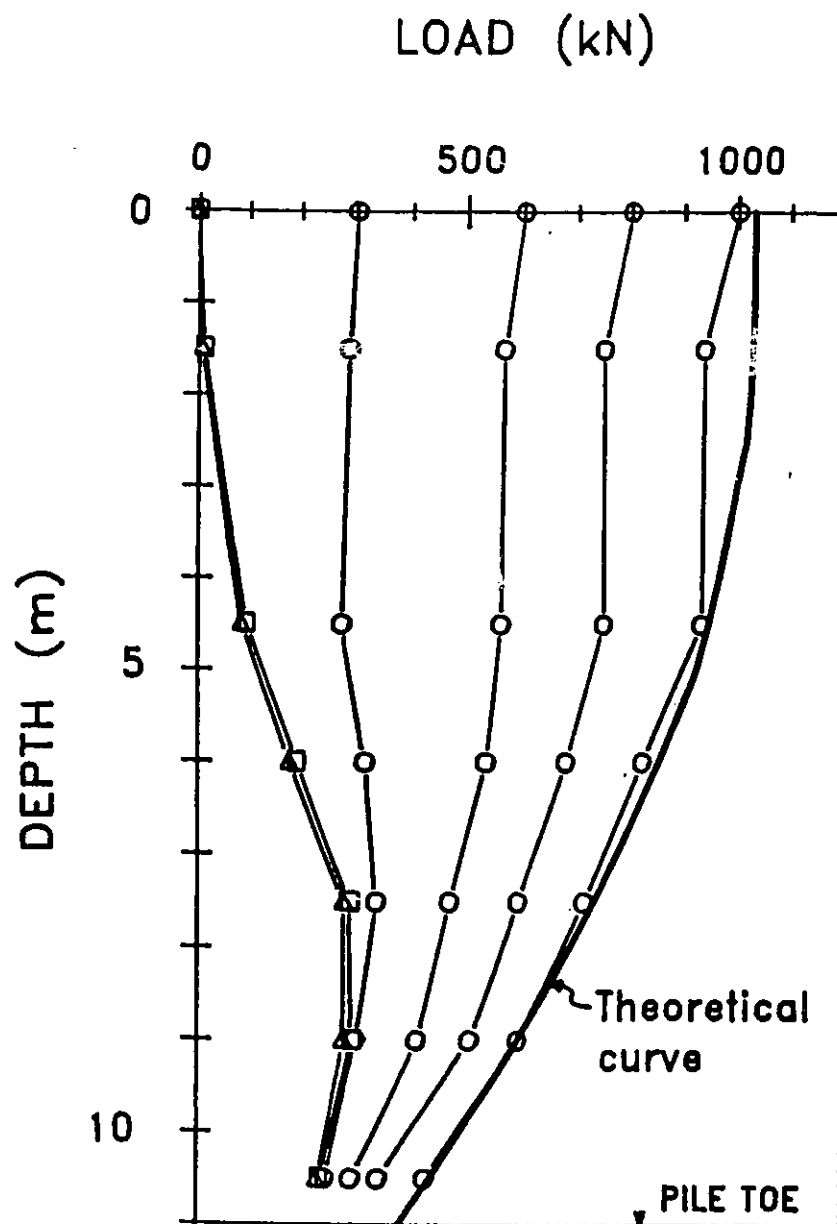


Fig. 8.9 Test I load distribution,
 (b) with the correction for residual load (square symbol is
 measured load remaining after full unloading, Triangular
 symbol is calculated residual load before the start of test)

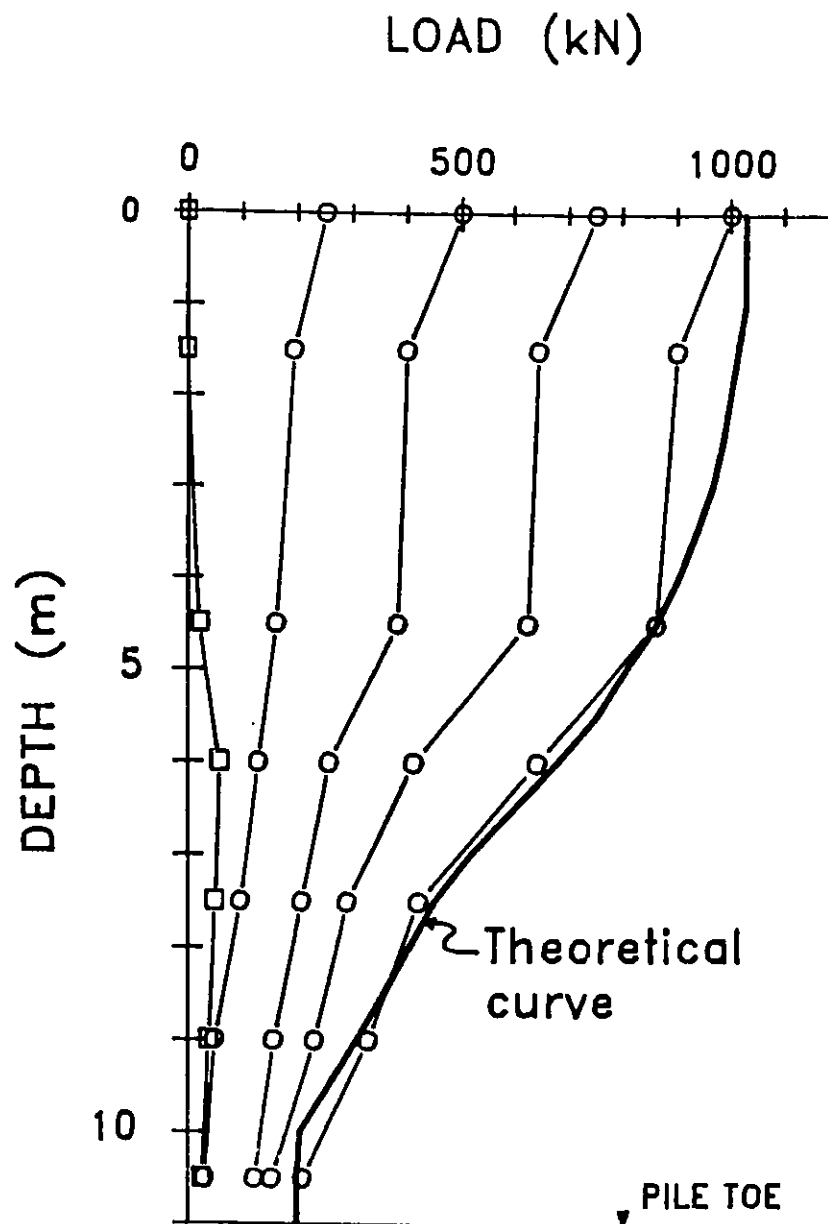


Fig. 8.10 Test II load distribution,
 (a) without the correction for residual load (square symbol
 is measured load remaining after full unloading)

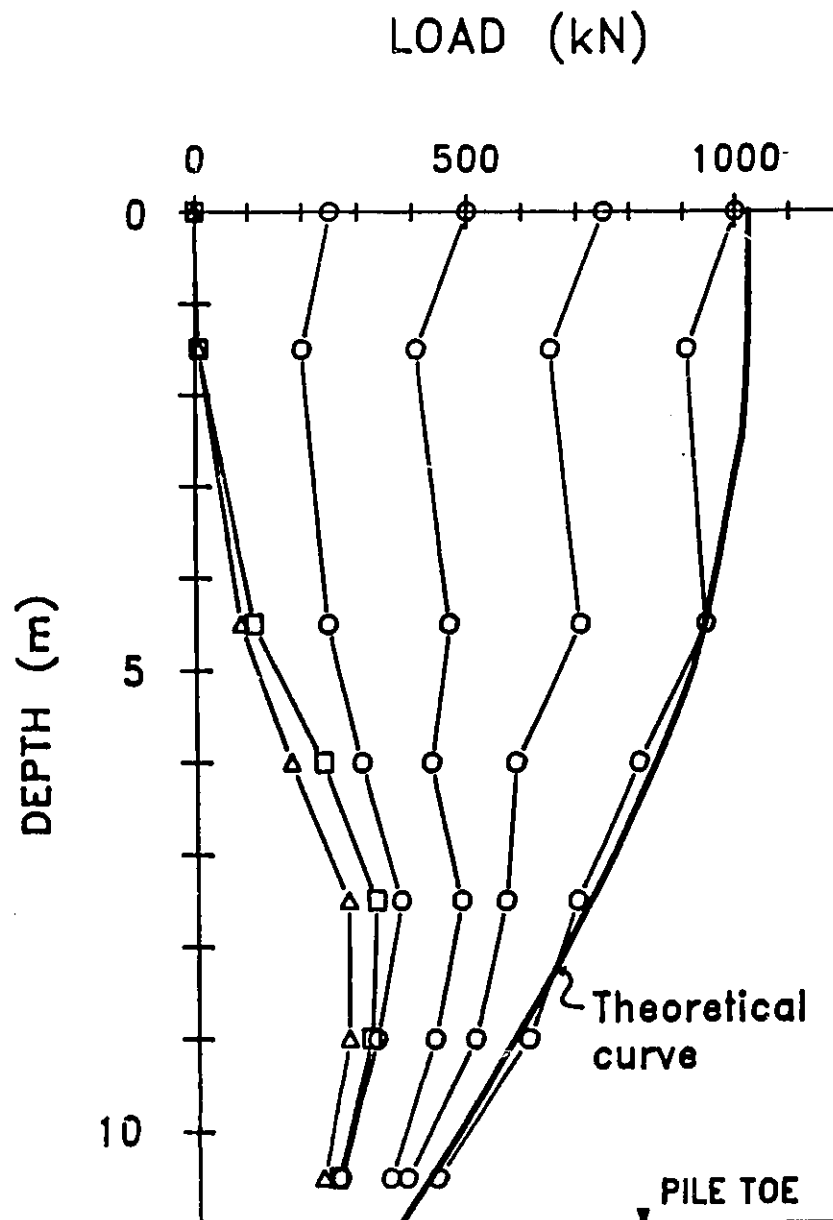


Fig. 8.10 Test II load distribution,

(b) with the correction for residual load (square symbol is measured load remaining after full unloading, Triangular symbol is calculated residual load before the start of test)

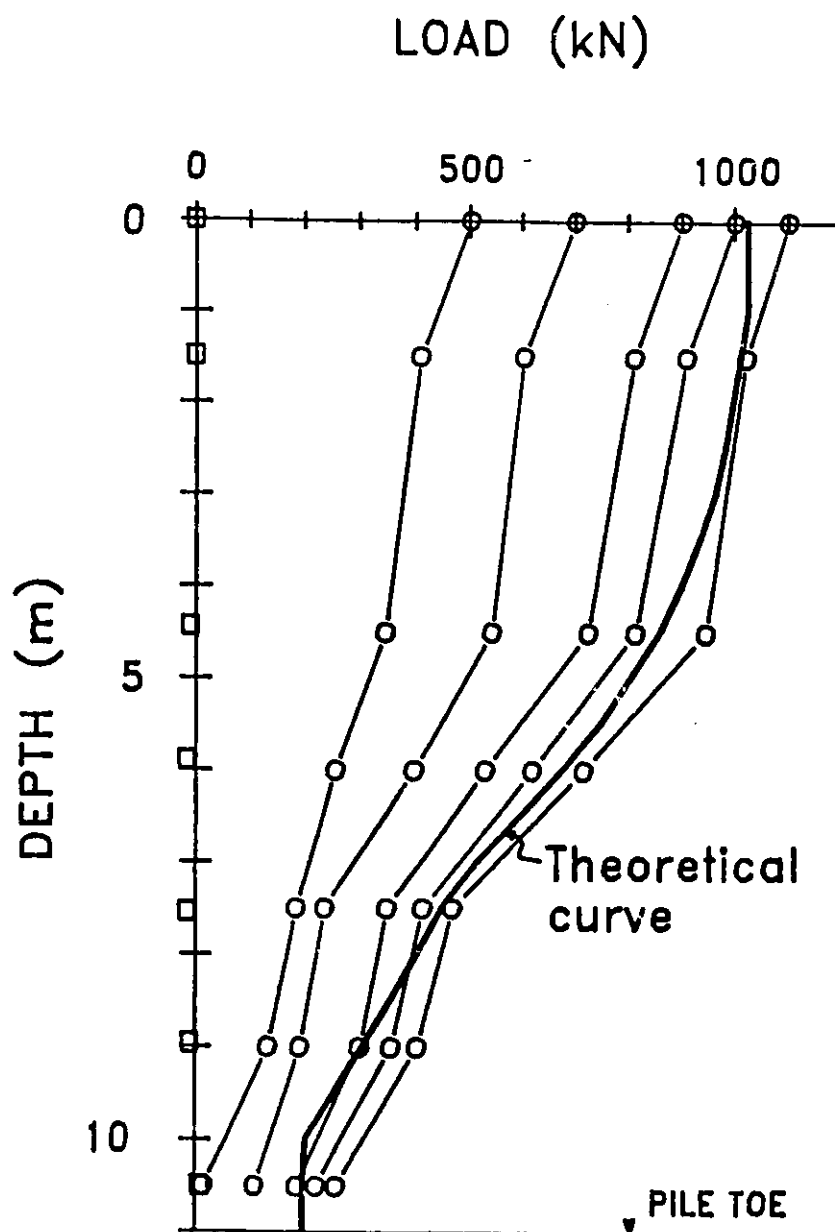


Fig. 8.11 Test III load distribution,
 (a) without the correction for residual load (square symbol
 is measured load remaining after full unloading)

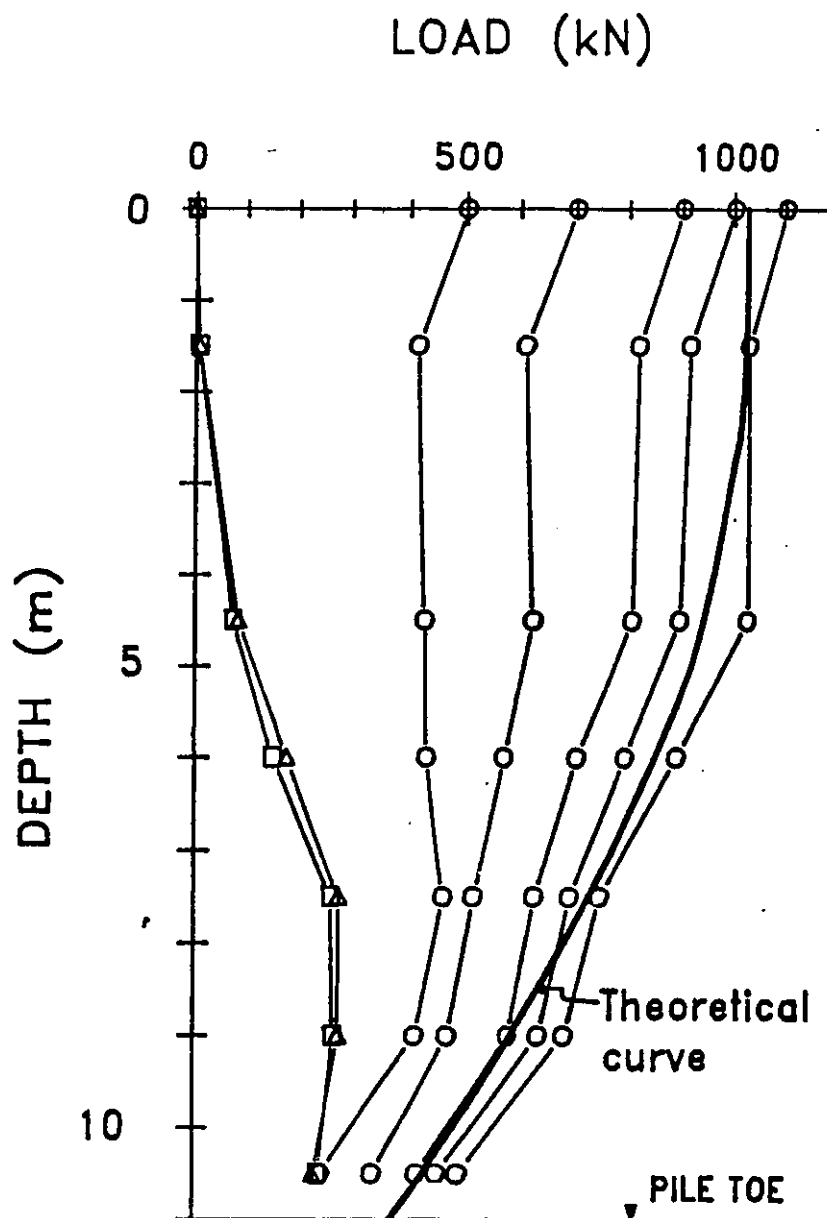


Fig. 8.11 Test III load distribution,
 (b) with the correction for residual load (square symbol is measured load remaining after full unloading, Triangular symbol is calculated residual load before the start of test)

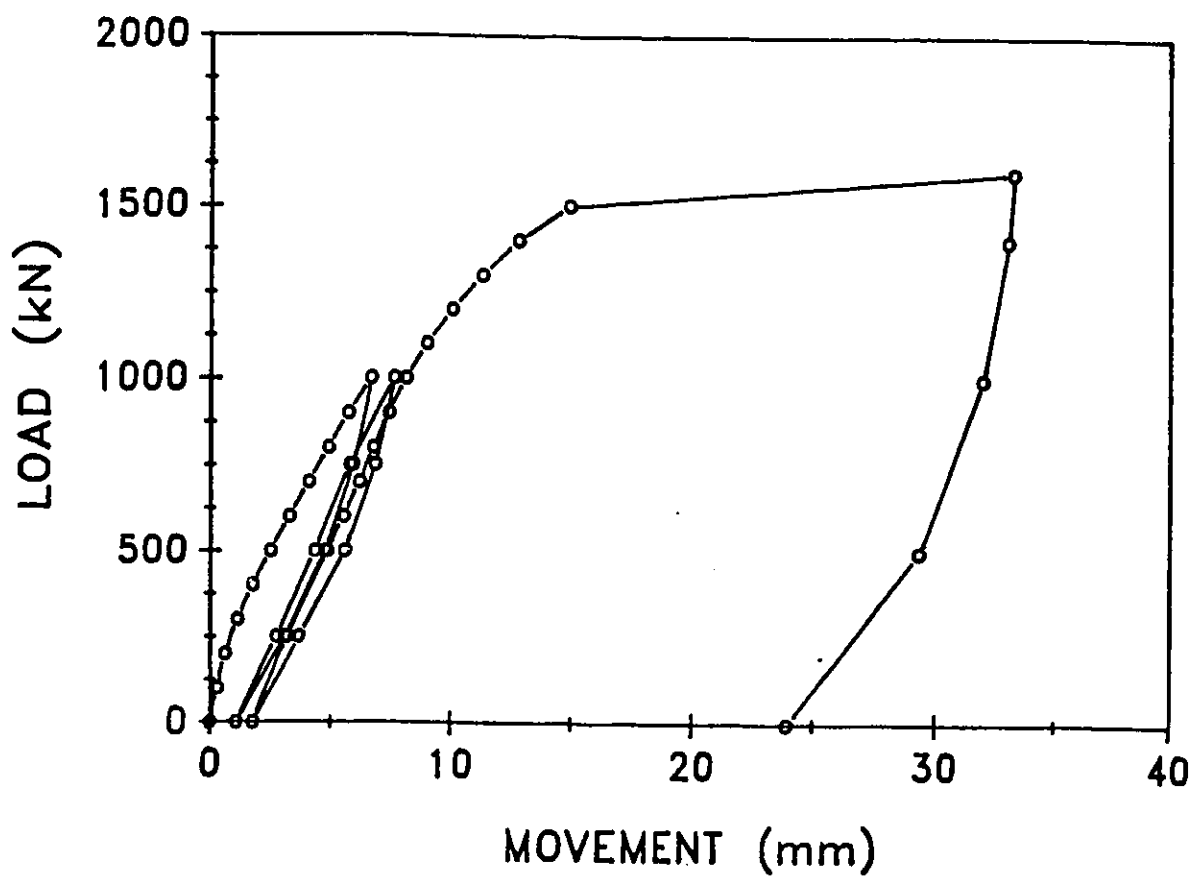


Fig. 8.12 Pile head load-movement diagram (Pile 2)

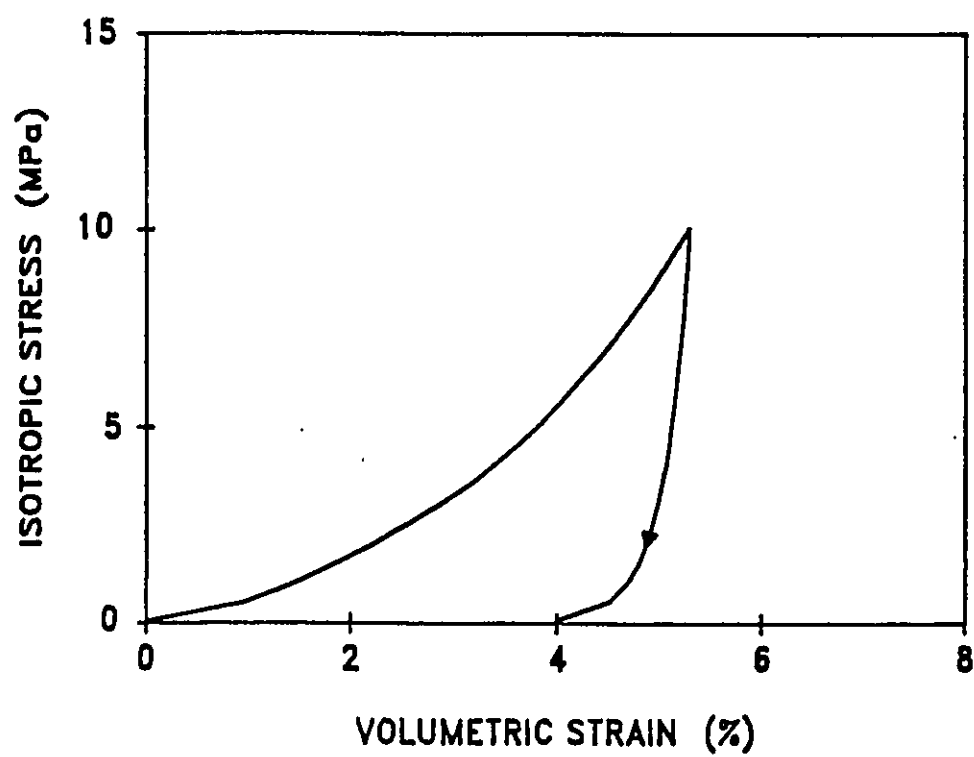


Fig. 9.1 Simulated isotropic compression test

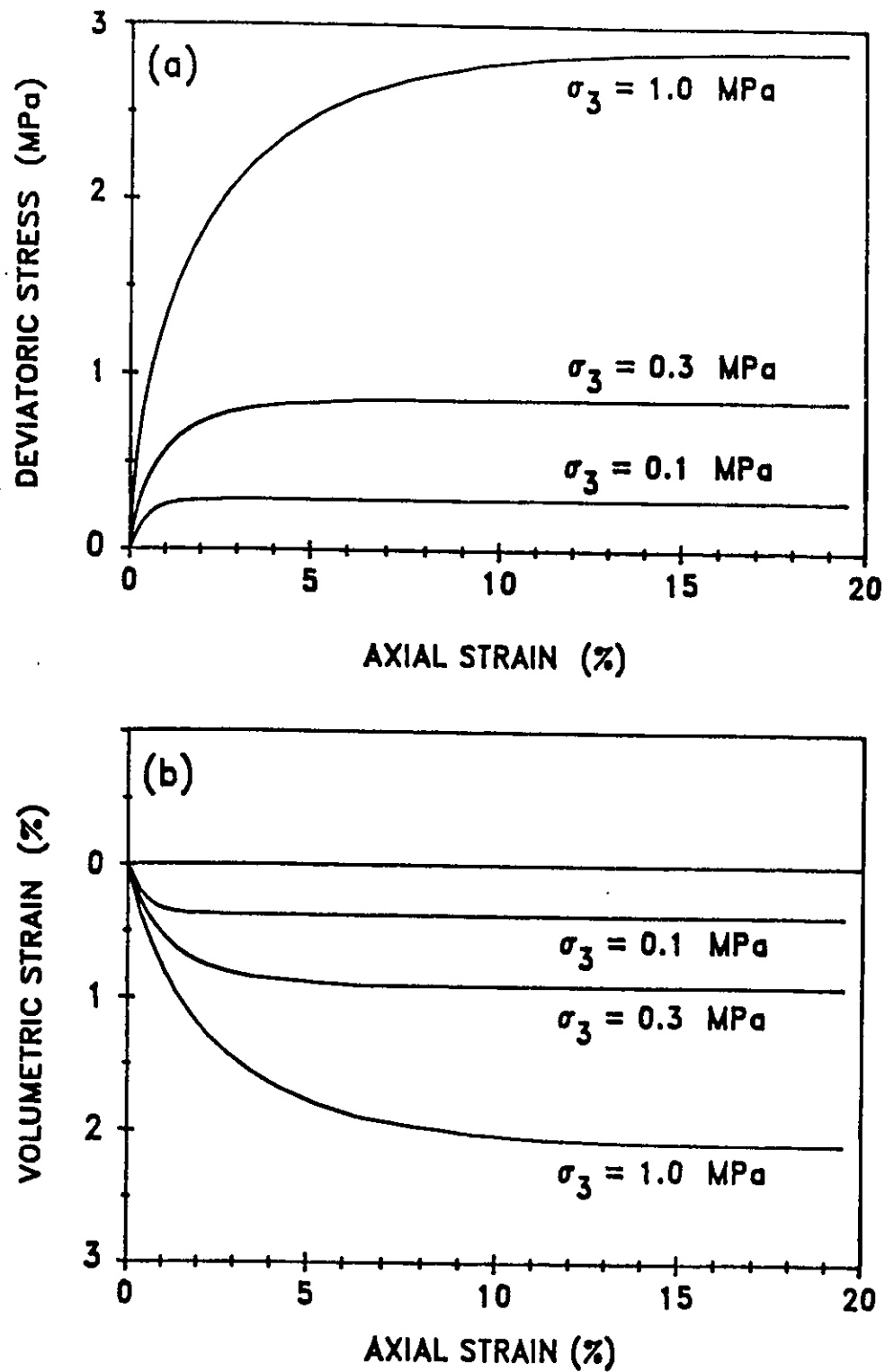


Fig. 9.2 Simulated drained triaxial compression tests
(a) deviatoric stress versus axial strain,
(b) volumetric strain versus axial strain

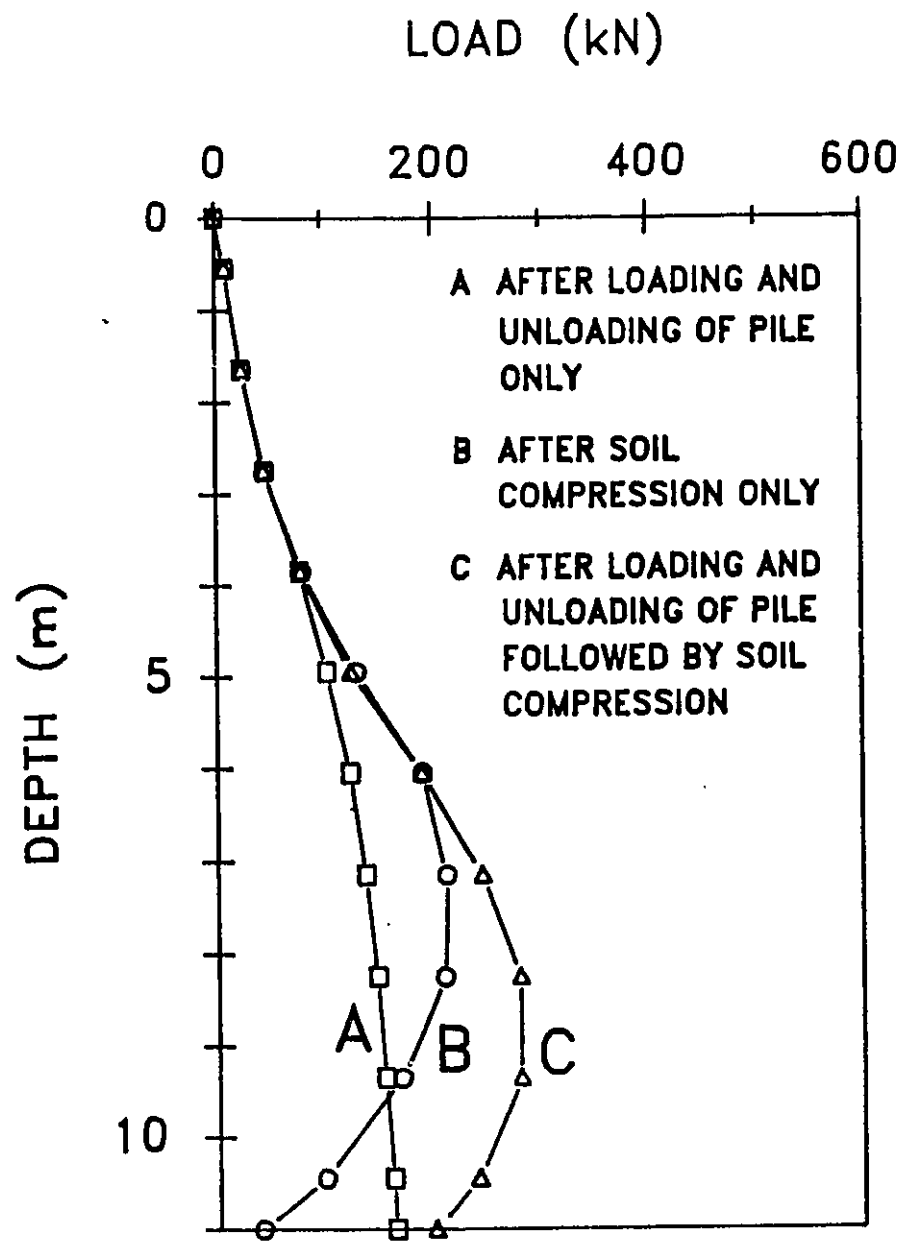


Fig. 9.3 Computed residual loads as a result of pile installation

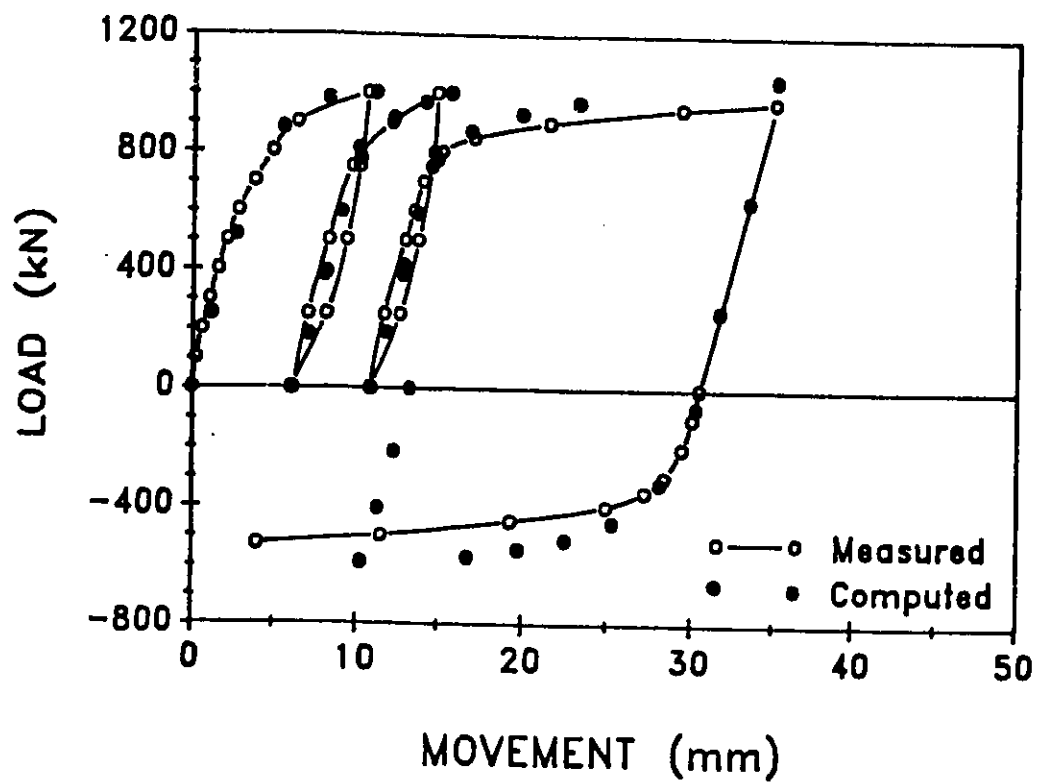


Fig. 9.4 Applied load versus pile head movement response;
Calculations versus measurements

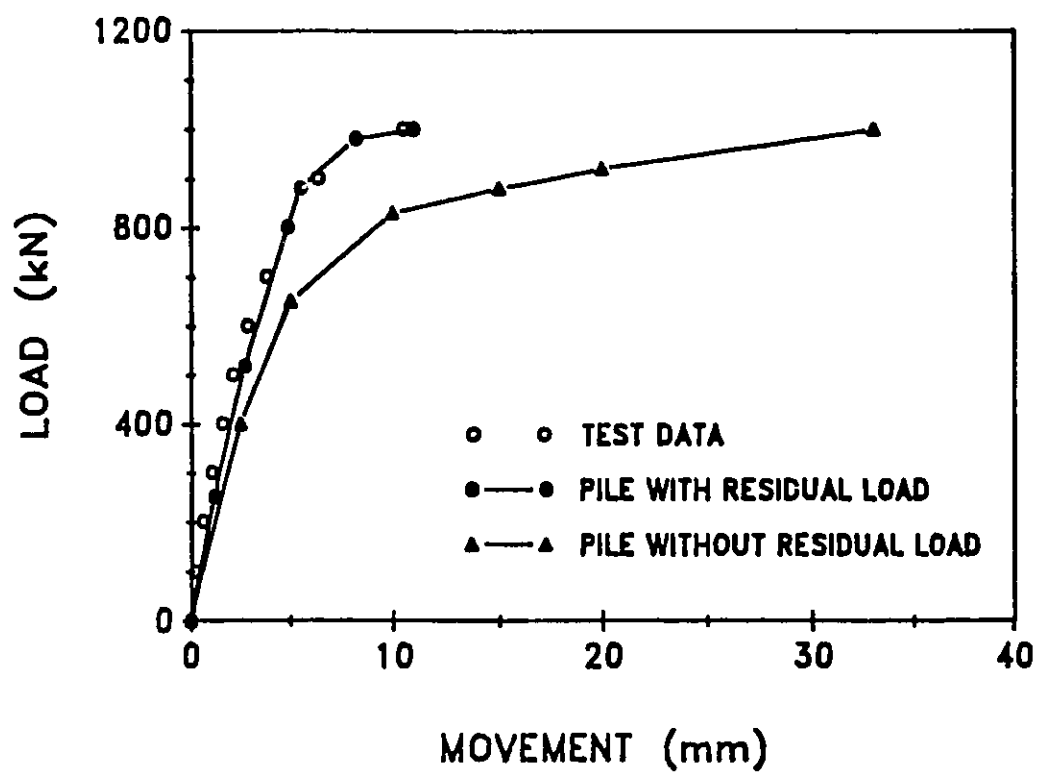


Fig. 9.5 Applied load versus pile head movement response;
Pile with residual load versus stress free pile

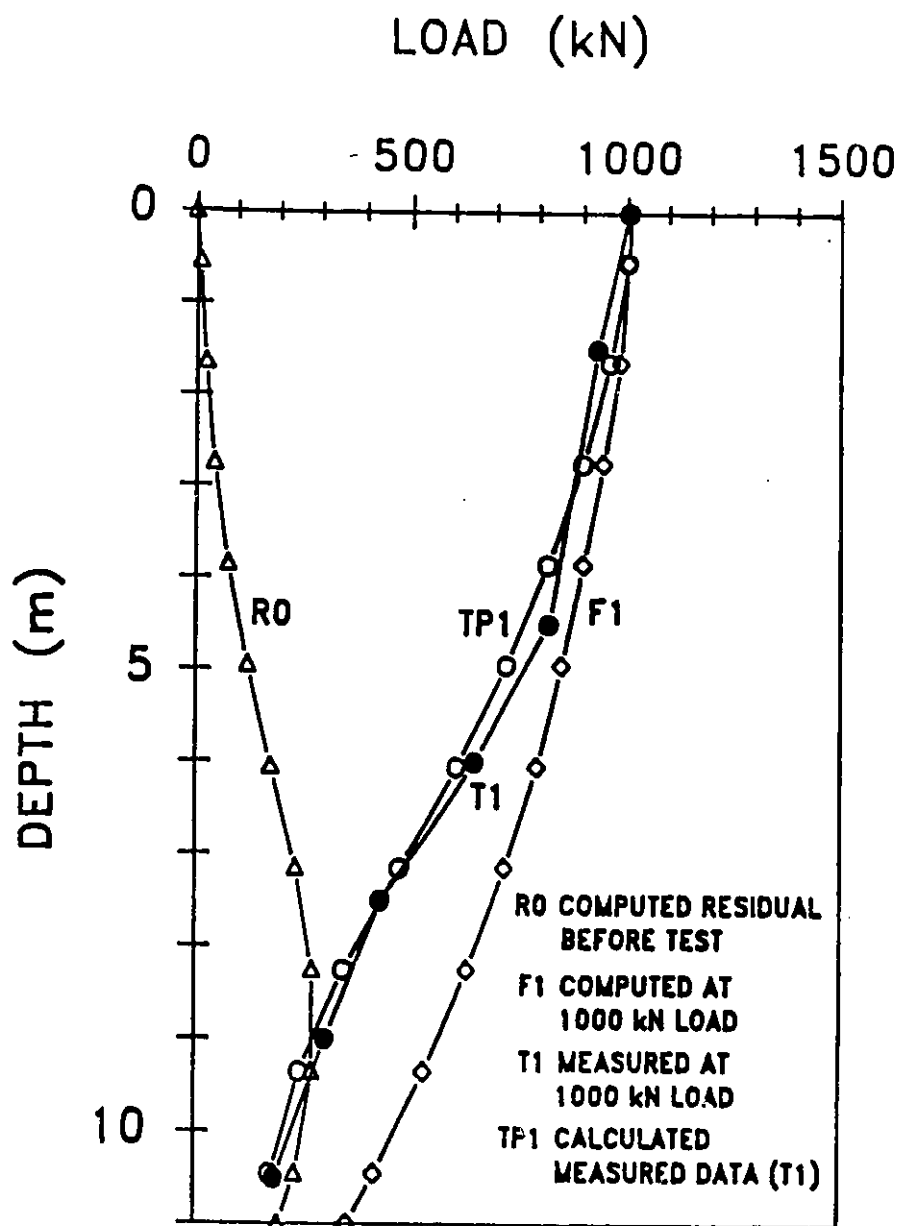


Fig. 9.6 Distribution of axial load in the first compression test

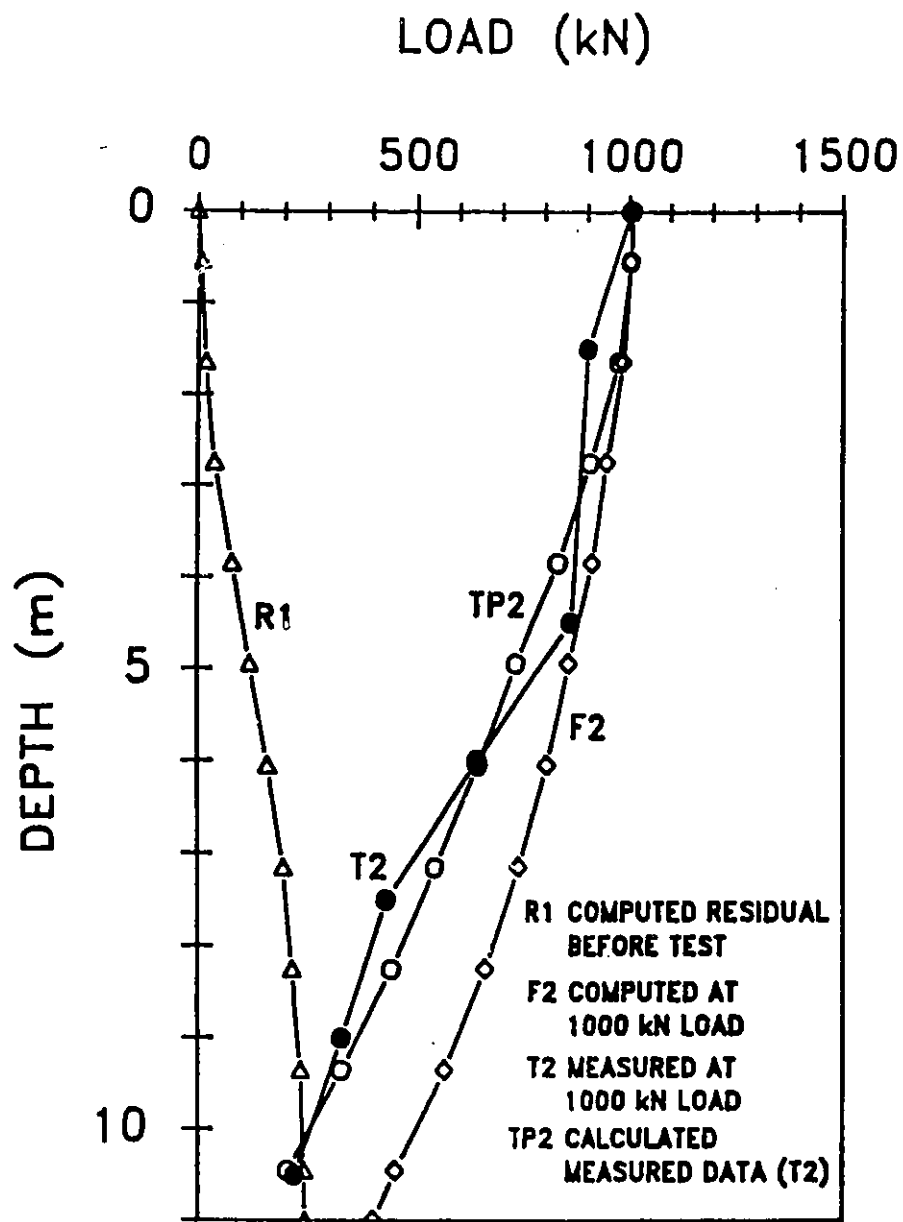


Fig. 9.7 Distribution of axial load in the second compression test

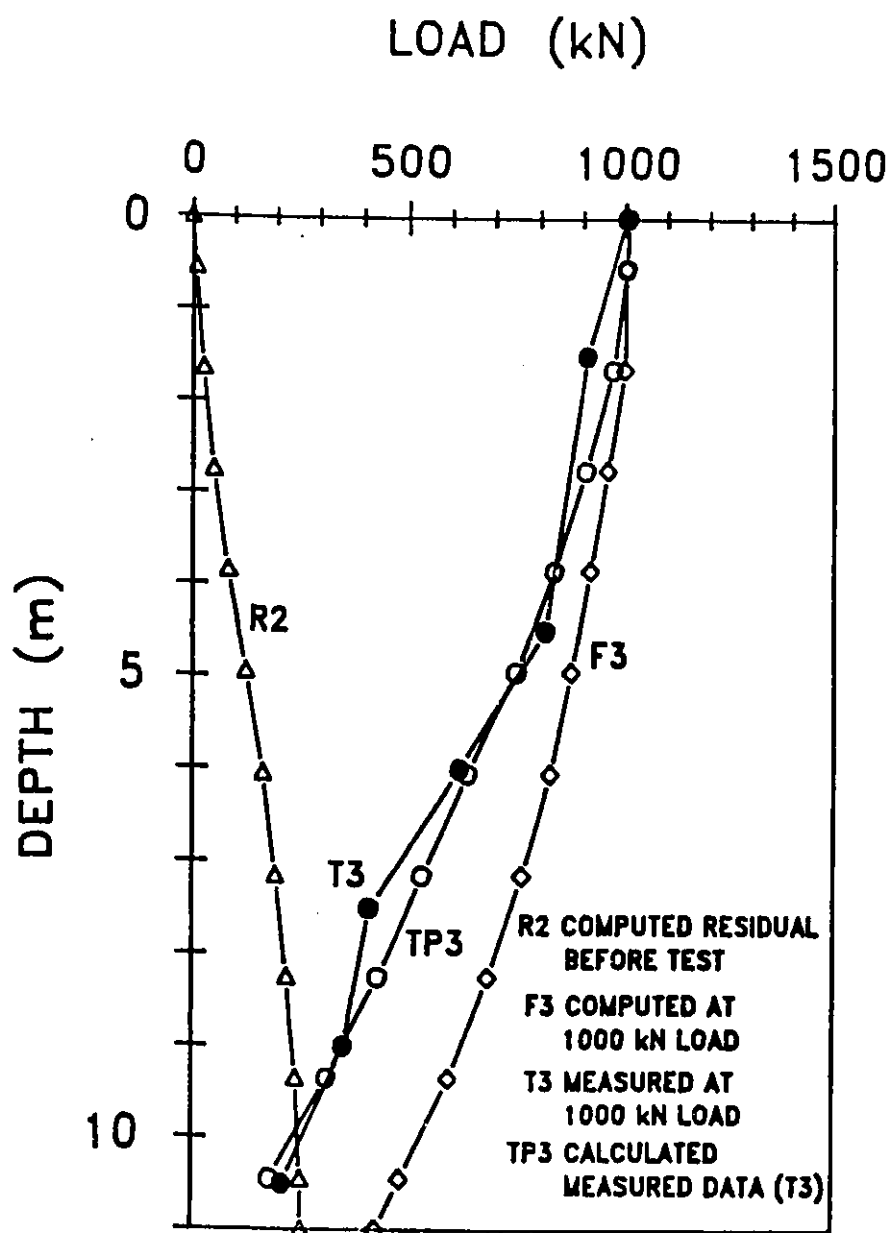


Fig. 9.8 Distribution of axial load in the third compression test

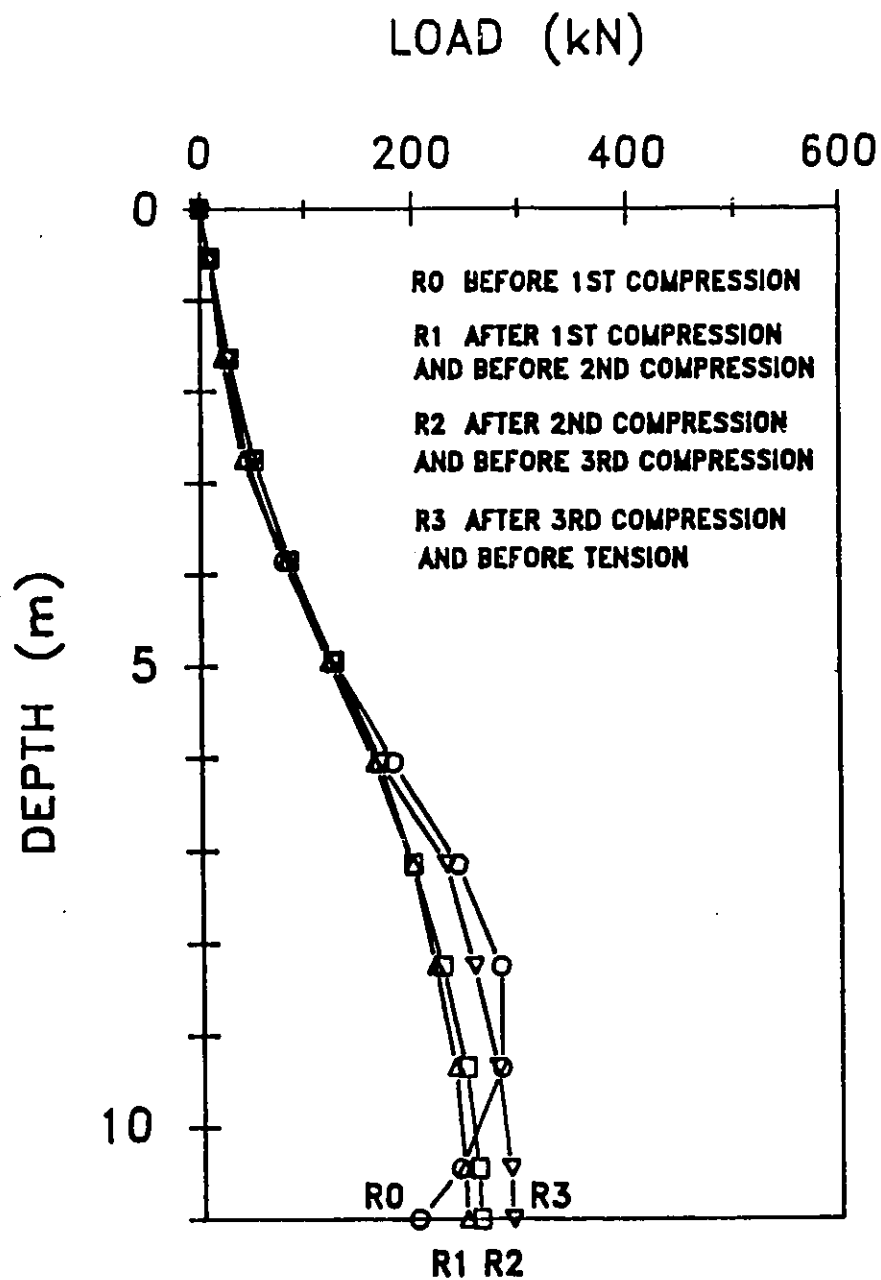


Fig. 9.9 Distribution of residual load at different stages of testing

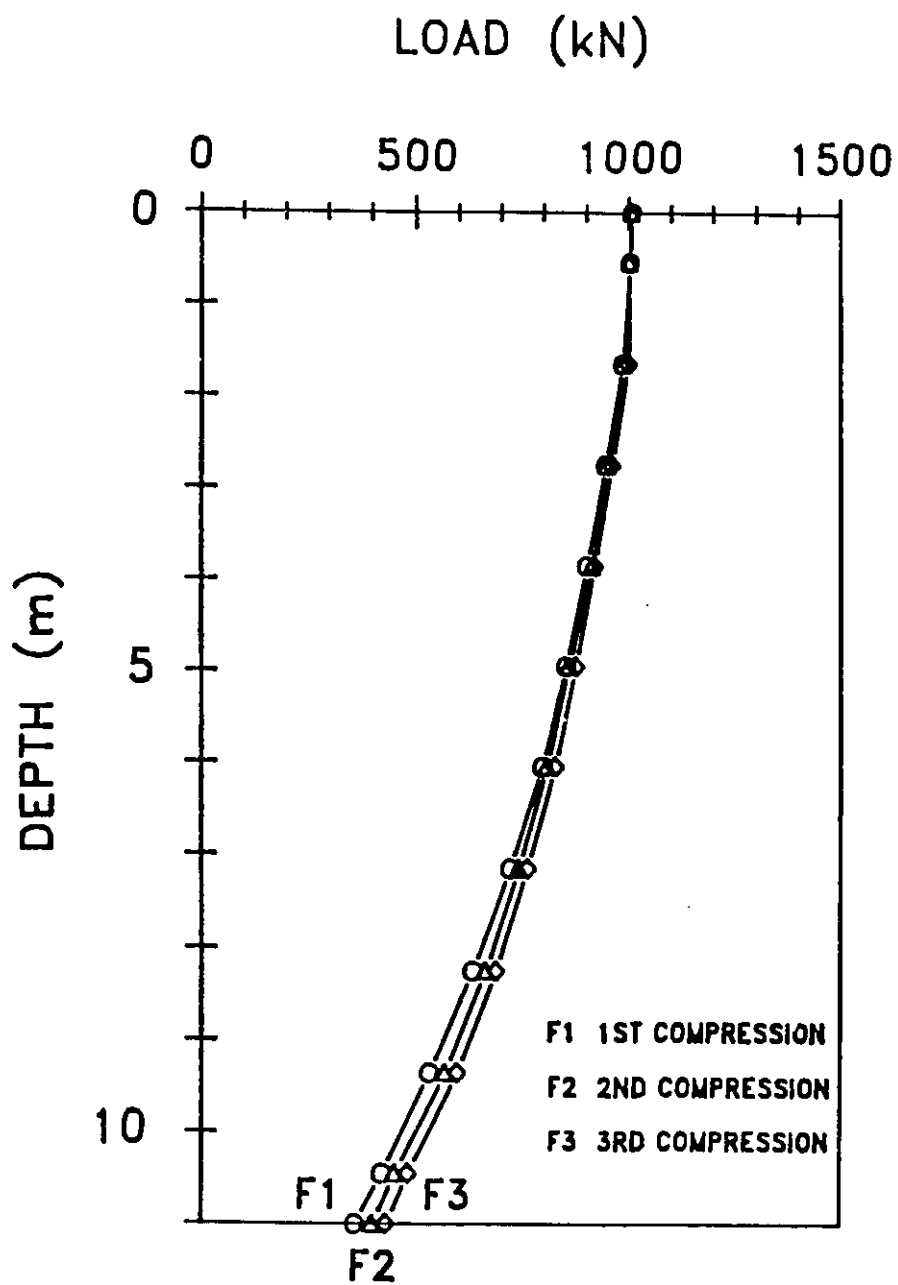


Fig. 9.10 Distribution of axial load in the pile at the end of each compression test

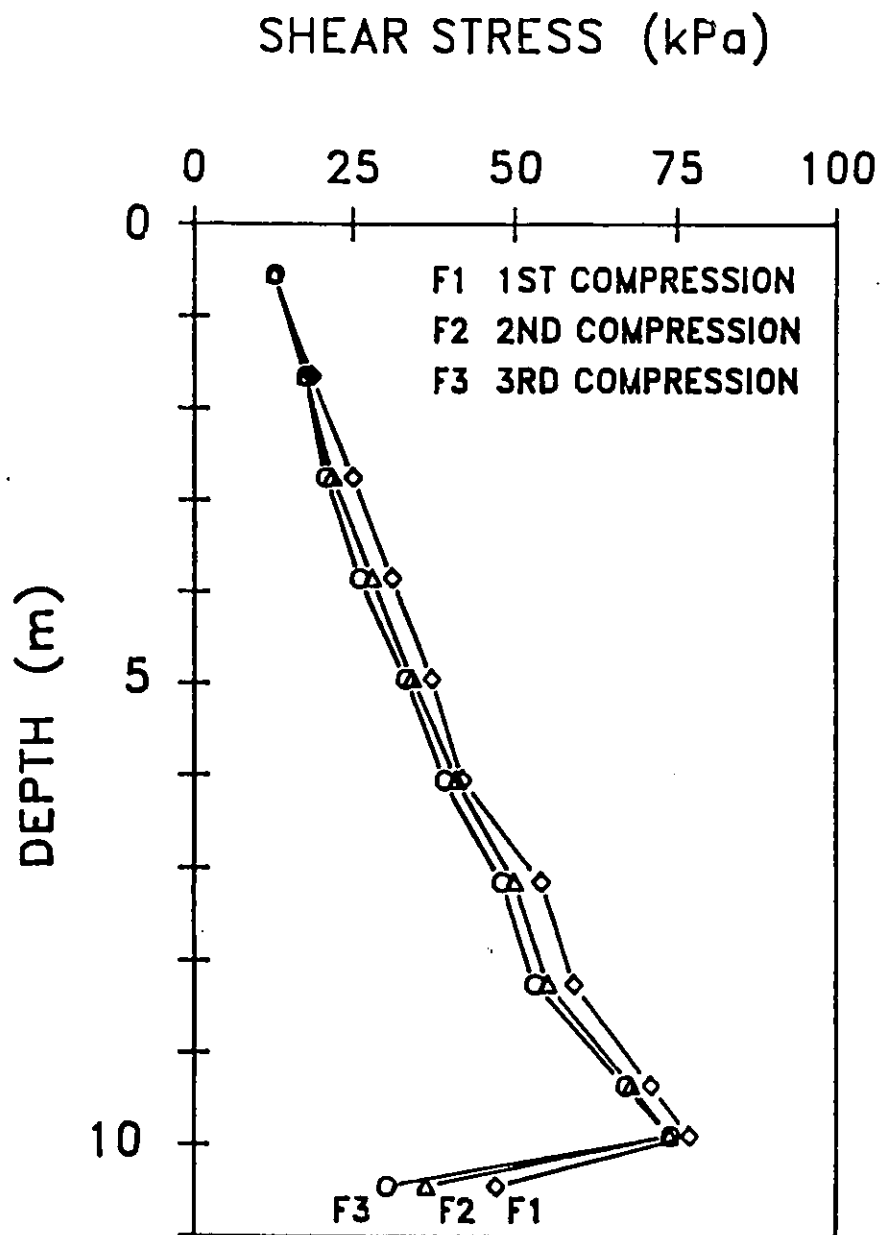


Fig. 9.11 Distribution of shear stresses in the soil adjacent to the pile shaft in three compression tests

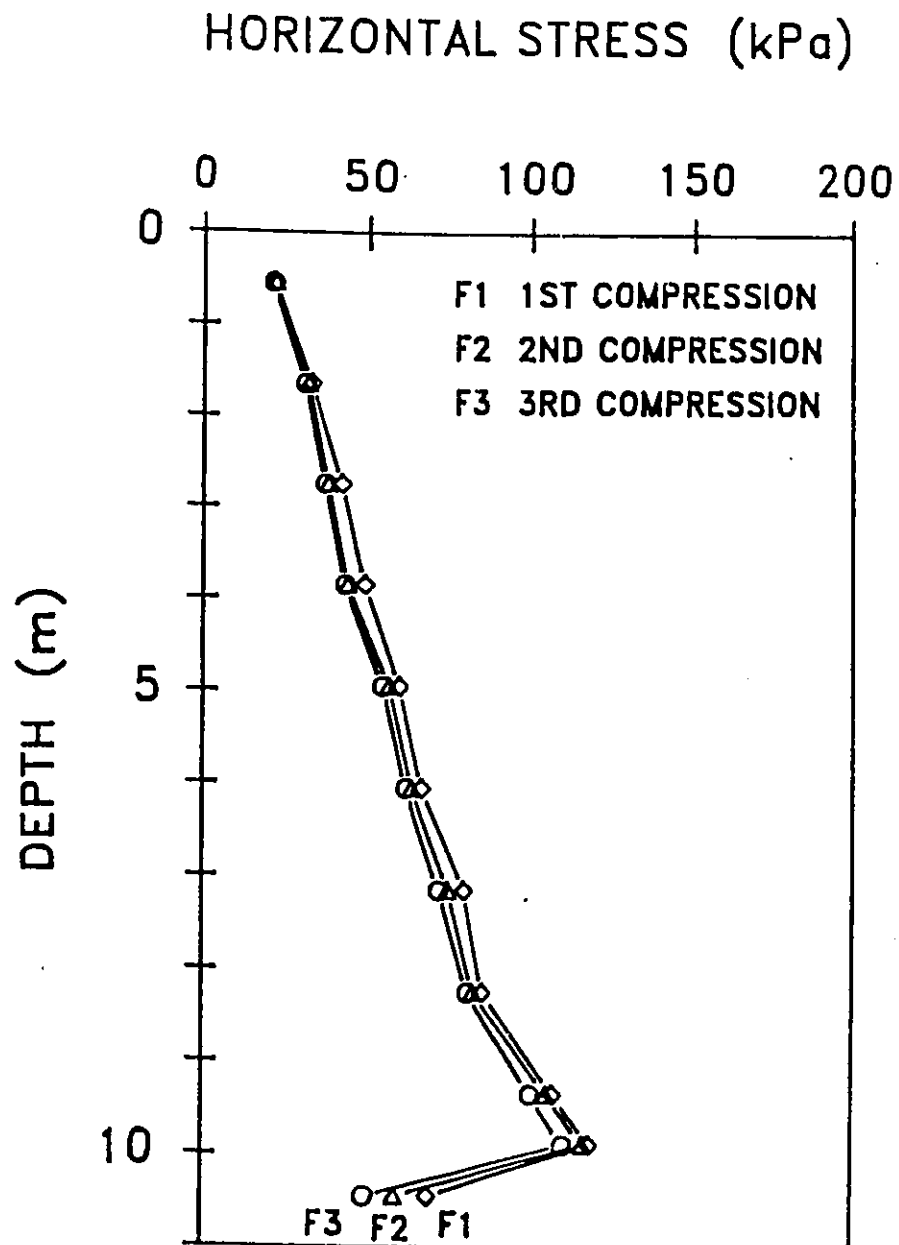


Fig. 9.12 Distribution of horizontal stresses in the soil adjacent to the pile shaft in three compression tests

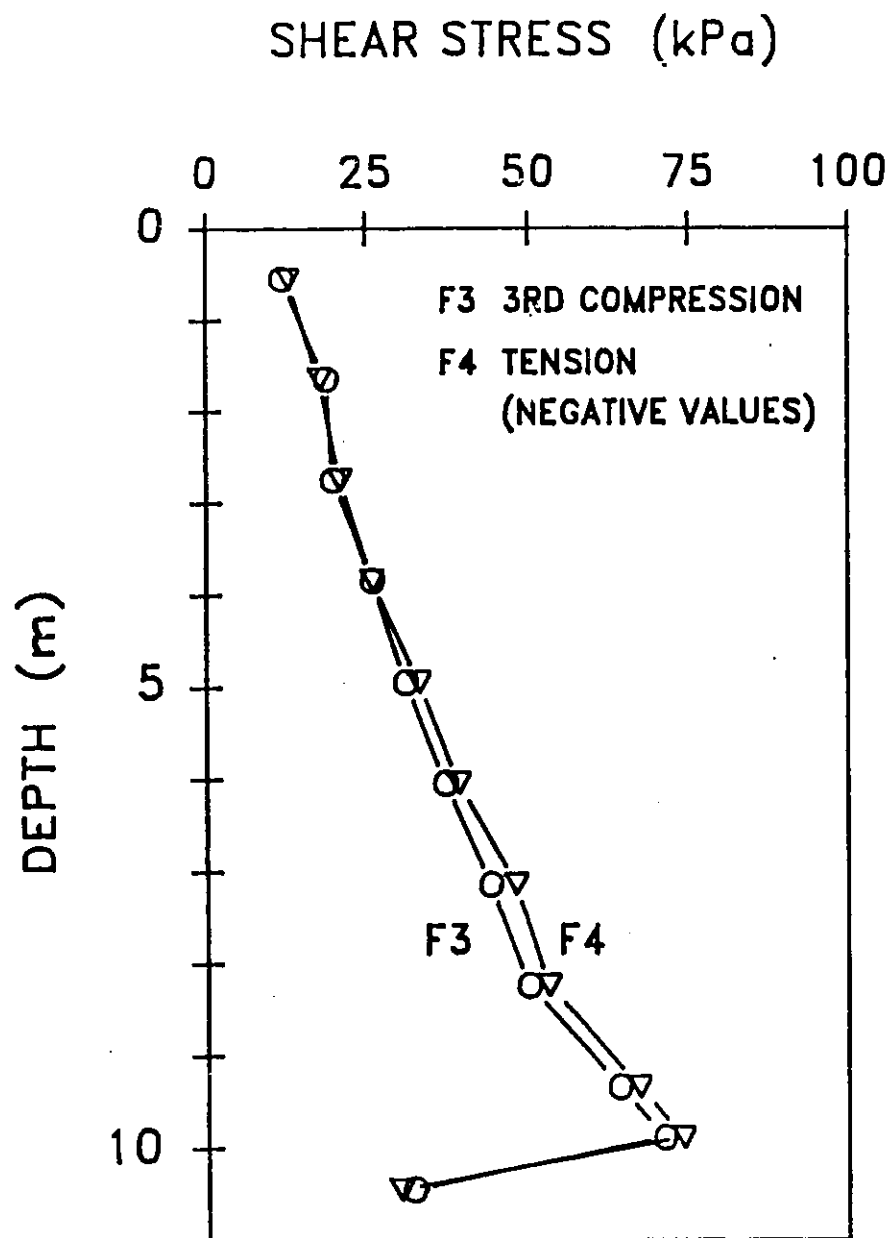


Fig. 9.13 Distribution of shear stresses in the soil adjacent to the pile shaft in the third compression and tension tests

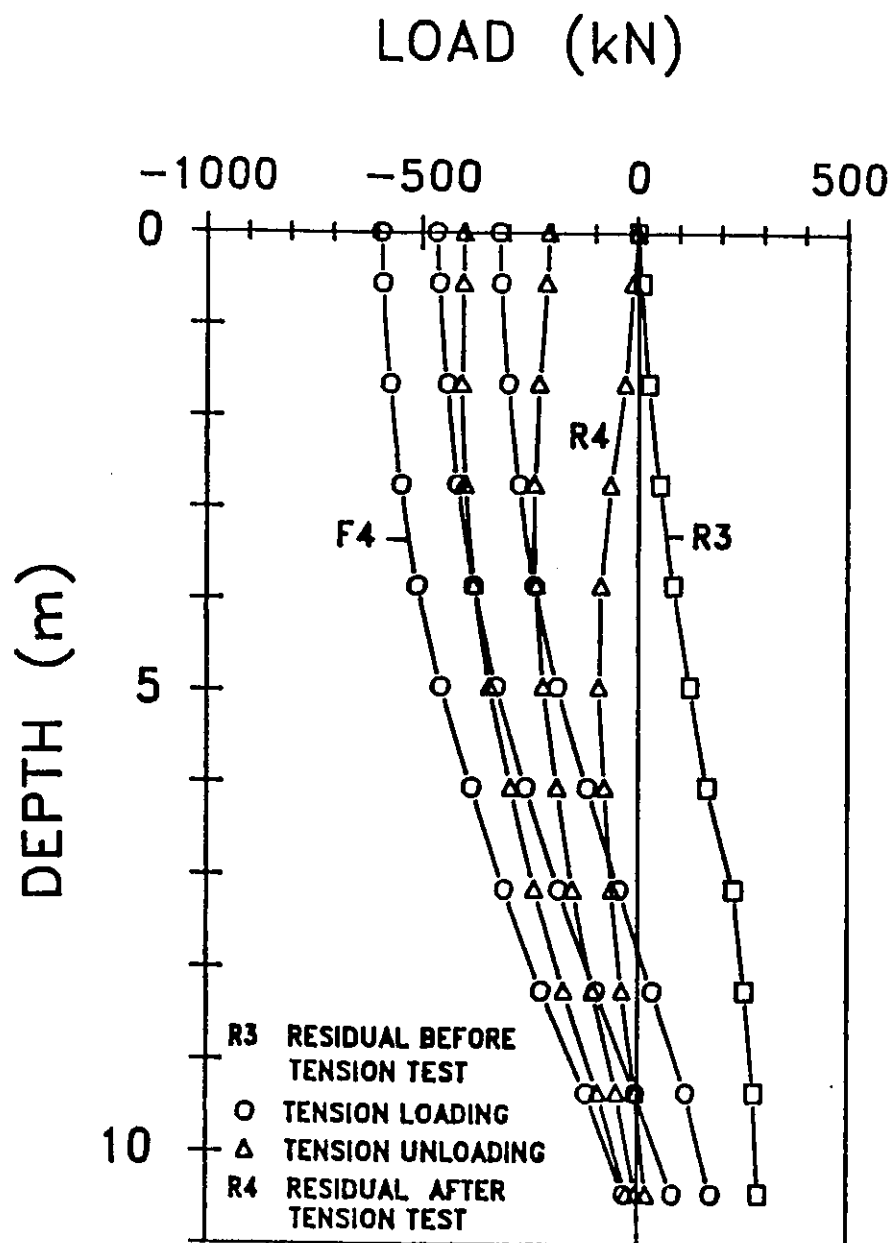


Fig. 9.14 Distribution of axial load in the pile in the tension test