AN EXPERIMENTAL STUDY OF AT REST LATERAL STRESS IN CEMENTED SANDS

FANYU ZHU

AN EXPERIMENTAL STUDY OF AT REST LATERAL STRESS IN CEMENTED SANDS

by

©FANYU ZHU, B. Sc., M. Sc.

A thesis submitted to the School of Graduate Studies
in partial fulfillment of the requirements for
the degree of Master of Engineering

Faculty of Engineering and Applied Science
Memorial University of Newfoundland

May 1993

St. John's Newfoundland Canada

DEDICATED TO

MY MOTHER

AND

MY WIFE

Abstract

At rest lateral stress in soils is an important parameter in geotechnical applications. In the past decades, much work has been done on the investigation of the lateral stress in sands. However, little information regarding lateral stress in cemented sands has been reported.

This thesis presents the results of a laboratory study on the at rest lateral stress and coefficient of at rest lateral stress (K_o) of cemented sands. A state of the art literature review is presented, in order to provide a background to the static and dynamic behaviours of cemented sands. A modified oedometer ring was used to measure the at rest lateral stress in cemented and uncemented sands. Test materials were No. 3 Ottawa sand and a marine sand, and portland cement. The specimens were prepared by the method of undercompaction using 0, 0.5, 1.0, 2.0, 4.0 and 8.0% of cement by the weight of dry sand. The water content of the specimens was 4% of dry sand and cement. The test program was designed to investigate the influences of cement content, vertical stress, sand density, curing period and stress history.

The test results indicate that the lateral stress in cemented sand decreases significantly with increasing cement content. The value of K_o increases with increasing vertical stress. The lateral stress decreases with increasing sand density and curing period. Stress history also has a significant influence on the behaviour of lateral stress in cemented sands.

Acknowledgements

I would like to appreciate the help and assistance from many people in my study for the degree of Master of Engineering. Particular thanks are due:

(a) My supervisor, Dr. J. I. Clark, for his guidance and support throughout this program.

(b) Dr. P. Morin and Dr. F. Poorooshasb for their help in using the laboratory.

(b) Mr. A. Bursey and Mr. C. Ward for their help in preparing the experimental device.

(d) Mr. H. Dye and other staff in the Machine Shop for their help and suggestions in manufacturing the oedometer ring.

(e) Mr. M. Paulin for his suggestions and discussions related to this thesis.

(f) Finally, my wife Ling for her help and understanding.

Contents

List of Figures

List of Tables

List of Symbols

c'	cohesion intercept
CC	cement content
CP	curing period
C_u	uniformity coefficient
D_{10}	effective grain size
D_{50}	mean grain size
D_{60}	limited grain size
e	void ratio of sand
E	vertical deformation modulus
I_D	density index of sand
K_o	coefficient of at rest lateral stress
OCR	overconsolidation ratio
P_a	atmospheric pressure
P_c	preconsolidation stress
q_f	unconfined compressive strength
U	degree of undercompaction
γ	shear strain

ε_v	vertical strain
ρ	sand density
σ_3	confining pressure (triaxial shear)
σ_h	lateral (or horizontal) stress (during loading)
σ_{hrl}	lateral stress during reloading
σ_{hrs}	residual lateral stress
σ_{hul}	lateral stress durring unloading
σ_v	vertical stress
σ_h^*	normalized lateral stress during reloading
τ	shear stress
ϕ'	internal friction angle

Chapter 1

Introduction

Lateral (or horizontal) stress in soils is an important parameter for the design of earth retaining structures and pile foundations and is a component of soil in-situ stress state which affects the static and dynamic properties of the soils. When lateral deformation is not allowed, the ratio of lateral stress, σ_h, to vertical stress, σ_v, is commonly defined as the coefficient of at rest lateral stress, expressed as

$$K_o = \frac{\sigma_h}{\sigma_v} \tag{1.1}$$

and in this case σ_h is called the at rest lateral stress. It should be noted that all the stresses mentioned in this thesis refer to effective stresses.

K_o is affected by soil physical properties (Andrawes and El-Sohby, 1973) and stress history (Brooker and Ireland, 1965; and Mayne and Kulhawy, 1982). Laboratory investigations have indicated that in many soils K_o remains constant during vertical loading and increases during unloading with increasing overconsolidation ratio (OCR). Mayne and Kulhawy (1982) have suggested that K_o is only a function of the effective stress friction angle (ϕ) and OCR, and can be approximated by the expression,

$$K_o = (1 - sin\phi)(OCR)^{sin\phi} \tag{1.2}$$

For normally consolidated soil, OCR = 1, and equation 1.2 becomes

$$K_o = 1 - sin\phi \tag{1.3}$$

Equation (1.3) is identical to that proposed by Jaky (1944). However, published investigations of K_o have been limited to uncemented soils. Little information regarding lateral stress in cemented sands has been obtained.

Cemented sands exist naturally or are made artificially. Stabilization of sands using cement has been used to increase the liquefaction resistance of foundations (Dupas and Pecker, 1979). Many naturally cemented sands occur in the marine environment due to the precipitation of calcite cement and are found in coastal and offshore areas (Murff, 1987; Clough *et al.*, 1981; and King *et al.*, 1980). The time-dependent stiffness and strength increases in freshly deposited or densified sands may be also due to the development of cementation bonds among the sand particles (Mitchell and Solymar, 1984; Mitchell, 1986; and Charlie *et al.*, 1992). The behaviour of cemented sands have been shown to be different in many ways from those of uncemented sands, especially at low strain and stress. In the past decades, much attention has been drawn to the studies of the behaviours of cemented sands for geotechnical applications. Laboratory tests were carried out to study the static stress-strain behaviour (Clough *et al.*, 1981), the low strain dynamic properties (Acar and El-Tahir, 1986; and Saxena *et al.*, 1988) and the cyclic strength and liquefaction resistance (Dupas and Pecker, 1979; and Clough *et al.*, 1989). Some parameters of cemented sands at low strain, such as the maximum shear modulus, cyclic strength and liquefaction resistance, are much greater than those of uncemented sands. However, at high strain cementation bonds are broken and at high confining pressure the effect of friction is more important than that of cementation cohesion.

In offshore engineering, the studies of the behaviour of cemented sands are especially important because it is recognized that in many areas, continental shelves are covered with calcareous sediments which are often cemented. Extensive investigations, summarized by Murff (1987), have indicated that driven piles in calcareous sands can develop much lower friction resistance than in silica sands due mainly to the lack of lateral stress development. However, a clear understanding of the influence of cementation on pile capacity has not been achieved. To further complicate matters, many cemented marine sediments are overconsolidated due to erosion and/or glaciation. Study of the lateral stress response of cemented sands as well as the effect of stress history is of importance in predicting the in-situ stress state of cemented sediments.

To provide more insight into the behaviour of cemented sands, this thesis presents and analyzes laboratory test results in which measurements of lateral stress in two artificially cemented sands under K_o conditions were carried out. The following chapter will present a general review of the behaviour of cemented sands. The experimental program and results will be introduced later.

Chapter 2

Behaviour of Cemented Sands

Cemented sands exist naturally (Murff, 1987; and Saxena and Lastrico, 1978), or are made artificially to stabilize soil foundations (Dupas and Pecker, 1979; and Kolias and Williams, 1984). In recent decades, extensive investigations of the dynamic and static behaviour of cemented sands have been carried out. It is recognized that the behaviour of cemented sands is quite different in many ways from that of uncemented sands. Although little information directly related to the at rest lateral stress in cemented sands is available, this chapter will present a review of the available studies of cemented sands in order to provide some insight to the general behaviour. This review is believed to be helpful to comprehend the lateral stress behaviour in cemented sands to be presented later in this thesis.

2.1 Cemented Sands under Static Loading

2.1.1 Stress-strain behaviour

Clough *et al.* (1981) studied the stress-strain relations and volume change of artificially and naturally cemented sands in triaxial shear tests. Four naturally cemented sands were used, two of which were weakly cemented, one was strongly cemented and the other was termed moderately cemented. During sampling, the strongly cemented sand had to be cut by a power saw. The weakly cemented sands could be easily cut by hand trowel and broken by light finger pressure. A sand with medium grain size of 0.75 mm was artificially cemented using 2 and 4% of portland cement to study the effect of density and cementation. The authors found that for both artificially and naturally cemented sands, the stress-strain behaviour is influenced by the degree of cementation and the confining pressure. Based on the test results, they concluded that:

(1). The stiffness and peak strength increase with increasing confining pressure.

(2). The post-peak response depends highly on the degree of cementation and the confining pressure. There is a transition from brittle to ductile failure as the confining pressure increases. Sand with a lower degree of cementation demonstrates the transition at lower confining pressure.

(3). The cementation bonds are broken at low strain and the frictional component is developed at larger strain.

(4). Compared with uncemented sands, cemented sands generally demonstrate volumetric increases during shear at a faster rate and at a smaller strain.

Stress-strain behaviour of natural slightly cemented sands was presented by Saxena and Lastrico (1978). Typical results of undrained triaxial compression tests have indicated that the maximum deviator stress at failure increases with increasing confining pressure. Most of the stress-strain curves showed linear behaviour up to about 1% axial strain. At approximately 1% axial strain, the cementation bonds break and a transition to a purely frictional resistance begins. They also noted that high hydrostatic confining pressure could also destroy the cementation.

In the studies of the effects of reinforcing elements on the behaviour of cemented sands in the plane strain test, Li and Mitchell (1988) noted that failure of weakly cemented sands developed at strains of 0.5-1%, while mesh element reinforcements and anchored wire reinforcements increased the shear strength and the ductility of weakly cemented sands. Reinforced cemented sands could sustain strains of 4-6% before failures developed.

2.1.2 Strength

Unlike uncemented sands of which the internal friction angle is the only strength parameter, the strength of cemented sands results from both cementation cohesion and friction. The contribution of cementation or friction to the strength depends mainly on the degree of cementation and confining pressure (Clough *et al.*, 1981; and Saxena and Lastrico, 1978). Cohesive shear strength appears to be destroyed at strain of less than 1%; subsequently the frictional strength becomes predominant. Strength parameters of cemented sand are effective cohesion intercept c' and effective internal friction angle ϕ'. Clough *et al.* (1981) observed linear relationships between deviator stress and mean effective principal stress at failure in triaxial compression tests for uncemented sand and artificially cemented sand. Similar results were also obtained for naturally cemented sands. The friction angles appeared not to be significantly affected by the degree of cementation; the cohesion intercept increases with increasing amount of cement.

Table 2.1: Strength Parameters of Cemented Monterey No. 0 Sand (After Acar and El-Tahir, 1986)

Cementation (%)	Density Index (%)	Cohesion intercept (kPa)	Friction Angle (degree)
0	31	0	33
	45	0	35
	77	0	39
1	25	5	33
	35	7	34
	50	9	35
	80	14	38
2	25	12	34
	35	17	35
	50	20	36
	80	30	39
4	60	123	29
	75	143	35
	90	153	41

Acar and El-Tahir (1986) reported the cohesion intercept and internal friction of uncemented and artificially cemented Monterey No. 0 sand; the results are listed on Table 2.1. The cohesion intercept and friction angle on Table 2.1 were determined by a straight line approximation of peak strength in CIU tests with confining pressure range of 35-345 kPa. The cohesion intercept increases with increasing cementation and the internal frictional angle is not significantly affected by cementation. The results are in accordance with those reported by Clough *et al.* (1981). However, the increases in friction angles of cemented soils were also reported, although the increase is relatively small. Wissa *et al.* (1965) observed that cementation leads to increases in friction angles of up to 5°. In the studies of the effects of cementation in frictional soils, Lade and Overton (1989) concluded that an increasing degree of cementation results in increasing cohesion intercept as well as increasing friction angles at low confining pressures.

Correlation of unconfined compressive strength from cohesion intercept was obtained by Acar and El-Tahir (1986) for artificially cemented Monterey No. 0 sand with cement contents of 1, 2, and 4%. The expression is given by

$$q_f = 2.1c' \tag{2.1}$$

where q_f is unconfined compressive strength and c' is cohesion intercept.

Residual strength of artificially and naturally cemented sands was reported by Clough *et al.* (1981). The residual strength was determined from stress-strain data where the curves leveled out after the peak strength. It was found that the residual friction angle was not influenced by the degree of cementation, although there was a small residual cohesion intercept. It was concluded that the residual strength of a cemented sand is close to that of an uncemented sand and the degree of cementation has no significant effect on the residual strength. It appears that failure of cemented sands can completely eliminate the cementation effect. The results of Clough *et al.* (1981) is in accordance with those of Wissa *et al.* (1965).

Due to the cementation bonds among soil particles, cemented sands exhibit the ability of sustaining small tensile stresses. Investigation of the tensile strength is of importance in engineering practice because theoretical and field evidence has indicated that failures in steep cemented sand slopes are often initiated by tensile splitting in the upper portion of the slopes (Sitar and Clough, 1983). Clough *et al.* (1981) performed tension tests on cemented sand and found that the tensile strength of cemented sand is approximately 10% of unconfined compressive strength. Similar results were also observed by Kolias and Williams (1984) from the test data of cement stabilized soils.

2.1.3 Deformation modulus

In the studies of naturally and artificially cemented sands, Clough *et al.* (1981) reported that the initial tangent modulus from drained triaxial shear test could be expressed as

$$E_i = K P_a (\frac{\sigma_3}{P_a})^n \tag{2.2}$$

where, E_i is the initial tangent modulus; K is the intercept at $\sigma_3/P_a = 1$; n is a constant parameter; σ_3 is confining pressure; and P_a is atmospheric pressure. The K value increases with degree of cementation, while the n value decreases with increasing cement content.

2.1.4 Pore pressure during undrained shear

From the undrained triaxial test results of naturally cemented sands, Saxena and Lastrico (1978) have found that the developed pore pressure reaches a peak value within one percent of axial strain and then decreases with increasing strain. This behaviour is similar to that of dilating or dense sand, and the cementation creates an "apparent high density" performance. The dilating behaviour of cemented sands is also reflected by the volume increase during triaxial shear tests conducted by Clough *et al.* (1981) and Saxena and Lastrico (1978).

2.2 Dynamic Properties of Cemented Sands

The dynamic modulus of soils decreases with increasing amplitude of strain. A relationship between dynamic shear modulus and the amplitude of shear strain is given by (Hardin and Drnevich, 1972a)

$$\frac{G}{G_{max}} = \frac{1}{1+\gamma_h} \tag{2.3}$$

where the hyperbolic strain, γ_h, is determined from the relationship

$$\gamma_h = \frac{\gamma}{\gamma_r}[1 + ae^{-b(\gamma/\gamma_r)}] \tag{2.4}$$

while G is the shear modulus, G_{max} is the maximum shear modulus, γ is the shear strain, γ_r is a reference strain, and a and b are constant parameters. Although the above two equations were originally proposed for uncemented soils, they appear to be suitable for cemented sands (Acar and El-Tahir, 1986). The reference strain, γ_r, is defined by Hardin and Drnevich (1972a) as

$$\gamma_r = \frac{\tau_{max}}{G_{max}} \tag{2.5}$$

where,

$$\tau_{max} = \overline{\sigma}_o sin\phi' + c' cos\phi' \tag{2.6}$$

and the maximum shear modulus of uncemented sands is expressed as (Acar and El-Tahir, 1986)

$$G_{max} = \frac{S}{0.3+0.7e^2}(P_a)^{1-n}(\overline{\sigma}_o)^n \tag{2.7}$$

in which, S is a stiffness coefficient, e is void ratio, $\overline{\sigma}_o$ is mean effective confining stress, P_a is atmospheric pressure.

2.2.1 Maximum shear modulus of cemented sand

Acar and El-Tahir (1986) carried out torsional resonant column tests to study the effect of cementation on dynamic properties of Monterey No. 0 sand. The maximum cement content was 4%, and the maximum confining pressure was 400 kPa. A tamping method was used for preparing uncemented sand specimens and the pluviation method was used for cemented sand specimens.

Table 2.2: Parameters for Maximum Shear Modulus of Cemented Sand (After Acar and El-Tahir, 1986)

Density index I_D (%)	Cementation CC(%)	Stiffness Coefficient S or S_c	Exponent n (Mean value)
25	0	621	0.42
	1	867	0.43
	2	1,122	0.43
	4	1,396	0.43
35	0	638	0.44
	1	918	0.43
	2	1,184	0.42
	4	1,481	0.42
50	0	624	0.43
	1	1,028	0.42
	2	1,318	0.42
	4	1,586	0.43
75	0	658	0.43
	1	1,115	0.42
	2	1,387	0.42
	4	1,651	0.41

It is found that for maximum shear modulus, the stiffness coefficient of cemented sand, S_c, increases with increasing cementation, while the exponent, n, appears not to be affected by the amount of cement. The test results are shown on Table 2.2.

From the data listed on Table 2.2, maximum shear modulus of the cemented sand is revised as

$$G^*_{max} = R\frac{S}{3+0.7e^2}(P_a)^{0.57}(\overline{\sigma}_o)^{0.43} \tag{2.8}$$

where G^*_{max} is the maximum shear modulus of cemented sand; R is the stiffness ratio (S_c/S); S_c is a dimensionless stiffness coefficient of cemented sand; and S is a dimensionless stiffness coefficient of uncemented sand (the mean value of S is 631 for the Monterey No. 0 sand). The stiffness ratio, R, which is a function of cement content, CC, and void ratio, e, is expressed as

$$R = 1 + (CC)^{0.49} - 2.0(CC)^{0.1}e^{4.6} \tag{2.9}$$

and correlation of the stiffness ratio with the unconfined compressive strength yields

$$R = 0.61q_f^{0.33} \tag{2.10}$$

where R is a dimensionless parameter; the unit of q_f is kPa.

Saxena, Avramidis and Reddy (1988) also used Monterey No. 0 sand to study the effect of cementation on the dynamic properties at low strains. The resonant column device used was modified for cemented sands in order to increase the overall stiffness of the apparatus. All the specimens were prepared using undercompaction (Ladd, 1978). The maximum shear modulus of cemented sand, G^*_{max}, is expressed as

$$G^*_{max} = G_{max} + \Delta G_{max} \tag{2.11}$$

where G_{max} is the maximum shear modulus of uncemented sand; ΔG_{max} is the increase in maximum shear modulus due to cementation. Based on regression analysis, when CC is less than 2%, ΔG_{max} can be obtained from

$$\frac{\Delta G_{max}}{P_a} = \frac{172}{(e - 0.5168)}(CC)^{0.88}(\frac{\overline{\sigma}_o}{P_a})^{(0.515e-0.13cc+0.285)} \tag{2.12}$$

and when CC is between 2 and 8%, ΔG_{max} is given by

$$\frac{\Delta G_{max}}{P_a} = \frac{773}{e}(CC)^{1.2}(\frac{\overline{\sigma}_o}{P_a})^{(0.698e-0.04cc-0.2)} \tag{2.13}$$

where CC is expressed as a percentage. The maximum shear modulus of uncemented Monterey No. 0 sand is given by

$$G_{max} = \frac{428.6}{(0.3 + 0.7e^2)}(P_a)^{0.426}(\overline{\sigma}_o)^{0.574} \tag{2.14}$$

Saxena, Avramidis and Reddy (1988) also discussed the results of resonant column tests on a cement-treated sand reported by Chiang and Chae (1972). The maximum shear modulus is expressed as

$$G^*_{max} = [G_{max} - 0.343CC(\overline{\sigma}_o)^{0.5}](\overline{\sigma}_o)^{0.06CC} \tag{2.15}$$

where, G^*_{max}, G_{max} and $\overline{\sigma}_o$ are expressed in psi (1 psi=6.89 kPa), and CC is in percent. To calculate G_{max}, relations proposed by Hardin and Drnevich (1972a, b) were recommended. For round-grained sand,

$$G_{max} = 2630\frac{(2.17 - e)^2}{(1 + e)}(\overline{\sigma}_o)^{0.5} \tag{2.16}$$

where G_{max} and $\overline{\sigma}_o$ are in psi.

Another study of low strain shear modulus of cemented sands was reported by Chang and Woods (1992) and the degree of cementation was expressed in terms of percent void filled with cementation. The test results indicate that the shear modulus of cemented sand is related to the degree of cementation and the properties of the sand. It is concluded that the effective grain size (D_{10}) and coefficient of uniformity (C_u) are the most important index properties controlling the cementation effect. The low strain shear modulus of cemented sand increases with increasing C_u and decreases with increasing D_{10}.

The resonant column technique was widely used to evaluate the dynamic properties of cemented sands at low strain. Because resonant column tests are costly and complicated, correlations of dynamic moduli from static strength parameters is very valuable for a crude estimation of dynamic properties of cemented sands.

Saxena, Avramidis and Reddy (1988) correlated the maximum shear modulus from the static triaxial strength of cemented Monterey No. 0 sand. When the effective confining pressure is 49 kPa, the relation is given by

$$G^*_{max} = 1109.22(\sigma_d/P_a) + 72.47 \tag{2.17}$$

where σ_d is the deviator stress at 1% axial strain in static triaxial drained test.

Saxena, Avramidis and Reddy (1988) also used another correlation obtained by Chiang and Chae (1972) by conducting static undrained triaxial tests at a confining pressure of 20 psi (138 kPa). The relation is expressed as

$$G^*_{max} = 13.867 + 0.419\sigma_d \tag{2.18}$$

where σ_d is the deviator stress at 1% strain level in psi (1 psi = 6.89 kPa) and G^*_{max} is in ksi (1 ksi = 6.89 MPa).

2.2.2 Cyclic strength and liquefaction potential

The liquefaction potential and cyclic strength of uncemented sands are mainly influenced by sand density and confining pressure. For cemented sands, the resistance to liquefaction is significantly increased and only a small amount of cement is required to prevent liquefaction (Dupas and Pecker, 1979).

Saxena, Reddy and Avramidis (1988) used an artificially cemented Monterey No.0 sand in stress-controlled cyclic triaxial tests. The cement contents were 1, 2, 5, and 8%, and the confining pressure was 98 kPa. The initial tests indicated that samples with 5 and 8% cement were not susceptable to liquefaction, and further studies were carried out on the samples with 1 and 2% cement. Dupas and Pecker (1979) also found that sand samples with 5% of cement are not liquefiable in laboratory tests.

Test results of Saxena, Reddy and Avramidis (1988) have indicated that cyclic strength and liquefaction resistance of cemented sand increases significantly with increasing cement content. The behaviour of cemented loose sand is similar to that of uncemented dense sand. Increase of density and curing period leads to the increase in liquefaction resistance. The relationship between ratio of cyclic deviator stress to confining pressure, SR, and the number of cycles to liquefaction, N_l, can be expressed as

$$SR = a(N_l)^b \tag{2.19}$$

where a and b are constant parameters depending on sand density and cement content.

From the test results, Saxena, Reddy and Avramidis (1988) also found good correlation of cyclic strength from maximum shear modulus of the cemented sand. The correlations make it possible to estimate the cyclic strength of cemented sand from the measured maximum shear modulus or shear wave velocity, although the correlation is dependent on the type of sand, degree of cementation and effective confining pressure.

Another study of the influence of cementation on liquefaction potential of cemented sand was presented by Clough *et al.* (1989). They used both triaxial and cubical cyclic shear devices to investigate the effects of cementation, unit weight and stress path on liquefaction of cemented sands. The test results were used to separate the effects of cementation and unit weight. It was found that increases in cementation and unit weight lead to an increase in liquefaction resistance. When cementation reaches a critical value, it trends to override the effects of unit weight.

Cyclic triaxial shear tests of undrained cemented sands were also conducted by Frydman *et al.* (1980) and Dupas and Pecker (1979). The results indicate that cementation can considerably increase the cyclic strength and the liquefaction resistance.

2.2.3 Dynamic damping

Based on resonant column tests on cemented Monterey No. 0 sand, Acar and El-Tahir (1986) obtained quantative assessment of dynamic shear damping ratios. It was concluded that an increase in cementation leads to a decrease in damping ratio at all strain levels.

However, the test results of Saxena, Avramidis and Reddy (1988) are quite different. They also used Monterey No. 0 sand in resonant column tests to study the behaviour of dynamic damping and expressed shear damping ratio of cemented sand as

$$D_s^* = D_s + \Delta D_s \tag{2.20}$$

where D_s is the shear damping ratio of uncemented sand, and ΔD_s is the change of damping ratio due to cementation. D_s is related to confining pressure and shear strain, and is expressed as

$$D_s = 9.22(\frac{\overline{\sigma}_o}{P_a})^{-0.38}(\gamma)^{0.33} \tag{2.21}$$

in which γ is the amplitude of shear strain. Damping value of cemented sand also depends on strain level. When the shear strain (γ) is approximately equal to $10^{-3}\%$, based on test results, ΔD_s at the lower range of cementation is given by

$$\Delta D_s = 0.49(CC)^{1.07}(\frac{\overline{\sigma}_o}{P_a})^{-0.36} \tag{2.22}$$

Saxena, Avramidis and Reddy (1988) have emphasized that the above equation for damping is only available for lower ranges of cementation, as decreases in shear damping ratio were observed at higher ranges of cementation. Although they were not able to find the exact threshold of cement content at which the shear damping ratio reaches the maximum value, their tests data have indicated that the above equation is suitable for cement contents at least up to 5%. It is also stated that the relation is in agreement with the experimental results of Chiang and Chae (1972).

2.3 Cemented Sands in Offshore Engineering

Noorany (1989) classified marine sediments into three major groups: terrigenous; biogenous; and hydrogenous. Biogenous sediments cover about one half of the continental shelves and they are primarily calcareous or siliceous sediments. Calcareous deposits also exist in hydrogenous sediments due to the precipitation of calcium carbonates. It is recognized that a large portion of continental shelves is covered with calcareous sands containing calcium carbonates (Noorany, 1989; Poulos, 1988; and Murff, 1987).

Calcareous sands originate from biological precesses or chemical precipitations (Noorany, 1989; Poulos, 1988; and Murff, 1987). One important property of calcareous sands is the carbonate content. Although there is no explicit relationship between the carbonate content and engineering properties, it is recognized that soils with carbonate content greater than 50% are often troublesome (Murff, 1987). An important feature of calcareous sands is the presence of cementation bonds among soil particles (Poulos, 1988) and calcareous sands without cementation bonds are uncommon (Murff, 1987). The cementation usually results from the precipitation of calcite cement (Saxena and Lastrico, 1978). The assessment of cementation in the brittle and crushable calcareous sands is very difficult. Sampling for laboratory tests, especially by offshore percussion, may clearly destroy the cementation bonds, and sample trimming also results in additional disturbance (Murff, 1987; Saxena and Lastrico, 1978; and Frydman *et al.*, 1980). In situ standard penetration tests by Frydman *et al.* (1980), Angemeer *et al.* (1973) and Hagenaar (1982) and in situ cone penetration tests have also brought confusing interpretations (Murff, 1987), although laboratory cone penetration test of artificially cemented sand (Rad and Tumay, 1986) has indicated that the tip resistance and the sleeve friction increase with increasing cement content.

Calcareous sands are often troublesome to the construction of offshore facilities, especially for driven piles. Extensive investigations have indicated that driven piles in calcareous sands develop much lower skin friction than in silica sands (Angemeer *et al.*, 1973; Datta *et al.*, 1980; Dutt and Cheng, 1984; Ismael, 1989; McClelland, 1974; Noorany, 1985; Nauroy and LeTirant, 1985; and Poulos and Chua, 1985). The skin friction is typically only 20-25% of that used for silica sands (Murff, 1987). Although the understanding is limited, there is a consensus that the lack of lateral stress is the main cause of the low skin friction (Angemeer *et al.*, 1973; Noorany, 1985; and Murff, 1987). The low lateral stress results from the high compressibility and the presence of cementation of calcareous sands. Nauroy and LeTirant (1985) have indicated that shaft resistance decreases with increasing compressibility of soils. However, this result seems to fit best to non-cemented soils which are not typical for calcareous sands (Murff, 1987). Many investigators agree that cementation is very important to the shaft resistance (Angemeer *et al.*, 1973; Datta *et al.*, 1980; Dutt and Cheng, 1985; Hagenaar, 1982; and Nauroy and LeTirant, 1985), but the mechanism remains unclear. It is suggested that well-cemented calcareous sands develop high skin friction, while lightly, or irregularly cemented sands result in low shaft resistance, as lightly cemented calcareous sands have more open structures (Murff, 1987). However, due to the difficulty to assess the degree and distribution of cementation in calcareous sands, the understanding of the effect of cementation on the skin friction of driven piles is still limited.

2.4 Closure

Extensive investigations of the behaviour of cemented sands have been carried out in the past decades. The effect of cementation is very prominent in the behaviour of cemented sands at low strain or low stress. The parameters of cemented sands at low strain, such as the maximum shear modulus and cyclic strength, are much greater than those of uncemented sand. However, at high strain, cementation bonds are broken, and at high pressure internal friction in more important than the cementation cohesion. Due to the difficulty of sampling and the uncertainty of the cementation in naturally cemented sands, much work has been done in laboratories with artificially cemented sands. For offshore engineering, due to the difficulty to assess the degree of cementation, the influence of cementation of calcareous sands on the behaviour of piles is still unclear.

To provide more insight to the behaviour of cemented sands, the following sections of this thesis will present an experimental study of the lateral stress in two artificially cemented sands under K_o conditions.

Chapter 3

Experimental Program

The purpose of this thesis is to study the behaviour of lateral stress in cemented sands under K_o conditions. To do so, a modified oedometer ring was developed to measure the at rest lateral stress in cemented and uncemented sands in order to investigate the effect of degree of cementation. Materials used were No. 3 Ottawa sand and a marine sand with type 10 portland cement. Sand specimens were prepared by the method of undercompaction using 0, 0.5, 1.0, 2.0, 4.0 and 8.0% of cement by the weight of dry sands. All specimens were prepared at a density index of 50% and were cured for 10 days before tests were carried out, except for those used to investigate the effects of sand density and curing period.

3.1 Measurement of Lateral Stress

Devices for measuring lateral stress under K_o conditions can be divided into two main categories (Ofer, 1981); one uses triaxial cells, and the other employs modified oedometer rings. In some cases, simple shear devices were also used (Budhu, 1985; and Youd and Craven, 1975). Bishop and Eldin (1953) introduced at rest lateral stress measurement in a triaxial cell. Using this technique, it is possible to maintain the lateral strain of the soil specimen at zero by adjusting the cell pressure. Similar triaxial devices were developed by Boyce and Brown (1976), El Ruwayil (1976), Campanella and Vaid (1972), Andrawes and El-Sohby (1973), Kochi and Tatsuoka (1984), Feda (1984), Lo and Chu (1991), and Tsuchida and Kikuchi (1991).

Modified oedometer rings for measuring lateral stress in soils can be divided into two sub-categories: thin wall oedometer rings and thick wall oedometer rings. The first type of rings were used by Brooker and Ireland (1965), Calhoun and Triandafilidis (1969), Edil and Dhowian (1981), and Ofer (1981). Using this technique, strain gauges are cemented to the oedometer ring to measure the lateral stress which causes small lateral deformation of the ring. Thick wall oedometer rings were used by Abdelhamid and Krizek (1976) and Thomann and Hryciw (1990), in which small horizontal holes were made in the midheight of the ring walls through which pistons were used to measure the lateral stress in the soils of zero lateral deformation.

It is believed that lateral stress under true K_o conditions can be measured in both the triaxial cells and thick wall oedometer rings. However, the devices and measurement systems and the test procedures are complicated for the adjustment of zero lateral strain. For thin wall oedometer ring, lateral deformation may affect the test results. However, as described later in this chapter, the effect of lateral deformation of thin wall oedometer ring on the measurement of lateral stress is not significant. Because of the convenience of operation, a thin wall oedometer was used in the tests described in this thesis, as shown in Figure 3.1.

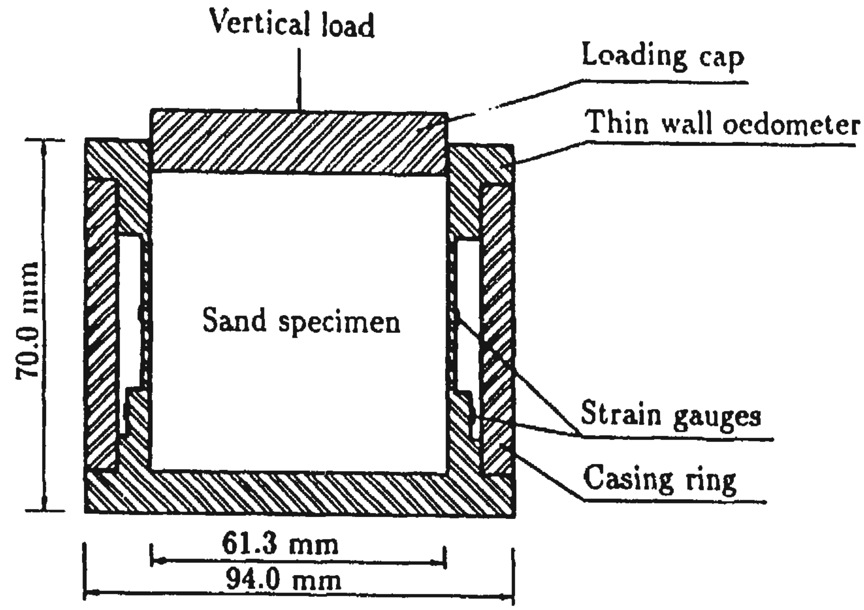
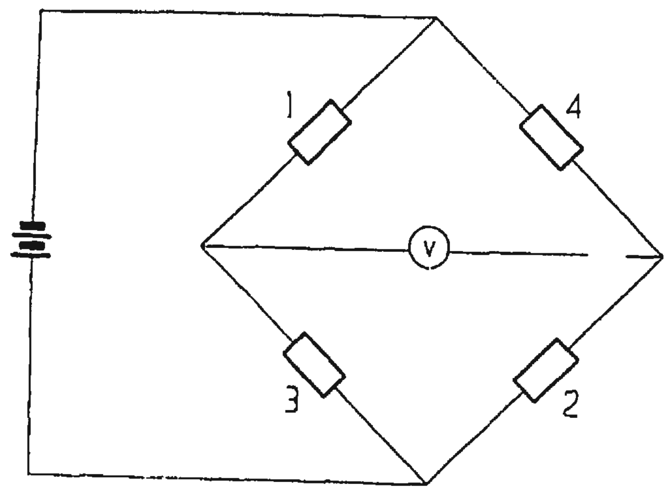


Figure 3.1: Device for Lateral Stress Measurement

The oedometer ring has an inside diameter of 61.3 mm, an outside diameter of 94.0 mm and a height of 70.0 mm. The thickness of the thin wall is 1.5 mm. Strain gauges were cemented on the thin wall to measure the hoop strain of the ring caused by the lateral stress in the sand specimen. Measurements were carried out using a strain indicator.

3.1.1 Arrangement of Strain Gauges

Soil lateral stress causes the thin wall of the oedometer ring shown in Figure 3.1 to deform laterally. By measuring the lateral deformation of the thin wall, lateral stress in the soil can be obtained. Electrical resistance strain gauges were cemented to the thin wall to measure thin wall hoop strain induced by the soil lateral stress. The strain gauges were of foil type. The gauge resistance was 120±0.15%



1,2: active gauges   3,4: compensation gauges

Figure 3.2: Wheatstone-bridge Circuit for Strain Measurement

ohms, and the gauge factor was 2.045±0.10%. Two strain gauges were horizontally cemented 180° apart at the midheight of the thin wall to measure the hoop strain induced by soil lateral st ·ess. These two strain gauges were called active gauges. In addition, two other strain gauges providing temperature compensation were bound to the bottom part of the oedometer ring where no measurable deformation occurred during tests. The two active gauges and the temperature compensation gauges were connected together to form a full Wheatstone-bridge circuit, as shown in Figure 3.2.

This arrangement has two advantages. The strain sensed by each active strain gauge is added together and the accuracy of lateral stress measurement is enhanced. The second advantage is that it can provide temperature compensation with which changes of temperature in the gauges resulting from environmental factors or exiting electrical current through the gauges, or both, will not affect the results of the measurement. Temperature compensation is very important to obtain stable and accurate measurements.

Careful attention was paid to the bonding of the strain gauges, for the proper functioning of a strain gauge depends completely on the bond between the gauge and the structure undergoing tests (Perry and Lissner, 1961). The surface where a strain gauge would be bound was rubbed with sand paper and cleaned using an acid and then using a neutral liquid before bonding the strain gauge.

3.1.2 Effect of lateral deformation

The thin wall thickness of 1.5 mm of the oedometer was chosen by the criteria that the wall should be thin enough to achieve reliable measurement results and be thick enough to satisfy the K_o conditions. One of the main disadvantages using a thin wall oedometer ring is that deformation of the oedometer necessary for the measurement of lateral stress may influence the test results. It is necessary to investigate if the slight lateral deformation will significantly influence the result of lateral stress measurement.

Ofer (1981) constructed a oedometer ring which could measure lateral stress either with or without lateral deformation. The test results show that under true K_o conditions of zero lateral strain, lateral stress is higher than that when a small lateral strain occurs. However, the conclusions of other investigators are different. Calhoun and Triandafilidis (1969) used several rings with different wall thickness to study the effects of lateral deformation and concluded that lateral stresses are influenced by small lateral strain but not significantly. Tests results by Andrawes and El-Sohby (1973) using a triaxial cell show that small deviation of lateral strain from zero will not significantly affect the lateral stress measurement. In the test of Ofer (1981), the thickness of the thin wall was only 0.8 mm, which may be the main cause for the lateral stress measured with lateral strain being smaller than that when no lateral deformation was allowed. Another questionable aspect of the results of Ofer (1981) is that the value of K_o both with or without lateral deformation decreased significantly with increased vertical stress, especially in the case of zero lateral deformation when the vertical stress was less than 300 kPa. The value of K_o decreased approximately from 0.7 to 0.45 when the vertical stress changed from 50 to 300 kPa. It is well known that K_o of soils remains constant during loading (Mayne and Kulhawy, 1982).

For the new oedometer ring, the thickness of the thin wall is 1.5 mm. According to actual measurement, within the applied lateral stress in the tests, the lateral strain in the soil caused by the lateral deformation of the oedometer was less than the order of 10^{-5}. This strain is significantly less than the value required for the lateral stress to drop to the active state (Edil and Dhowian, 1981).

3.1.3 Calibration

The oedometer ring was calibrated by applying a known air pressure to the inside of the oedometer and the output of the strain gauge system was read. The relationship between the chamber air pressure and the output reading was linear. No hysteresis effects were observed during loading and unloading.

3.2 Test Materials

Tests were performed on No. 3 Ottawa sand and a marine sand. The main properties of the two sands are summarized on Table 3.1 and the grain size distribution curves are shown in Figure 3.3. The cement agent was type 10 portland cement.

Table 3.1: Soil Properties

Sand Type		No. 3 Ottawa	Marine
Specific Gravity:	1	2.65	2.65
Maximum Dry Density:	kg/m^3	1780	1750
Minimum Dry Density:	kg/m^3	1530	1490
Maximum Dry Unit Weight:	kN/m^3	17.64	17.17
Minimum Dry Unit Weight:	kN/m^3	15.01	14.62
Maximum Void Ratio:	1	0.73	0.88
Minimum Void Ratio:	1	0.49	0.55
Mean Grain Size, D_{50}:	mm	0.54	0.33
Effective Grain Size, D_{10}:	mm	0.42	0.20
Limited Grain Size, D_{60}:	mm	0.52	0.34
Uniformity Coefficient, C_u:	1	1.24	1.70

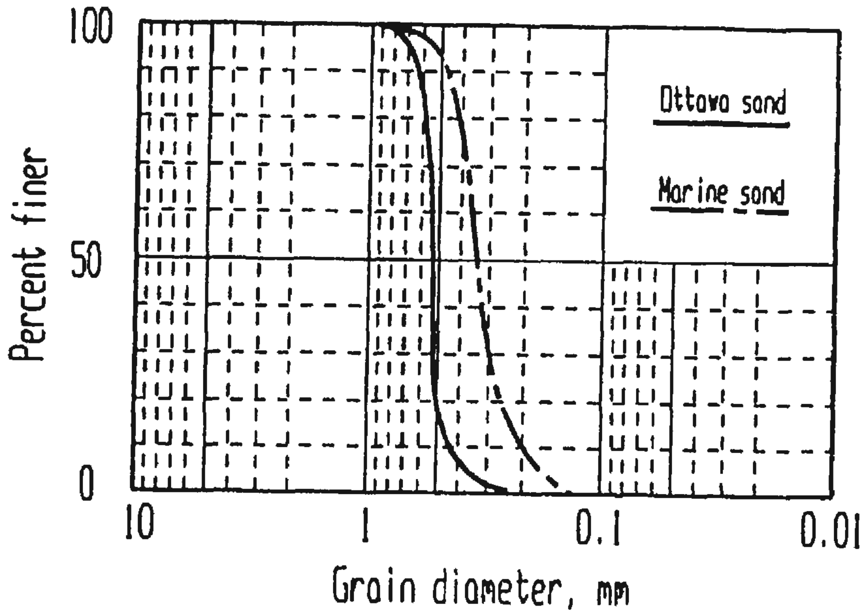


Figure 3.3: Grain Size Distribution

The two sands were used to study the effect of soil type on the lateral stress in cemented sands. Both were silica sand. Some physical properties of the two sands are different. The Ottawa sand is coarser in particle size (D_{50} = 0.54) and more uniform in grain distribution (C_u = 1.24). The marine sand is a medium-fine sand. The mean diameter (D_{50}) is 0.33 mm, the uniformity coefficient (C_u) is 1.70. Another difference between the two sands is that the Ottawa sand is round-grained while the shape of the marine sand grains is more angular.

The maximum and the minimum dry densities of the two sands were determined using a mold of 1000 cm^3 in volume. The minimum dry density was obtained by placing a tube in the mold, filling the tube with sand, and carefully withdrawing the tube (Bowles, 1986). To obtain the maximum dry density, the sand was put in the mold, a load of about 150 N was applied to the confining plate on the top of the sand according to the recommendation of Bowles (1986). The mold was then vibrated on a shaking table. Different levels of acceleration of vibrations were applied to achieve the maximum density.

3.3 Specimen Preparation

There are two main methods for cemented sand specimen preparation: pluviation (Acar and El-Tahir, 1986; and Clough *et al.*, 1989) and compaction (Saxena, Avramidis and Reddy, 1988; and Clough *et al.*, 1981). Using the pluvial method, the sand mixed with cement is pluviated to the specimen mold, and then the mold is tapped along its length to obtain the sand density required (Acar and El-Tahir, 1986). Two significant problems associated with this technique are the segregation of particles and the difficulty of obtaining uniform density specimens (Ladd, 1978). For cemented sands, great care should be taken to prevent segregation of cement and sand particles during pluviation (Acar and Tahir, 1986). To minimize the influence of specimen preparation on test results, all specimens described in this thesis were prepared using undercompaction.

The technique of preparing test specimens using undercompaction was introduced by Ladd (1978), initially for cyclic triaxial tests. It is well known that when a sand is compacted in layers, the compaction of the succeeding layer can further densify the soil under it. The method of undercompaction uses this fact and applies the concept of undercompaction to achieve specimens of uniform desity. Using this method, a soil layer is compacted to a lower density than the final required value by a prescribed amount which is defined as degree of undercompaction, U. The first layer (bottom) has the maximum value of U, and the U value of the last layer (top) is (usually) zero. For the ith layer, the degree of undercompaction is given by

$$U_i = U_1 \left(1 - \frac{i-1}{N_t - 1}\right) \tag{3.1}$$

where U_i is the degree of undercompaction of ith layer, U_1 is the degree of undercompaction of the first layer, and N_t is the total number of layers. Once the N_t value is chosen, U_1 is the only parameter to be selected. The height of specimens in the oedometer ring shown in Figure 3.1 is 52.0 mm, so the specimens were compacted in four layers ($N_t = 4$) in which each layer was 13.0 mm in thickness. With a proper selection of U_1 value, the specimen prepared could have a uniform density. The U_1 value is related to the density of specimen. The looser the specimen, the greater the U_1 value.

To date, there is no effective method for selecting the U_1 value of cemented sands. According to the experiences of Ladd (1978) and Saxena, Avramidis and Reddy (1988), for tests described in this thesis, U_1 values of 14% and zero are chosen for specimens with density index of 20% and 90% respectively. Linear change of U_1 value between 14% and zero is selected for specimens with density index betweeen 20% and 90%, that is

$$U_1 = 14\left(1 - \frac{I_D - 20}{70}\right) \tag{3.2}$$

where U_1 is the degree of undercompaction of a specimen with a density index of I_D. Both U_1 and I_D are in percent. For example, a specimen with a density index of 50% will have a U_1 value of 8%. The procedure for specimen preparation of cemented sands is summarized as follows.

(1) Determine the total weight of air-dried sand according to the density requirement of the specimen. Divide the sand into four equal groups and put them in four containers.

(2) For each container, weigh the amount of type 10 portland cement according to the required cement content by the weight of air-dried sand. Mix the sand with the cement throughly using a spatula.

(3) Add 4% of water to each container by the weight of the sand and the cement. Mix them throughly to obtain a uniform sand-cement-water mixture. Although Clough *et al.* (1981) and Saxena, Avramidis and Reddy (1988) used 8% of water for the specimen preparation of cemented sands, it was found that using 4% of water was more convenient for the specimen preparation. Actually, 4% of water could keep the specimens in a very damp state in order that there was enough moisture for the hydration of the cement.

(4) Put the sand-cement-water mixture from one of the containers into the oedometer ring and compact it to the prescribed degree of undercompaction. A very thin layer of grease was applied to the inside surface of the oedometer ring to prevent cementation between the sand mixture and the ring. The height of first layer before the second layer is compacted is given by

$$H_1 = \frac{H}{N_t}(1 + \frac{U_1}{100}) \tag{3.3}$$

where U_1 is obtained using equation (3.2).

(5) Compact the following layers one by one using a similar method as for step

(4). The height of the specimen at the top of the ith layer is calculated by

$$H_i = \frac{H}{N_t}(i + \frac{U_i}{100}) \tag{3.4}$$

where H is the height of the specimen, N_t is the number of layers ($N_t = 4$ here), and U_i is the degree of undercompaction of the ith layer calculated using equation (3.1).

(6) After the last layer (top) is compacted, put the loading cap (see Figure 3.1) on the specimen in the oedometer. Make sure the loading cap fits well with the oedometer ring. Seal the small gap between the cap and the oedometer ring with grease.

(7) Cure the specimen for a required period (under temperature of $22 \pm 2°C$).

Chapter 4

Experimental Results and Analyses

In the past decades, much work has been done on the investigations of lateral stress in uncemented sands (Brooker and Ireland, 1965; Anreawes and El-Sohby, 1973; and Fukagawa and Ohta, 1988). The coefficient of lateral earth pressure at rest with zero lateral deformation is expressed by equation (1.1) as

$$K_o = \frac{\sigma_h}{\sigma_v}$$

where σ_h is the at rest lateral (or horizontal) stress and σ_v is the vertical stress.

The value of K_o usually remains constant during loading and increases during unloading with increasing OCR (Mayne and Kulhawy, 1982). In the tests performed for this thesis, the at rest lateral stress in No. 3 Ottawa sand and a marine sand was measured using the oedometer ring shown in Figure 3.1. The results of the lateral stress and K_o value of the two uncemented sands with a density index of 50% are shown in Figure 4.1 and Figure 4.2. During loading, K_o of the two sands remains constant, that is, the value of K_o is not affected by the vertical stress. However, during unloading, K_o increases with decreasing vertical stress and

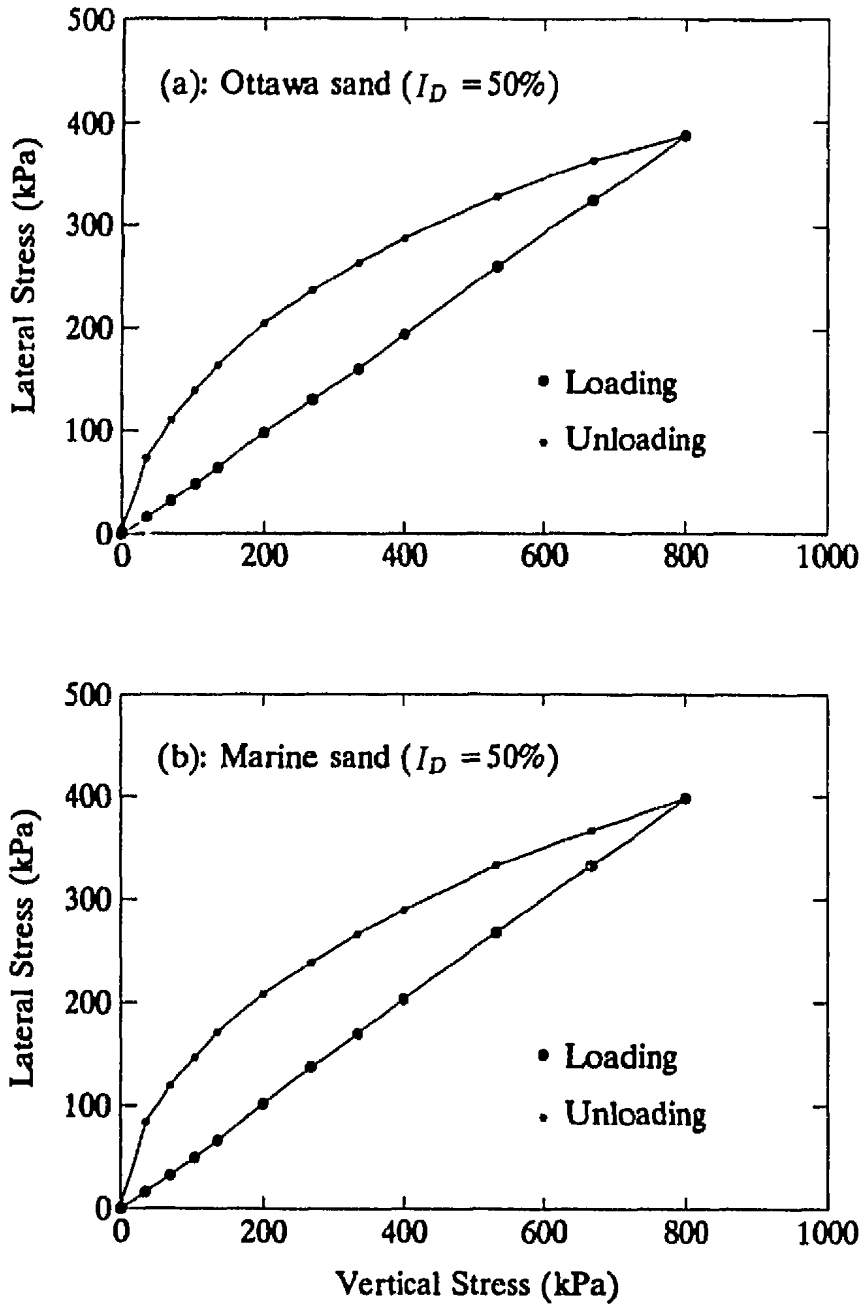


Figure 4.1: Lateral Stress in Uncemented sands ($I_D = 50\%$)

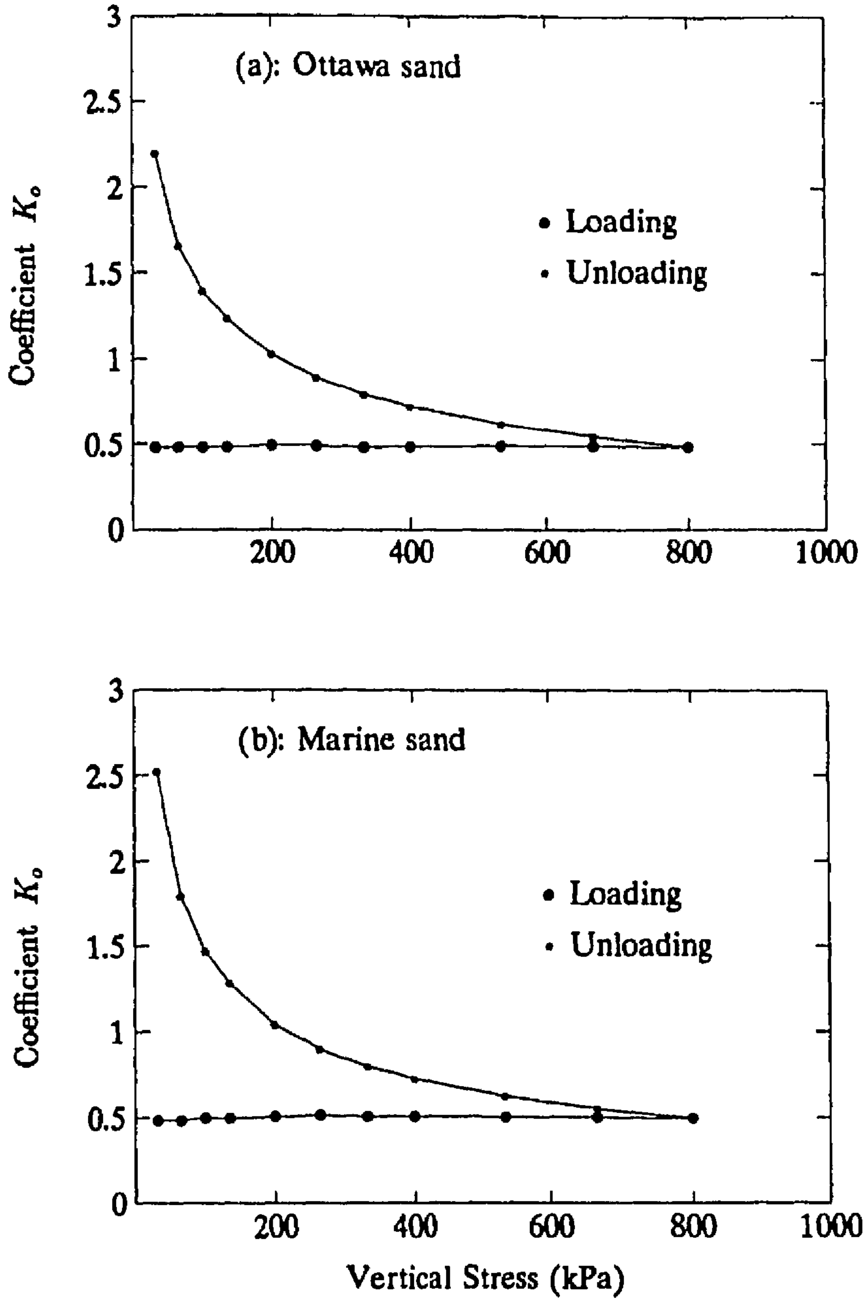


Figure 4.2: K_o of Uncemented Sands ($I_D = 50\%$)

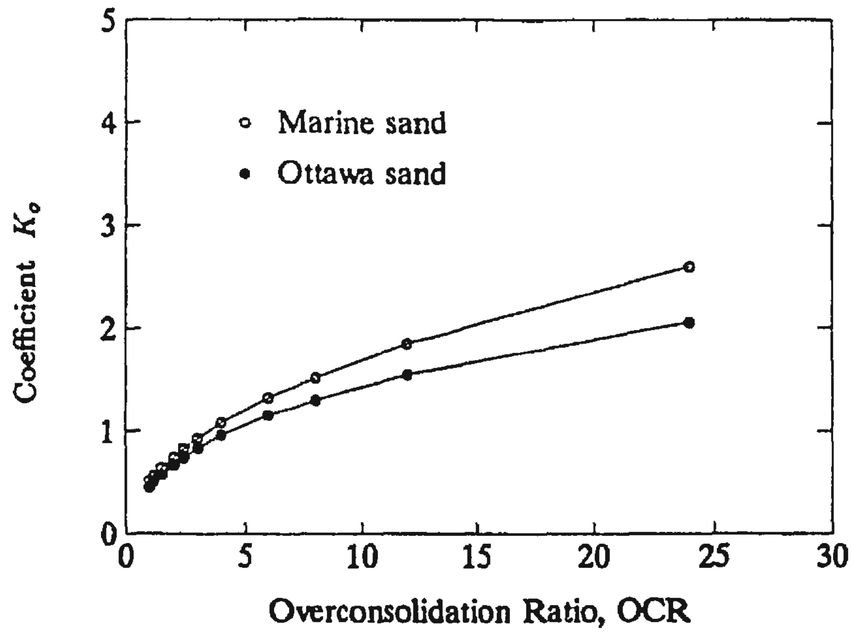


Figure 4.3: Relation between K_o and OCR (Uncemented Sands, $I_D = 50\%$)

increasing overconsolidation ratio (OCR). The relationships between OCR and K_o of the two sands are shown in Figure 4.3.

K_o of normally consolidated sand decreases with increasing density index. This behaviour was clearly demonstrated by Bishop and Eldin (1953) in tests using a triaxial cell. For the Ottawa sand and the marine sand tested using the oedometer ring shown in Figure 3.1, the relationships between K_o and I_D are shown in Figure 4.4. K_o of the Ottawa sand is a little smaller than that of the marine sand.

Although extensive laboratory investigations of at rest lateral stress in uncemented sands have been done, little information on the lateral stress in cemented sand has been obtained. Therefore, tests on cemented Ottawa sand and marine

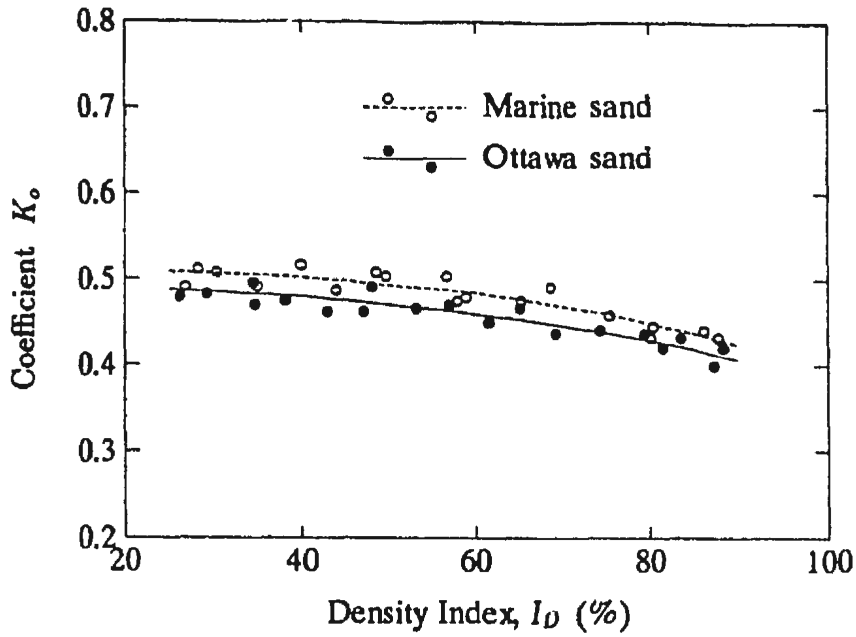


Figure 4.4: Relation between K_o and I_D of Uncemented Sands

sand were performed using the oedometer ring to measure the lateral stress. From the test results, it is found that the behaviour of lateral stress in cemented sands are quite different from those of uncemented sands. The details are described in the following sections.

4.1 Effect of Cement Content

To investigate the influence of cement content on lateral stress in sands, No. 3 Ottawa sand and the marine sand were tested at a density index of 50%. Specimens were prepared using 0, 0.5, 1.0, 2.0, 4.0, and 8.0% of portland cement by weight of dry sand. The curing period was 10 days. The relationships between lateral stress and vertical stress are shown in Figure 4.5.

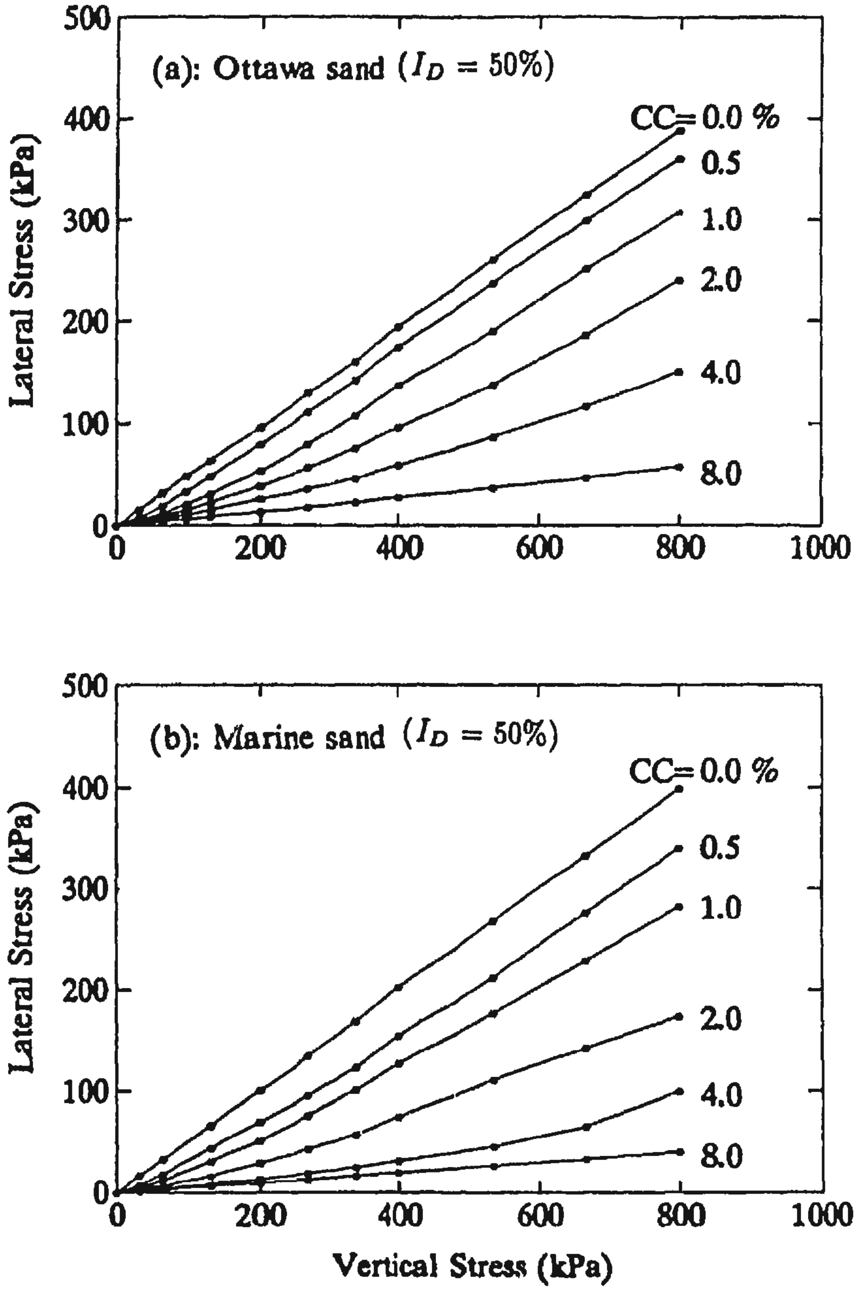


Figure 4.5: Relations between Lateral Stress, σ_h, and Vertical Stress, σ_v

The common characteristic of both cemented sands is that the lateral stress decreases significantly with increasing cement content. It can also be seen that at the same cement content, lateral stress in the marine sand is smaller than that in the Ottawa sand; that is, cementation is more effective in reducing lateral stress in the marine sand than in the Ottawa sand. The effect of cementation on the lateral stress depends on the type of sand. As described in chapter 3, the physical properties of the two sands are different. The Ottawa sand is coarser in grain size (D_{50} =0.54) and more uniform in grain distribution (C_u =1.24). The marine sand is a medium-fine sand, the mean diameter (D_{50}) is 0.33 mm, the uniformity coefficient (C_u) is 1.70. Another difference between the two sands is that the Ottawa sand is round-grained while the marine sand grains are more angular. It is believed that the differences in the grain size and the angularity are the main causes of the difference of the cementation effect on lateral stress in the two sands. It is understandable that the marine sand with finer particle size and wider range of grain size has more surface area for particle contact and the cementation among the sand particles is stronger. In addition, the cementation in the marine sand is more effective due to the more effective interlocking of its angular particles (Clough *et al.*, 1981). The test results are in accordance with the fact that cementation in sands increases with increasing C_u and decreases with increasing D_{10} (Chang and Woods, 1992).

An important characteristic of lateral stress in the cemented sands is that the relationship between lateral stress and vertical stress is nonlinear, as shown in Figure 4.5. This characteristic is more clearly demonstrated in Figure 4.6 by the values of K_o derived from the results shown in Figure 4.5. K_o of the cemented sands is

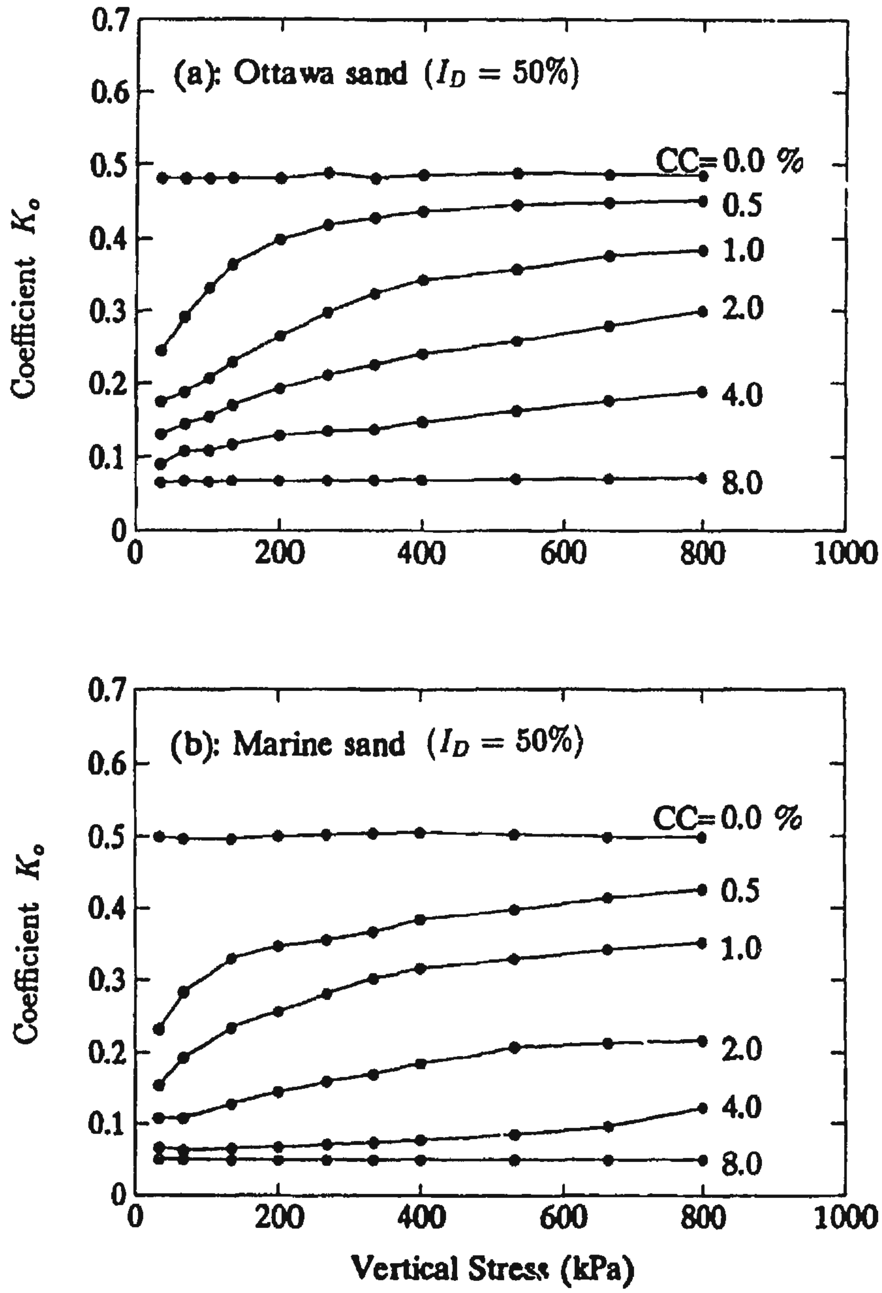


Figure 4.6: Relations between K_o and Vertical Stress, σ_v

influenced not only by cement content, but also by vertical stress. In the study of the dynamic properties of cemented sand, Saxena, Avramidis and Reddy (1988) treated the cement content of 2% as a critical value to distinguish the different behaviour between highly and weakly cemented sand. This critical cement content of 2% seems also applicable to the lateral stress differences in cemented sands, shown in Figure 4.5 and Figure 4.6. In the range of applied vertical stress, the lateral stress and K_o of both sands with cement content greater than 2% are less influenced by vertical stress. For the weakly cemented sands with cement content less than 2%, lateral stress and K_o are significantly influenced by vertical stress, especially at low stress level. The higher the vertical stress, the higher the K_o value. The increase in K_o with vertical stress indicates that the effect of cementation is weakened with increasing vertical stress, which is certainly due to the breaking of cementation bonds among soil particles.

In the cemented sand under K_o conditions, shear stress can be expressed as

$$\tau = \frac{1}{2}(\sigma_v - \sigma_h) \tag{4.1}$$

where σ_v is vertical stress and σ_h is lateral stress. Using equation (1.1), the above equation becomes

$$\tau = \frac{1}{2}(1 - K_o)\sigma_v \tag{4.2}$$

The shear stress which resulted in shear strain in the sands should be the main cause of the breaking of cementation bonds (Clough *et al.*, 1981; and Li and Mitchell, 1988). In addition to the shear stress, high confining pressure may also destroy the cementation bonds (Saxena and Lastrico, 1978).

It should be noted that the effect of vertical stress on K_o depends on the cement content. For weakly cemented sand at low vertical stress, K_o increases rapidly with increasing vertical stress, σ_v. It can be seen from Figure 4.6 that for the sands with cement contents of 0.5% and 1.0%, K_o is less influenced by vertical stress when σ_v is greater than approximately 100 kPa and 200 kPa respectively.

The stress-dependence of the value of K_o and the elimination of cementation bonds can also be illustrated by the deformation behaviour of the Ottawa sand shown in Figure 4.7 and Figure 4.8.

It can be seen from Figure 4 7 that the vertical strain decreases rapidly .vith increasing cement content. The vertical deformation modulus shown in Figure 4.8 is derived from the data shown in Figure 4.5(a) and Figure 4.7 using

$$E = \frac{\sigma_v}{\varepsilon_v} \tag{4.3}$$

where σ_v is the vertical stress and ε_v is the vertical strain.

The deformation modulus increases with increasing cement content. It is also dependent on stress level. For the sand with zero cement content, the deformation modulus increases with vertical stress. For the weakly cemented Ottawa sand with cement contents of up to 2%, the deformation modulus decreases rapidly with vertical stress, especially at low stress levels. The rate of decrease in the deformation decreases with vertical stress.

Because the tested sands were under K_o conditions, vertical strain resulted in shear strain in the sands. It is widely recognized that shear strain leads to the

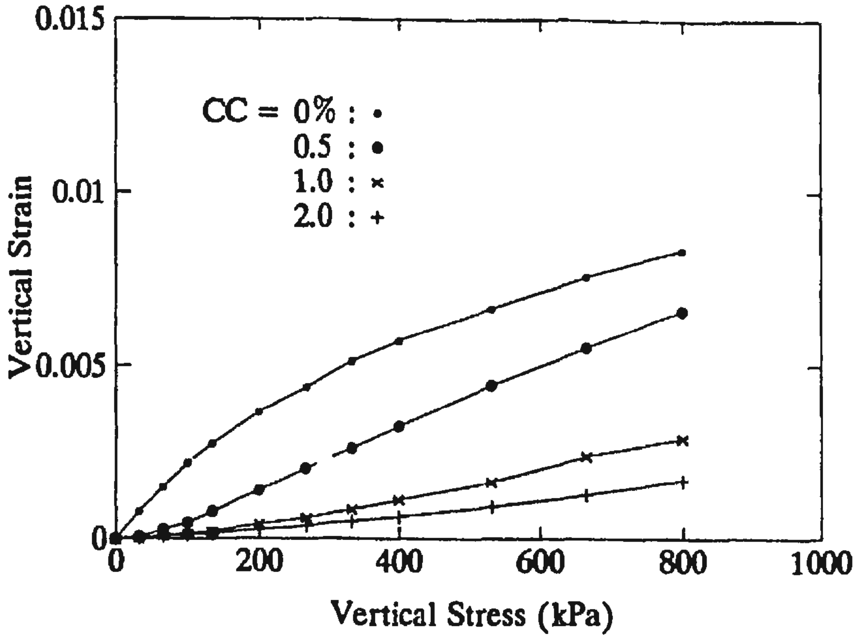


Figure 4.7: Vertical Strain of Ottawa Sand $(I_D = 50\%)$

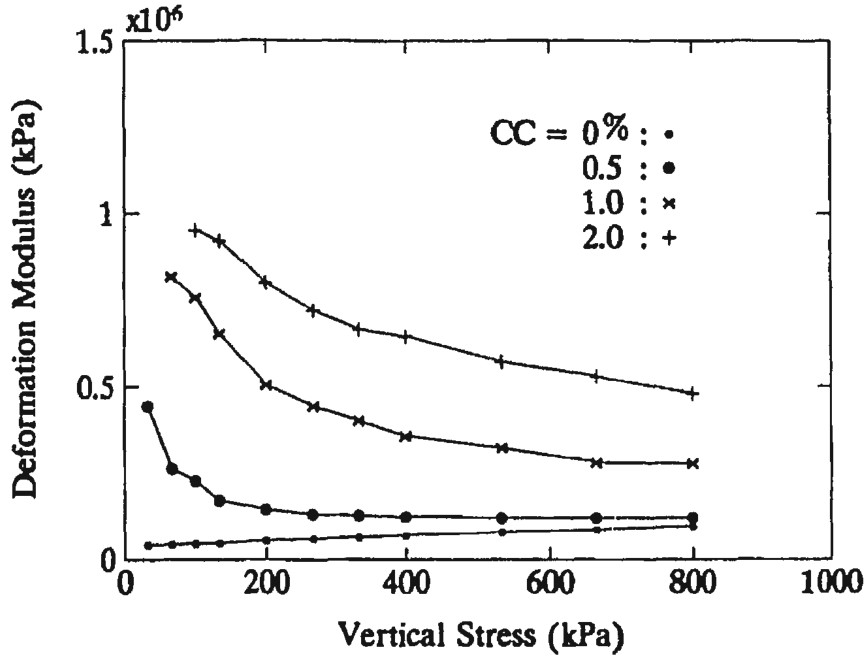


Figure 4.8: Vertical Deformation Modulus of Ottawa Sand $(I_D = 50\%)$

breaking of cementation bonds of cemented sands (Clough *et al.*, 1981; Saxena and Lastrico, 1978; and Li and Mitchell, 1988), and the cementation bonds are usually destroyed within the range of strain of 1%. For the cemented Ottawa sand, at the same strain level, the higher the cement content, the higher the vertical stress. For sand at low cement content, the strain leading to the breaking of cementation bonds was achieved at low vertical stress. It is obvious that the deformation modulus of the Ottawa sand with cement contents of 0.5% and 1.0% decreases significantly with vertical stress within the value of approximately 100 kPa and 200 kPa respectively. Therefore, the deformation behaviour shown in Figure 4.7 and Figure 4.8 is in accordance with the lateral stress behaviour shown in Figure 4.5(a) and Figure 4.6(a).

4.2 Effect of Stress History

In the previous section it has been shown that K_o of cemented sands decreases with cement content and increases with vertical stress. In addition, tests were performed to study the effect of stress history on lateral stress response of cemented sand. The specimens were prepared using No. 3 Ottawa sand with cemented contents of 0, 0.5, 1.0, 2.0, 4.0 and 8.0%. The density index of all specimens was 50%. Each specimen was cured for 10 days before the tests were performed. Lateral stress in each specimen was measured during the loading-unloading-reloading cycle. The specimens were fully unloaded, that is, after the vertical stress was applied to a required value, the specimen was vertically unloaded to zero and than reloaded. A typical result of the measured lateral stress as a function of vertical stress is shown in

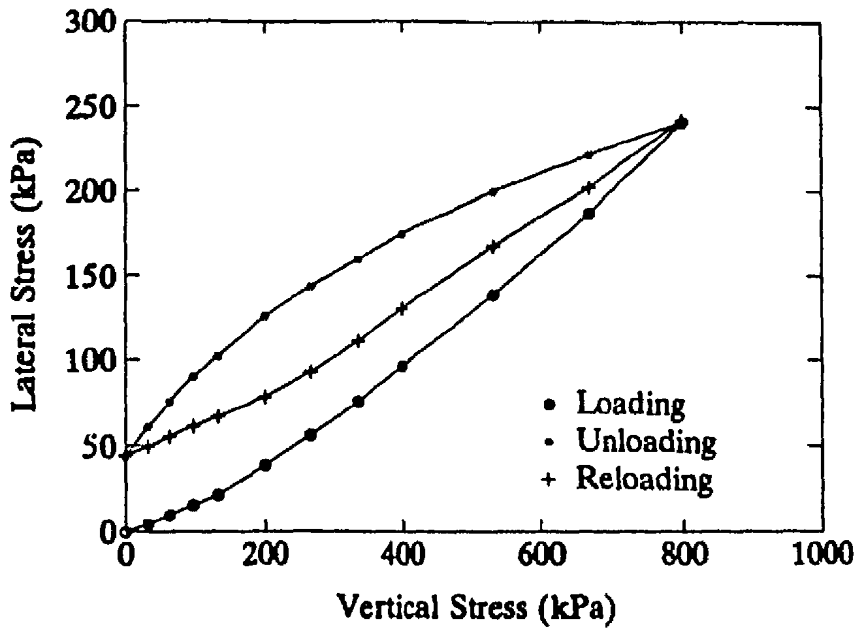


Figure 4.9: Lateral Stress in Ottawa Sand (I_D = 50%, CC = 2%)

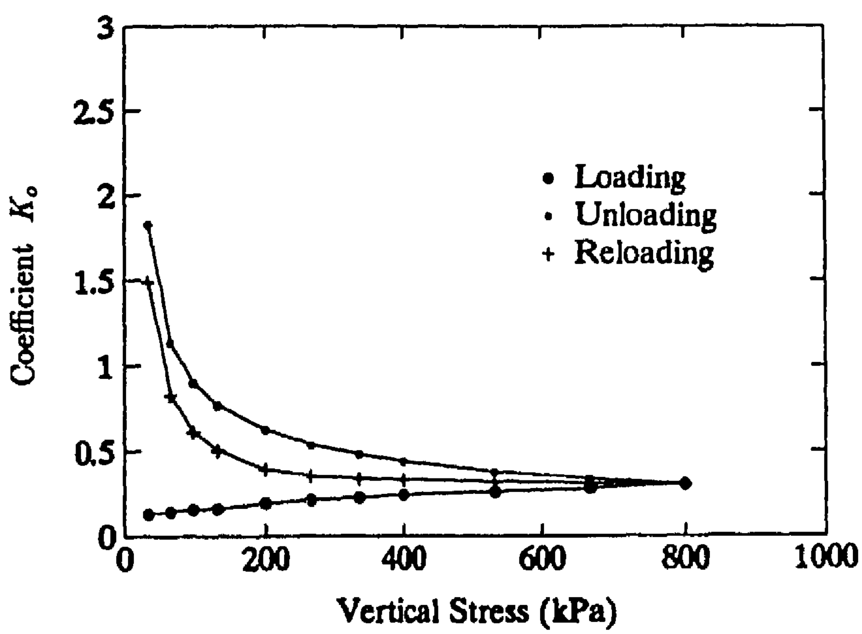


Figure 4.10: K_o of Ottawa Sand (I_D = 50%, CC = 2%)

Figure 4.9. The coefficient, K_o, during loading, unloading and reloading is shown in Figure 4.10. More complete test results for the Ottawa sand with different cement contents are shown in Figure A.1 to Figure A.24 in Appendix A.

Test results shown in Appendix A indicate that the effect of stress history on lateral stress in cemented sand is influenced by the cement content. For uncemented sand, the lateral stress is reduced to zero with the full unloading of vertical stress. However, there is a residual lateral stress in cemented sand after the full unloading of vertical stress. The residual stress depends on cement content and preconsolidation stress.

The stress path during reloading of cemented sand is different from that of uncemented sand. In uncemented sand, the lateral stress can be fully unloaded; there is no residual lateral stress after full unloading of vertical stress, as shown in Figure 4.1. The stress path during reloading is actually indentical to that during virgin loading. That is, the stress path of uncemented sand during reloading is not significantly affected by preconsolidation stress. However, there is a residual lateral stress in cemented sand after the full unloading of vertical stress; the stress path during reloading is affected by both cement content and vertical preconsolidation stress. During reloading, the stress path joins the virgin loading path when the vertical stress reaches the value of the preconsolidation stress.

The stress path of cemented sand during unloading is also influenced by cement content and preconsolidation stress. The effect of cementation on the residual lateral stress and the stress path during loading, unloading and reloading will be discussed in details in the following sections.

4.2.1 Residual lateral stress

Lateral stress in cemented sands increases with increasing vertical stress. When the vertical stress is reduced, the lateral stress decreases. Unlike uncemented sand which has no significant residual lateral stress after the full unloading of vertical stress, cemented sand exhibits a residual lateral stress after the vertical load removal. After full unloading of vertical stress on the sand, there is still a lateral stress in the cemented sand. The residual stress is influenced by vertical preconsolidation stress and the degree of cementation. The greater the preconsolidation stress, the greater the residual stress. The test results shown in Figure 4.11(a) indicates that the residual lateral stress increases with preconsolidation stress at all levels of cement content.

As shown in Figure 4.11(a), the effect of the degree of cementation on the residual lateral stress is complicated and is significantly influenced by the level of preconsolidation stress. At low cement content, the residual stress increases with cementation. However, with further increase in cementation, the residual stress begins to decrease, as shown in Figure 4.11(b). At a constant preconsolidation stress level, there is a critical cement content (CC_{cr}) at which the residual stress reaches the maximum value. When the cement content is smaller than CC_{cr}, the residual stress increases with cementation. When the cement content is greater than CC_{cr}, the residual stress decreases with cementation.

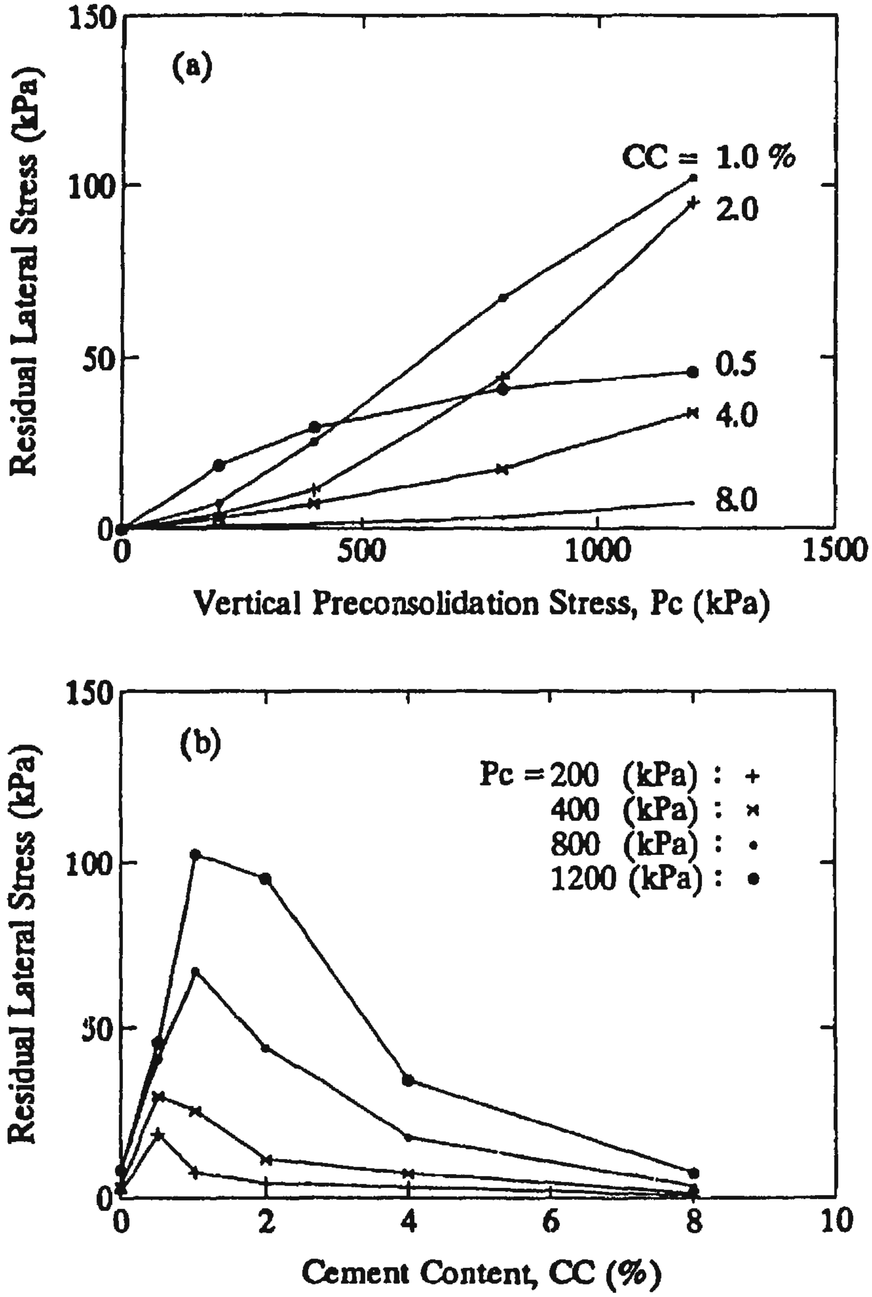


Figure 4.11: Residual Lateral Stress in Ottawa Sand (I_D = 50%)

4.2.2 Lateral stress during unloading

For uncemented sand, the relationship between K_o and OCR during unloading is independent of the preconsolidation stress, as shown in Figure 4.12(a). However, test results in Appendix A indicate that the behaviour of lateral stress in cemented sand during unloading is different from that of uncemented sand. The relationship between K_o and OCR is not unique for a given cement content. It is influenced by the preconsolidation stress, P_c. This relationship is also influenced by cement content. Typical relationships between K_o and OCR of cemented sand are shown in Figure 4.12(b).

The influence of preconsolidation stress magnitude on the relation between K_o and OCR of cemented sand is related to the vertical stress dependence of K_o during loading, and to the existence of residual lateral stress. For uncemented sand, the ratio of lateral stress to vertical stress, K_o, is constant during loading and there is no residual lateral stress after the full unloading of vertical stress. In cemented sand, K_o during virgin loading is not constant. It increases with increasing vertical stress, as shown in Figure 4.6. Furthermore, after the full unloading of vertical stress, there is a residual lateral stress in the sand. The residual stress depends on the degree of cementation and the level of preconsolidation stress.

The test results in Appendix A indicate that K_o of cemented sand during unloading increases with OCR. A typical result is shown in Figure 4.12(b). To demonstrate the influence of preconsolidation stress and cement content, the relationships between K_o and cement content at different levels of preconsolidation stress, when $OCR = 5$ and 10, are shown in Figure 4.13. It can be seen that the

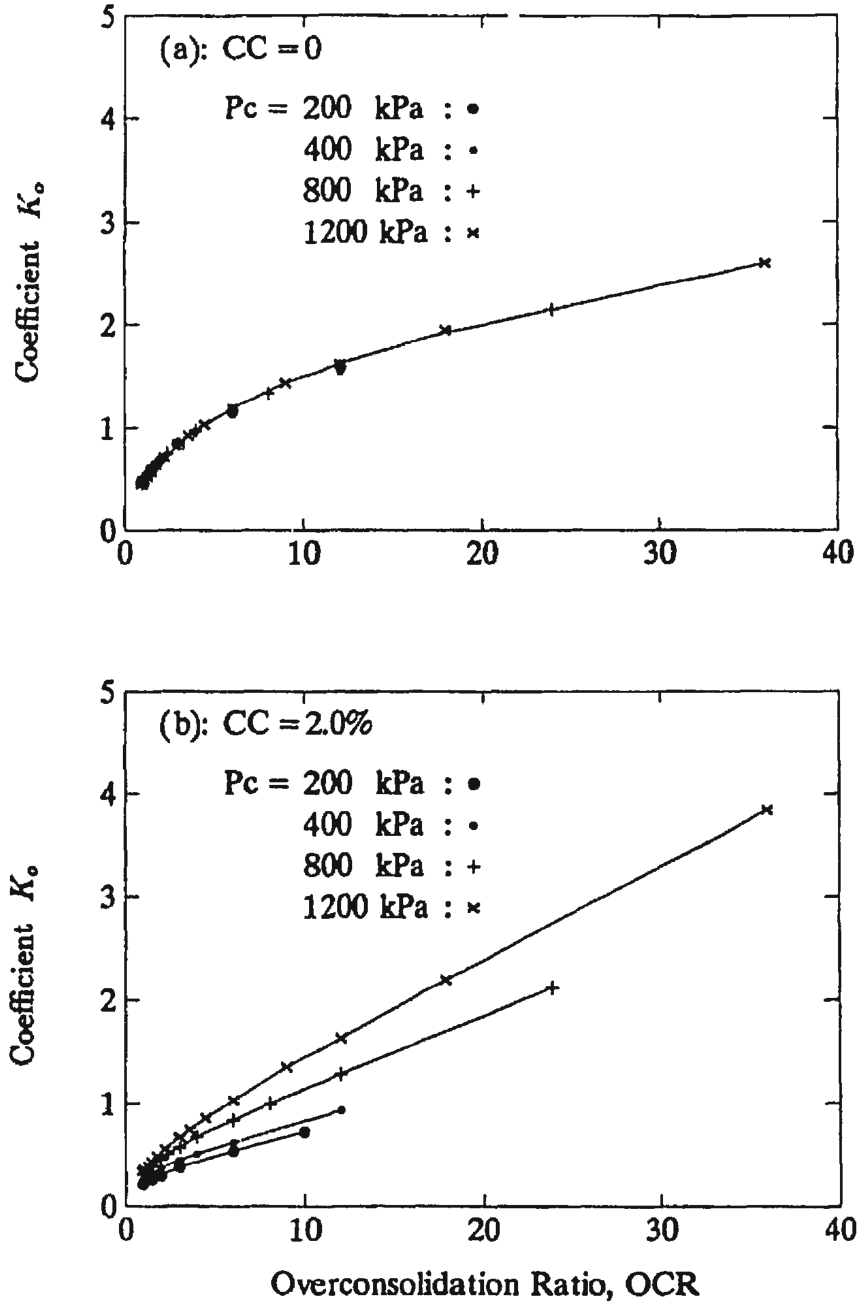


Figure 4.12: Relation between K_o and OCR of Ottawa Sand (I_D = 50%)

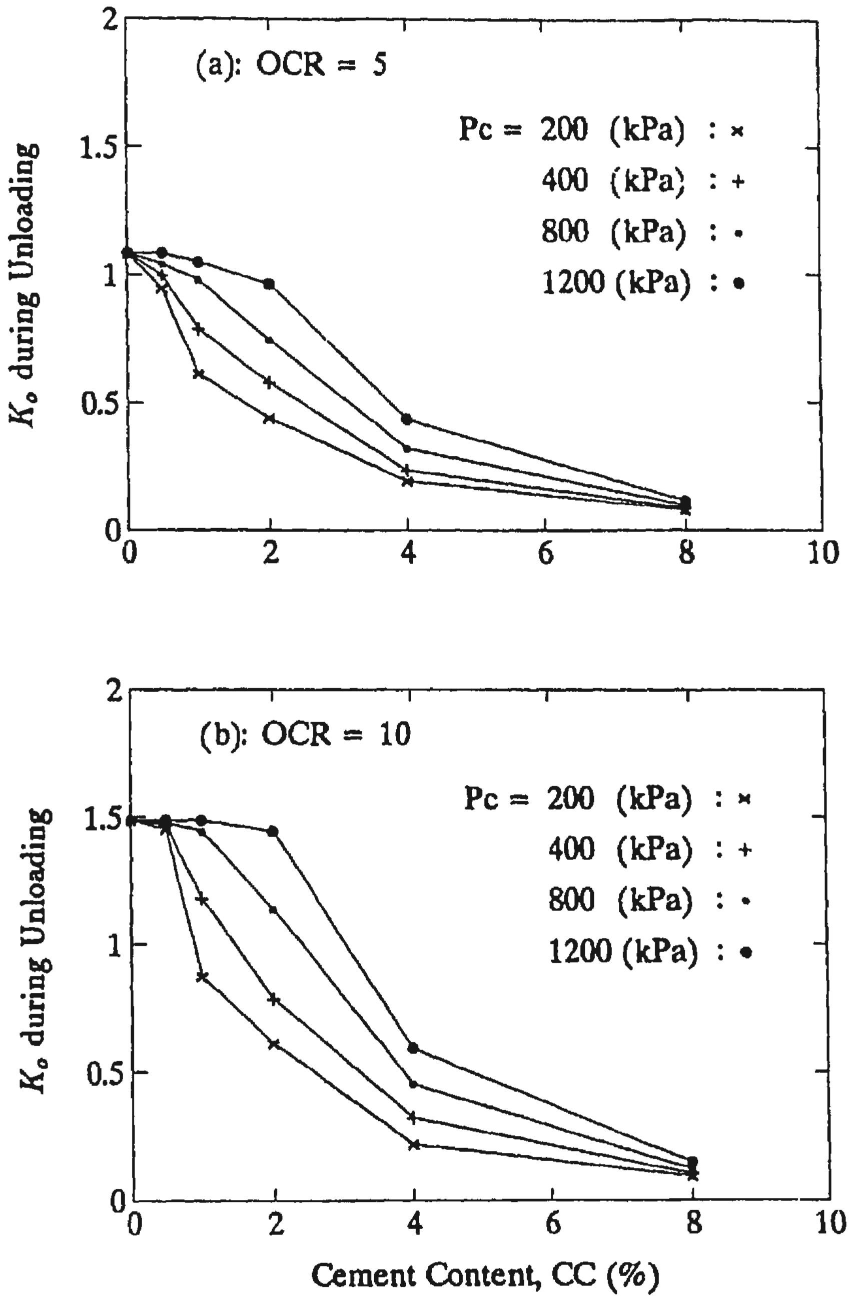


Figure 4.13: Effect of P_c on K_o-CC Relations (Ottawa sand, $I_D = 50\%$)

value of K_o decreases rapidly when cement content is up to 4%. It is also obvious that when the cement content is between 0.5-4%, the effect of preconsolidation stress on the relationship between K_o and cement content is significant. Without cementation, preconsolidation stress has no influence on the value of K_o. When the cement content is 8%, the influence of preconsolidation stress on K_o is not important, because the value of K_o at all preconsolidation stress is very small. The influence of preconsolidation stress on the relation between K_o and cement content, shown in Figure 4.13, is similar to that of P_c on the relation between residual lateral stress and cement content, as shown in Figure 4.11.

4.2.3 Lateral stress during reloading

Due to the existence of residual lateral stress resulting from the preconsolidation stress, the lateral stress in cemented sand during reloading is greater than that during virgin loading. However, the stress path during reloading is similar to that during virgin loading. When the vertical stress during reloading reaches the value of the preconsolidation stress, the reloading stress path joins the virgin loading stress path. Lateral stress during reloading can be normalized as

$$\sigma_h^* = \sigma_{hrl} - \sigma_{hrs}(1 - \frac{\sigma_v}{P_c}) \tag{4.4}$$

where $\sigma_v \leq P_c$, σ_v is the vertical stress, P_c is vertical preconsolidation stress, σ_{hrl} is the lateral stress during reloading, and σ_{hrs} is the residual lateral stress.

The relationships between σ_h^* and σ_v at different preconsolidation stresses are shown in Figure 4.14. It can be seen that at the same vertical stress and the same

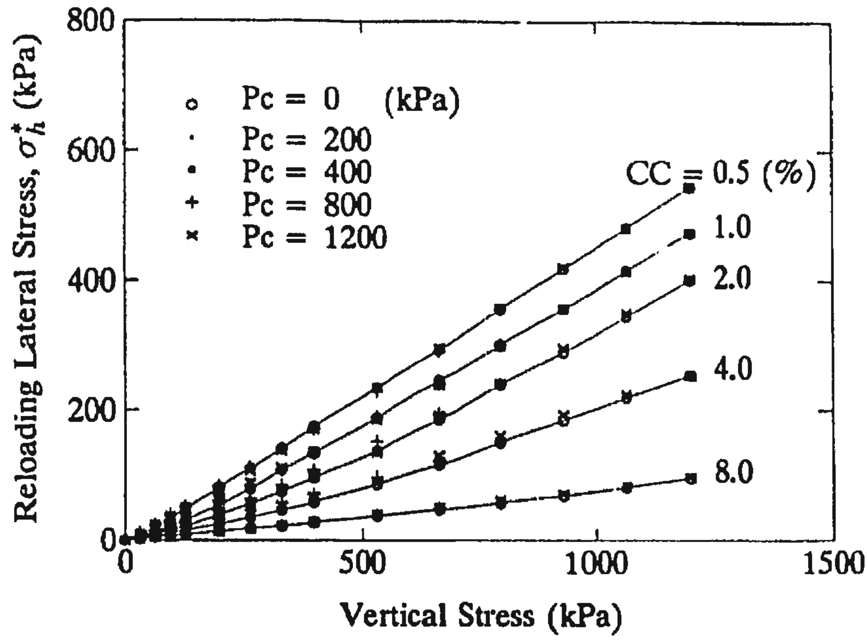


Figure 4.14: Relation between σ_h^* and σ_v

cement content, σ_h^* is not significantly influenced by P_c. That is, at the same vertical stress and the same cementation,

$$\sigma_h^* = \sigma_h \tag{4.5}$$

where σ_h^* is the normalized lateral stress during reloading and σ_h is the lateral stress during virgin loading.

Using equation (4.4) and equation (4.5), lateral stress during reloading can be obtained from

$$\sigma_{hrl} = \sigma_h + \sigma_{hrs}(1 - \frac{\sigma_v}{P_c}) \tag{4.6}$$

That is, for a cemented sand, the lateral stress during reloading depends on the lateral stress during virgin loading, the residual stress, the vertical stress level and the preconsolidation stress. Equation (4.6) is applicable only when $\sigma_v \leq P_c$. When

$\sigma_v = P_c$, the stress path during reloading joins that obtained during virgin loading. The residual lateral stress, σ_{hrs}, in equation (4.6) can be obtained from Figure 4.11. It depends on the degree of cementation and the level of vertical preconsolidation stress. The lateral stress during virgin loading, σ_h, is shown in Figure 4.5(a).

The coefficient of lateral stress at rest during reloading is defined as

$$K_{orl} = \frac{\sigma_{hrl}}{\sigma_v} \tag{4.7}$$

and it is given by

$$K_{orl} = K_o + \frac{\sigma_{hrs}}{\sigma_v}(1 - \frac{\sigma_v}{P_c}) \tag{4.8}$$

where K_{orl} is the coefficient of at rest lateral stress during reloading and K_o is the coefficient during virgin loading.

4.3 Factors Affecting Test Results

There are many factors affecting at rest lateral stress in cemented sand. As described above, the lateral stress is influenced by the degree of cementation, the vertical stress, stress history and sand type. The lateral stress decreases significantly with increasing cement content. The effect of cementation is more important at low vertical stress than at high vertical stress because more cementation bonds are broken at higher stress. Stress history has a significant effect on the lateral stress. Cementation bonds are stronger in the marine sand than in the Ottawa sand due to the different physical properties of the two sands.

In addition to cement content, vertical stress, stress history and sand type, lateral stress in cemented sand is also influenced by the density index of the sand and the curing period. The effects of sand density and curing period are discussed next.

4.3.1 Sand density

The test results described above are for the specimens with a density index of 50%. However, the behaviour of cemented sand is expected to be affected by the sand density. To investigate the influence of density index on the lateral stress in cemented sand, three specimens with density indexes of 30, 50 and 80% were prepared and tested. The cement content was 2.0 % and the curing period was 10 days. The results are shown in Figure 4.15.

It can be seen that the lateral stress and the value of K_o decrease with increasing density index (I_D). It is also obvious from the shape of the curves that the relationship between K_o and vertical stress is affected by I_D. The comparison of the results shown in Figure 4.15(b) with those in Figure 4.6 implies that the K_o behaviour of loose sand with more cementation is similar to that of dense sand with less cementation. The results in Figure 4.15 indicate that at the same cement content, cementation bonds are stronger in denser sand than in looser sand.

These results are in accordance with the conclusion of Chang and Woods (1992). In the study of low strain shear modulus of cemented sands, Chang and Woods (1992) defined the degree of cementation in terms of percent voids filled with cement in the sands. It was found that the shear modulus increases with the degree of cementation. It is obvious that when the cement content by the weight of dry sand is the same, the percentage of voids filled with cement is higher in dense sand than

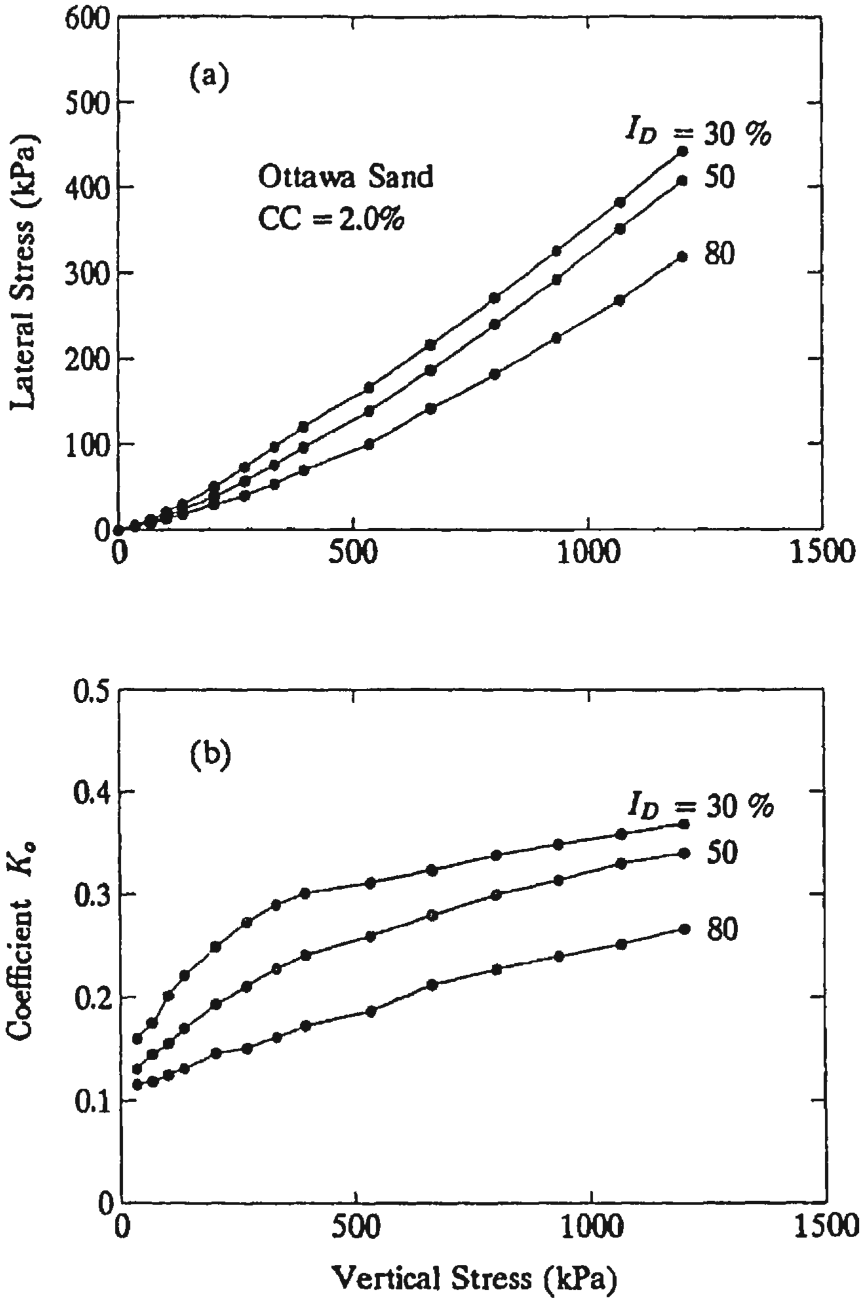


Figure 4.15: Effect of Density Index, I_D

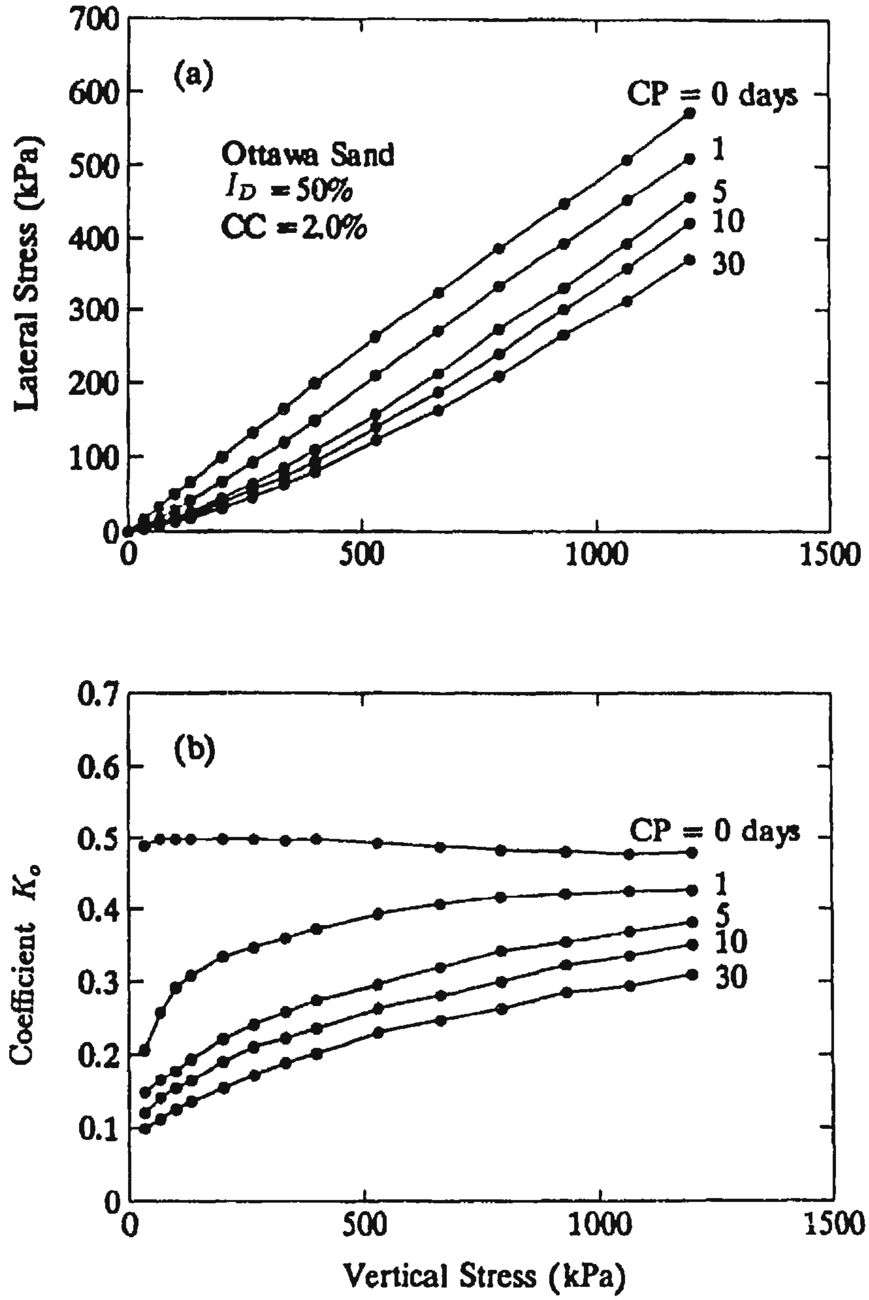


Figure 4.16: Effect of Curing Period, CP

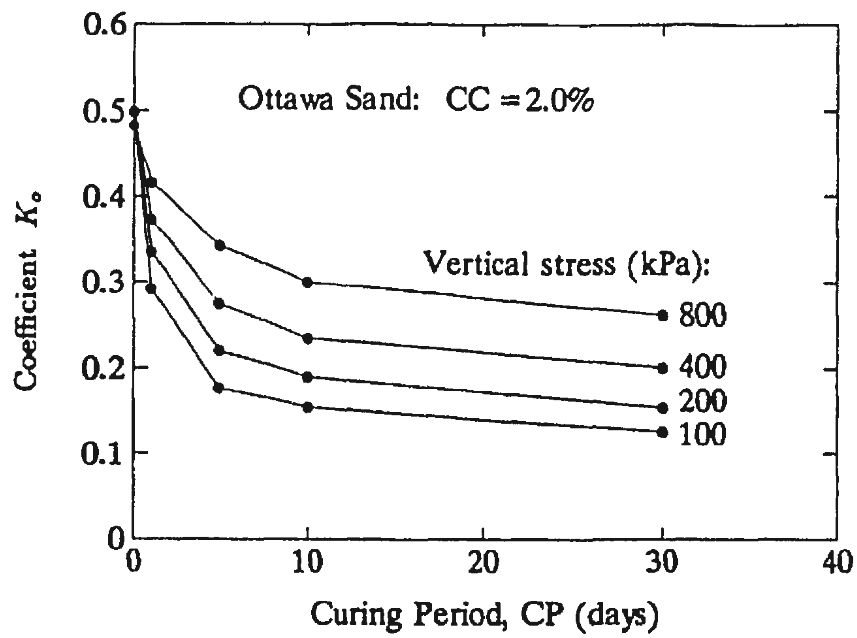


Figure 4.17: Relation between K_o and CP (I_D = 50%)

in loose sand.

4.3.2 Curing period

As described above, most of the specimens were cured for 10 ·ys before tests were carried out. For cemented sands, the cementation bonds among soil particles become stronger with time due to the hydration of cement (Saxena, Avramidis and Reddy, 1988). To study the effect of curing period on lateral stress in cemented sand, 5 Specimens with a density index of 50% and cement content of 2% were tested after being cured for 0, 1, 5, 10, and 30 days respectively. The test results in Figure 4.16 indicate that the lateral stress and K_o decrease with curing period.

The decrease in lateral stress with curing period indicates that cementation bonds in the sands become stronger with time. It can be seen from Figure 4.17 that the value of K_o decreases dramatically when the curing period is less than 5 days. After 5 days of curing, K_o of cemented sand decreases more and more slowly with curing period.

Chapter 5

Conclusions

A modified oedometer ring was used to measure the at rest lateral stress of cemented sands. Based on the test results described in this thesis, the following conclusions can be drawn.

1. The behaviour of at rest lateral stress in the artificially cemented sands is different from that in uncemented sands. At rest lateral stress and K_o of cemented sands are related to soil type, cement content, vertical stress, stress history, sand density and curing period.

2. Lateral stress and K_o of cemented sands decrease significantly with increasing cement content. They are influenced by soil type and vertical stress. Cementation is stronger in sand with smaller grain sizes and greater uniformity coefficient, and hence the lateral stress is smaller. Due to the break down of cementation bonds with increasing vertical stress, the relationship between vertical stress and lateral stress is nonlinear; the value of K_o increases with vertical stress, especially for weakly cemented sands at low stress.

3. Stress history has a significant influence on the behaviour of the lateral stress. Cemented sand exhibits a residual lateral stress after the full removal of vertical stress. The residual stress increases with the vertical preconsolidation stress (P_c). The effect of cement content on the residual stress is complicated. At given P_c, there is a critical cement content at which the residual stress reaches the maximum value. During unloading, K_o of cemented sand increases with OCR. The relationships between K_o and OCR are affected by cement content and P_c. The lateral stress during reloading is related to the preconsolidation stress level, residual lateral stress, lateral stress during virgin loading and vertical stress.

4. K_o of cemented sands is also influenced by sand density and curing period. It decreases with increasing density index and with increasing curing period. The lateral stress behaviour in a loose sand with higher cement content is similar to that in a dense sand with lower cement content.

References

Abdelhamid, M. S., and Krizek, R. J. (1976). "At-Rest Lateral Earth Pressure of Consolidating Clay." *Journal of Geotechnical Engineering Division*, ASCE, Vol. 102, No. GT7, pp. 721-738.

Acar, Y. B., and El-Tahir E. A. (1986). "Low Strain Dynamic Properties of Artificially Cemented Sand." *Journal of Geotechnical Engineering*, Vol. 112, No. 11, pp. 1001-1015.

Andrawes, K. Z., and El-Sohby, M. A. (1973). "Factors Affecting Coefficient of Earth Pressure K_o." *Journal of Soil Mechanics and Foundation Engineering*, ASCE, Vol. 99, No. SM7, pp. 527-539.

Angemeer, J., Carlson, E., and Klick, J. H. (1973). "Techniques and Results of Offshore Pile Loading Testing in Calcareous Soil." *Proceedings of Fifth Annual Offshore Technology Conference*, Houston, Texas, pp. 667-692.

Bishop, A. W., and Eldin, A. K. G. (1953). "The Effect of Stress History on the Relation between ϕ and Porosity of Sand." *Proceedings of the 3rd International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, pp. 100-105.

Bowles, J. E. (1986). *Engineering Properties of Soils and Their measurement.* third edition, McGraw-Hill Book Company.

Boyce, J. R., and Brown, S. F. (1976). "Measurement of Elastic Strain in Granular Material." *Geotechnique*, Vol. 26, No. 4, pp. 637-640.

Brooker, E. W., and Ireland, H. O. (1965). "Earth Pressures at Rest Related to Stress History." *Canadian Geotechnical Journal*, Vol. 2, No. 1, pp. 1-15.

Budhu, M. (1985). "Lateral Stresses Observed in Two Simple Shear Apparatus." *Journal of Geotechnical Engineering*, Vol. 111, No. 6, pp. 698-711.

Calhoun, D. E., and Triandafilidis, G. E. (1969). "Dynamic Oedometer Study of Lateral yielding Effects." *Proceedings of 7th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, pp. 65-72.

Campanella, R., and Vaid, Y. P. (1972). "A Simple K_o Triaxial Cell." *Canadian Geotechnical Journal*, Vol. 9, No. 3, pp. 249-260.

Chang, T.S., and Woods, R.D. (1992). "Effect of Particle Contact Bond on Shear Modulus." *Journal of Geotechnical Engineering*, Vol. 118, No. 8, pp. 1216-1233.

Charlie, W. A., Rwejyogo, M. F. J., and Doehring, D. O. (1992). "Time-Dependent Cone Penetration Resistance due to Blasting." *Journal of Geotechnical Engineering*, Vol. 118, No. 8, pp. 1200-1215.

Chiang, Y. C., and Chae, Y. S. (1972). "Dynamic properties of cement treated soils." *Highway Research Record*, No. 379, pp. 39-51.

Clough, G. W., Iwabuchi, J., Rad, N. S., and Kuppusamy, T. (1989). "Influence of Cementation on Liquefaction of Sands." *Journal of Geotechnical Engineering*, Vol. 115, No. 8, pp. 1102-1117.

Clough, G.W., Sitar, N., Bachus, R., and Rad, N. S. (1981). "Cemented Sands under Static Loading." *Journal of Geotechnical Engineering Division*, ASCE, Vol. 107, No. GT6, pp. 799-817.

Datta, M., Gulhati, S. K., and Rao, G. V. (1980). "An Appraisal of the Existing Practice of Determining the Axial Load Capacity of Deep Penetration Piles in Calcareous Sands." *Proceedings of Twelfth Annaul Offshore Technology Conference*, Houston, Texas, Vol. 4, pp. 119-130.

Dupas, J. M., and Pecker, A. (1979). "Static and Dynamic Properties of Sand-Cement." *Journal of Geotechnical Engineering Division*, ASCE, Vol. 105, No. GT3, pp. 419-435.

Dutt, R. N., and Cheng, A. P. (1984). "Frictional Response of Piles in Calcareous Deposits." *Proceedings of Sixteenth Annual Offshore Technology Conference*. Houston, Texas, pp. 527-534.

Edil, T. B., and Dhowian, A. W. (1981). "At-Rest Lateral Pressure of Peat Soils." *Journal of Geotechnical Engineering Division*, ASCE, Vol. 107, No. GT2, pp. 201-217.

El Ruwayih, A. A. (1976). "The Design Manufacture and Performance of a Lateral Strain Device." *Geotechnique*, Vol. 26, No. 1, pp. 215-216.

Feda, J. (1984). "K_o Coefficient of Sand in Triaxial Apparatus." *Journal of Geotechnical Engineering*, Vol. 110, No. 4, pp. 519-524.

Frydman, S., Hendron, D., Horn, H., Steinback, J., Backer, R., and Shoal, B. (1980). "Liquefaction Study of weakly cemented Sand." *Journal of Geotechnical Engineering Division*, ASCE, Vol. 106, No. GT3, pp. 275-297.

Fukagawa, R., and Ohta, H. "Effect of Some Factors on K_o-Value of a Sand." *Soils and Foundations*, Vol. 28, No. 3, pp. 93-106.

Hagenaar, J., and Van Seters, A. (1985). "Ultimate Axial Bearing Capacity of Piles Driven into Coral Rock and Carbonate Soils." *Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering*, Vol. 3, pp. 1599-1603.

Hardin, B. O., and Drnevich, V. P. (1972a). "Shear Modulus and Damping in Soils: Design Equations and Curves." *Journal of Soil Mechanics and Foundation Engineering*, ASCE, Vol. 98, No. SM7, pp. 667-692.

Hardin, B. O., and Drnevich, V. P. (1972b). "Shear Modulus and Damping in Soils: Measurement and Parameter Effect." *Journal of Soil Mechanics and Foundation Engineering*, ASCE, Vol. 98, No. SM7, pp. 667-692.

Ismael, N. F. (1989). "Skin Friction of Driven Piles in Calcareous Sands." *Journal of Geotechnical Engineering*, Vol. 115, No. 1, pp. 135-139.

Jaky, J. (1944). "The Coefficient of Earth Pressure at Rest." *Journal of the Society of Hungarian Architects and Engineers*, Budapest, Hungary, pp. 355-358.

King, R. W., van Hoorydonk, W. R., Kolk, H. F., and Windle, D. (1980). "Geotechnical Investigations of Calcareous soils on the Northwest Shelves, Australia." *Proceedings of Twelfth Annual Offshore Technology Conference*, Houston, Texas, pp. 303-313.

Kochi, Y., and Tatsuoka, F. (1984). "Some Factors Affecting K_o−Values Measured in Triaxial Cell." *Soils and Foundations*, Vol. 24, No. 3, pp. 52-68.

Kolias, S., and Williams, R. I. T. (1984). "Estimation of the Modulus of Elasticity of Cement Stabilized Materials." *Geotechnical Testing Journal*, Vol. 7, No. 1, pp.26-35.

Ladd, R. S. (1978). "Preparing Test Specimens using Undercompaction." *Geotechnical Testing Journal*, Vol. 1, No. 1, pp. 16-23.

Lade, P. V., and Overton, D. D. (1989). "Cementation Effects in Frictional Materials." *Journal of Geotechnical Engineering*, Vol. 115, No. 10, pp. 1373-1387.

Li, L., and Mitchell, R. (1988). "Effect of Reinforcing Elements on the Behavior of Weakly Cemented Sands." *Canadian Geotechnical Journal*, Vol. 25, No. 2, pp. 389-395.

Lo, S-C.R., and Chu, J. (1991). "Measurement of K_o by Triaxial Strain Path Testing." *Soils and Foundations*, Vol. 31, No. 2, pp. 181-187.

Mayne, P. W., and Kulhawy, F. H. (1982). "K_o-OCR Relationship in Soil." *Journal of Geotechnical Engineering Division*, Vol. 108, No. GT6, pp. 851-872. .

McClelland, B. (1974). "Design of Deep Penetration Piles for Ocean Structures." *Journal of Geotechnical Engineering Division*, ASCE, Vol. 100, No. GT7, pp. 705-747.

Mitchell, J. K. (1986). "Practical Problems from Surprising Soil Behavior." *Journal of Geotechnical Engineering*, Vol. 112, No. 3, pp. 259-289.

Mitchell, J. K., and Solymar, Z. V. (1984). "Time-dependent Strength Gain in Freshly Deposited or Densified Sand." *Journal of Geotechnical Engineering*, Vol. 110, No. 11, pp. 1559-1576.

Murff, J. D. (1987). "Pile Capacity in Calcareous Sands: State of the Art." *Journal of Geotechnical Engineering*, Vol. 113, No. 5, pp. 490-507.

Nauroy, J. F., and LeTirant, P. (1985). "Driven Piles and Drilled and Grouted Piles in Calcareous Sands." *Proceedings of Eleventh Annual Offshore Technology Conference*, Vol.1, Houston, Texas, Vol. 1, pp. 83-91.

Noorany, I. (1985). "Side Friction of Piles in Calcareous Sands." *Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering*, Vol. 3, pp. 1611-1614.

Noorany, I. (1989). "Classification of Marine Sediments." *Journal of Geotechnical Engineering*, Vol. 115, No. 1, pp. 23-37.

Ofer, Z. (1981). "Laboratory Instrument for Measuring Lateral Soil Pressure and Swelling Pressure." *Geotechnical Testing Journal*, Vol. 4, No. 4, pp. 177-182.

Perry, C. C., and Lissner, H. R. (1961). "Strain Gage Instrumentation." in *Shock and Vibration Handbook*, edited by Cyril M. Harris and Charles E. Crede, McGraw - Hill Book Company, Vol. 1, chapter 17, pp. 1-43.

Poulos, H. G. (1988). *Marine Geotechnics*, Unwin Hyman Ltd, London, 473 p.

Poulos, H. G., and Chua, E. W. (1985). "Bearing Capacity of Foundations on Calcareous Sand." *Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering*, Vol. 3, pp. 1619-1622.

Rad, N. S., and Tumay, M. T. (1986). "Effect of Cementation on the Cone Penetration Resistance of Sand: a Model Study." *Geotechnical Testing Journal*, Vol. 9, No. 3, pp. 117-125.

Saxena, S. K., Avramidis, A. S., amd Reddy, K. R. (1988). "Dynamic Moduli and Damping Ratios for Cemented Sands at Low Strains." *Canadian Geotechnical Journal*, Vol. 25, pp. 353-368.

Saxena, S. K., and Lastrico, R. M. (1978). "Static Properties of Lightly Cemented Sands." *Journal of Geotechnical Engineering Division*, ASCE, Vol. 104, No. GT12, pp. 1449-1464.

Saxena, S. K., Reddy, K. R., and Avramidis, A. S. (1988). "Liquefaction Resistance of Artificially Cemented Sand." *Journal of Geotechnical Engineering*, Vol. 114, No. 12, pp. 1395-1413.

Sitar, N., and Clough, G. W. (1983). "Seismic response of steep slope in cemented Soils." *Journal of Geotechnical Engineering Division*, ASCE, No. GT2, pp. 210-227.

Thomann, G. T., and Hryciw, R. D. (1990). "Laboratory Measurement of Small Strain Shear Modulus under K_o Conditions." *Geotechnical Testing Journal*, Vol. 13, No. 2, pp. 97-105.

Tsuchida, T., and Kikuchi, Y. (1991). "K_o Consolidation of Undisturbed Clays by Means of Triaxial Cell." *Soils and Foundations*, Vol. 31, No. 3, pp. 127-137.

Wissa, A. E. Z., Ladd, C. C., and Lambe, T. W. (1965). "Effective Stress Strength Parameters of Stabilized Soils." *Proceedings of the 6th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, pp. 412-416.

Youd, T. L., and Craven, T. N. (1975). "Lateral Stress in Sands During Cyclic Loading." *Journal of Geotechnical Engineering Division*, ASCE, Vol. 101, No. GT2, pp. 217-221.

Appendix A

At Rest Lateral Stress and K_0 during Loading, Unloading and Reloading

The at rest lateral stress (σ_h) and K_o of 24 specimens during loading, unloading and reloading are shown in Figure A.1 to Figure A.24 respectively. The specimens were prepared using No. 3 Ottawa sand with cement contents of 0, 0.5, 1.0, 2.0, 4.0 and 8.0%. The density index of all specimens was 50% and the curing period of all cemented specimens was 10 days. The results and analyses in section 4.1 and 4.2, chapter 4, are based on the data shown in the following figures.

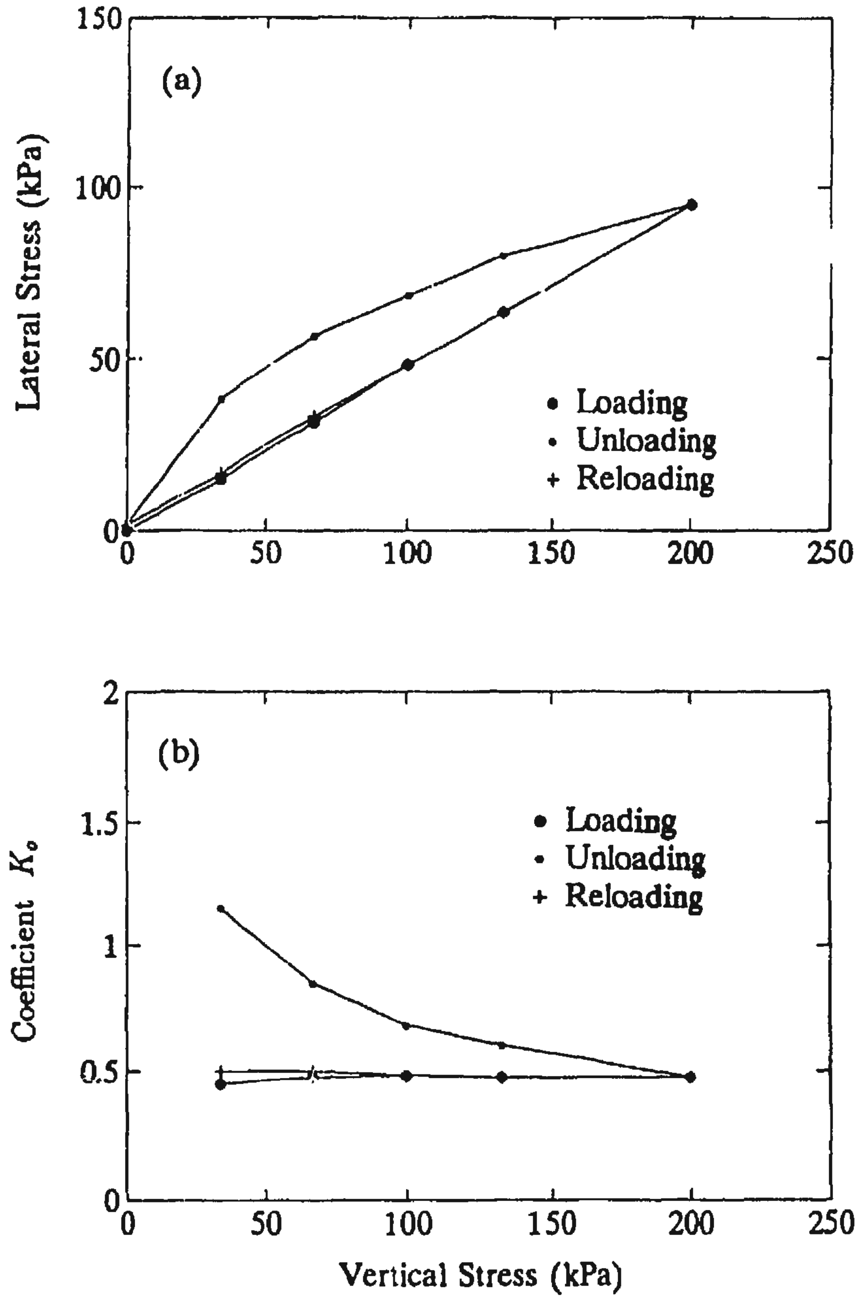


Figure A.1: Lateral Stress (σ_h) and K_o (CC = 0, $\sigma_v \leq 200$ kPa)

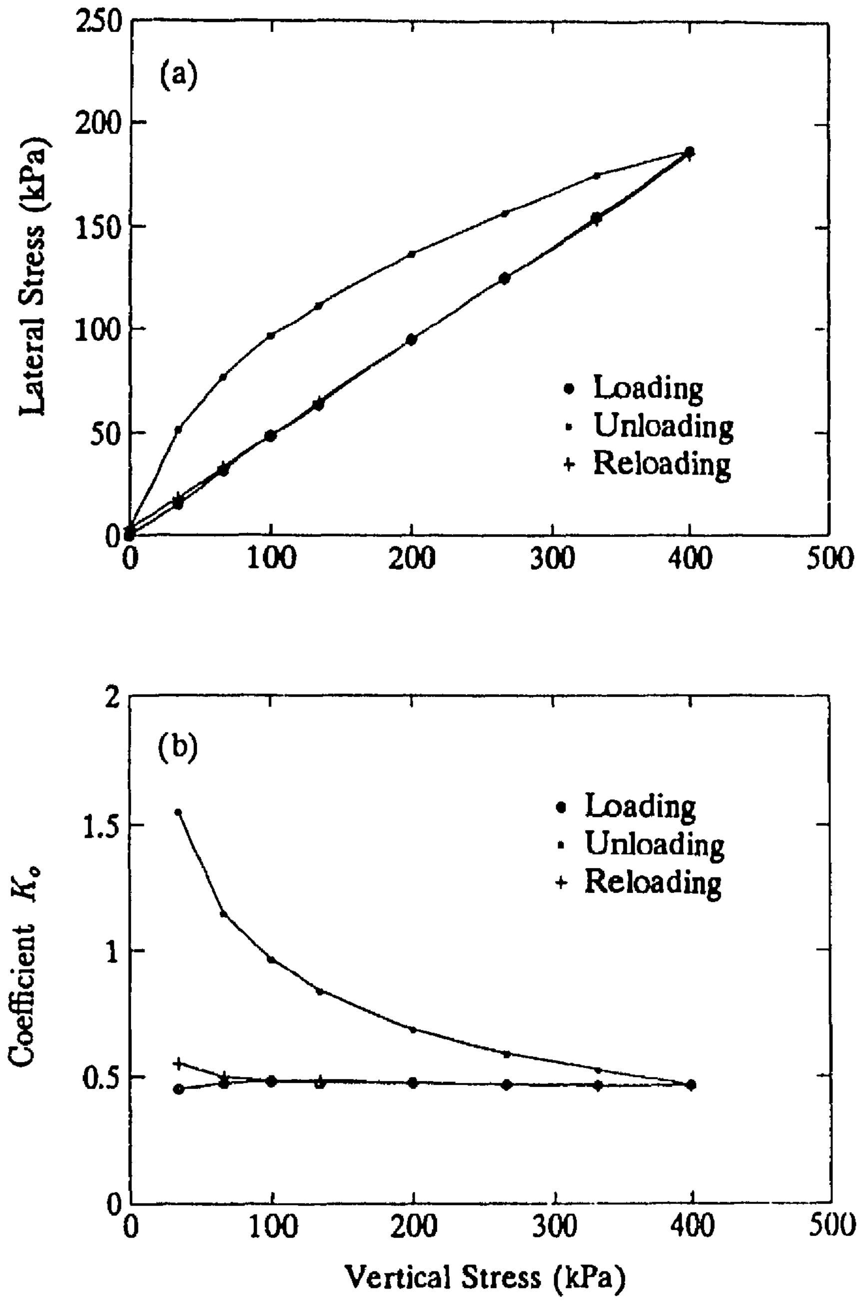


Figure A.2: Lateral Stress (σ_h) and K_o (CC = 0, $\sigma_v \leq 400$ kPa)

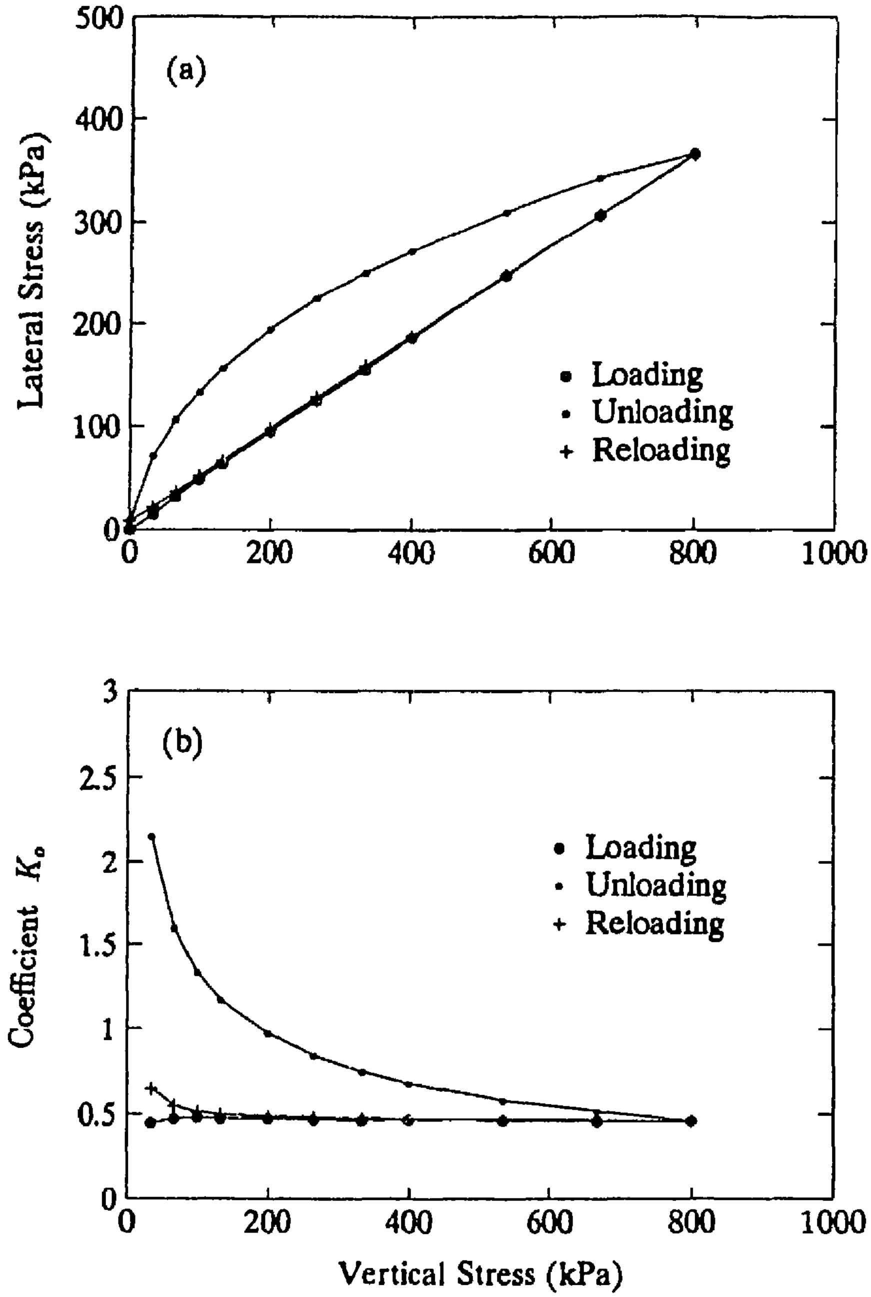


Figure A.3: Lateral Stress (σ_h) and K_o (CC = 0, $\sigma_v \leq 800$ kPa)

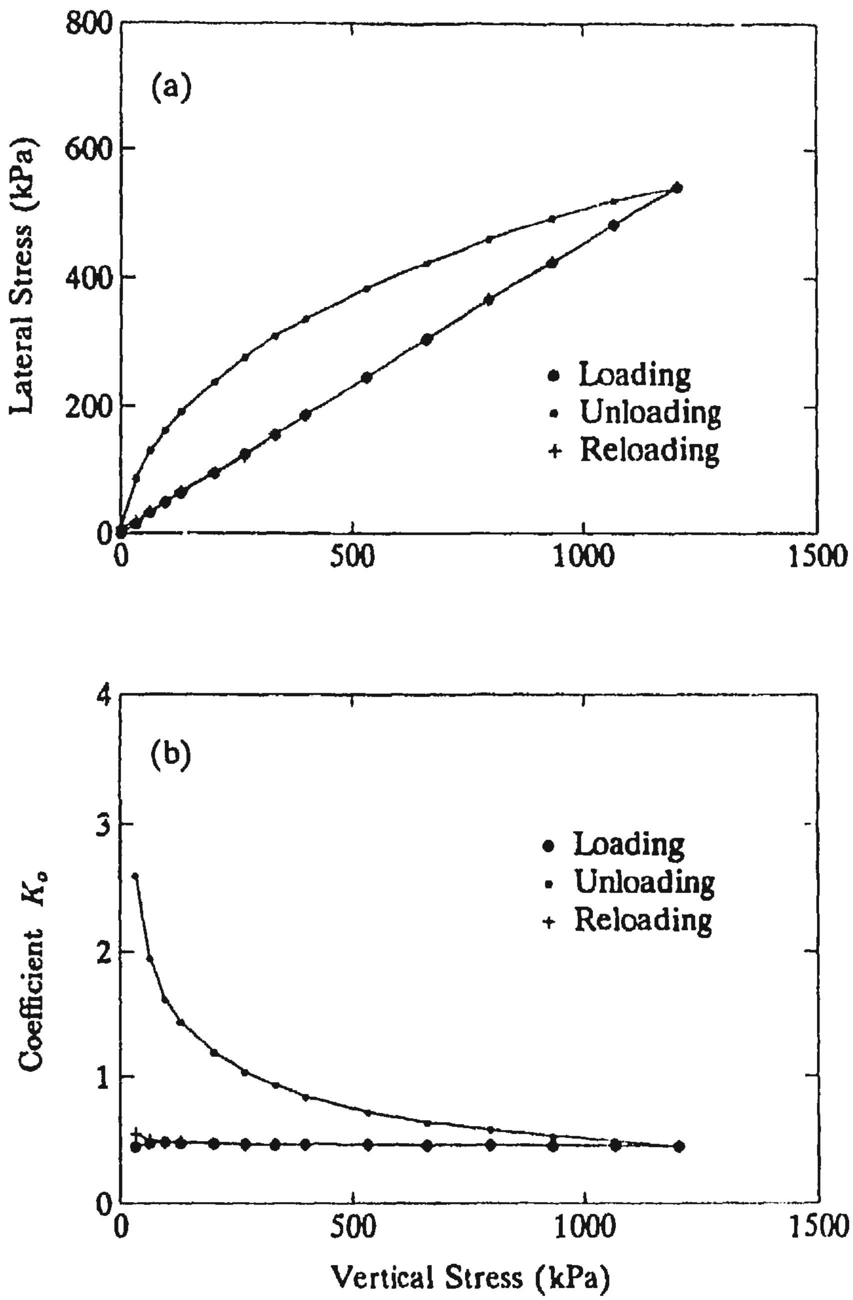


Figure A.4: Lateral Stress (σ_h) and K_o (CC = 0, $\sigma_v \leq 1200$ kPa)

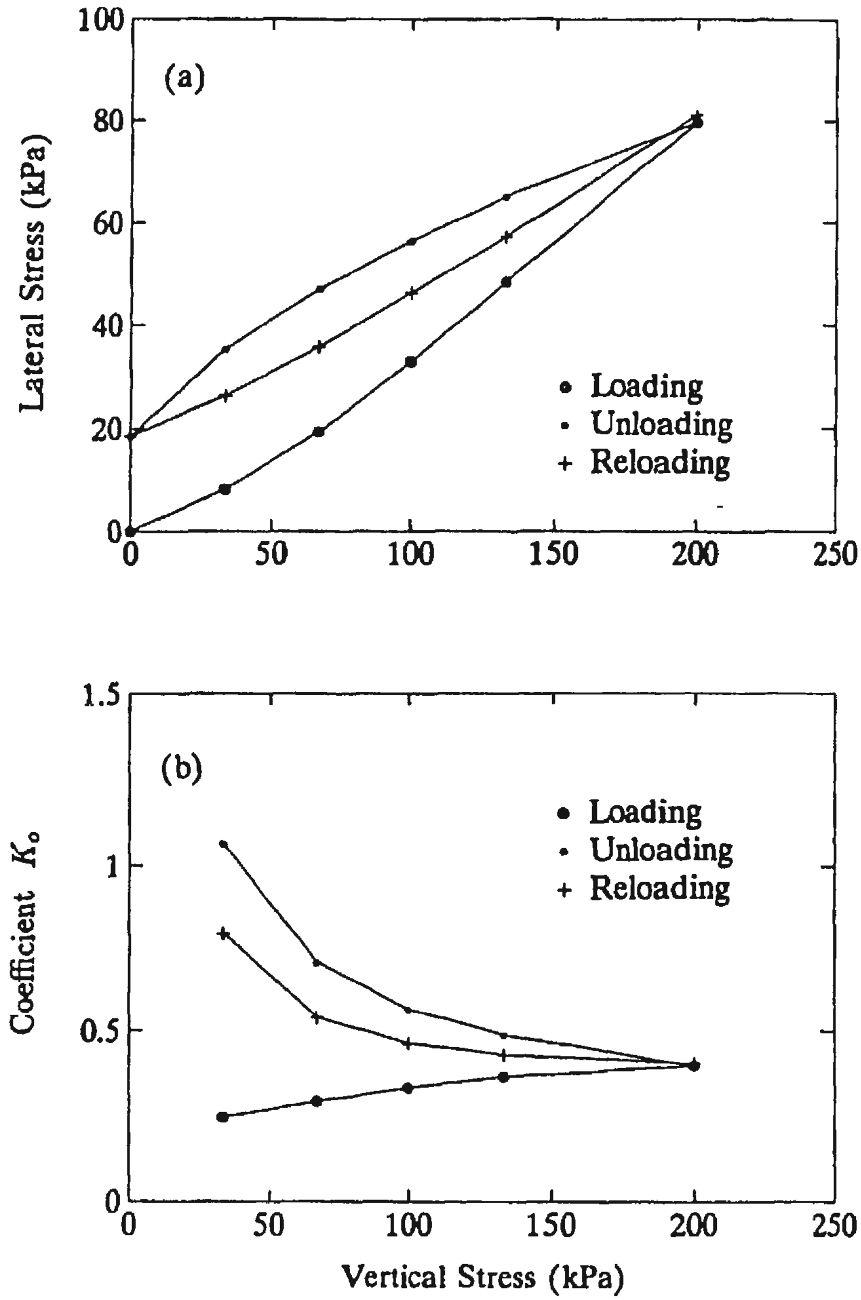


Figure A.5: Lateral Stress (σ_h) and K_o (CC = 0.5%, $\sigma_v \leq 200$ kPa)

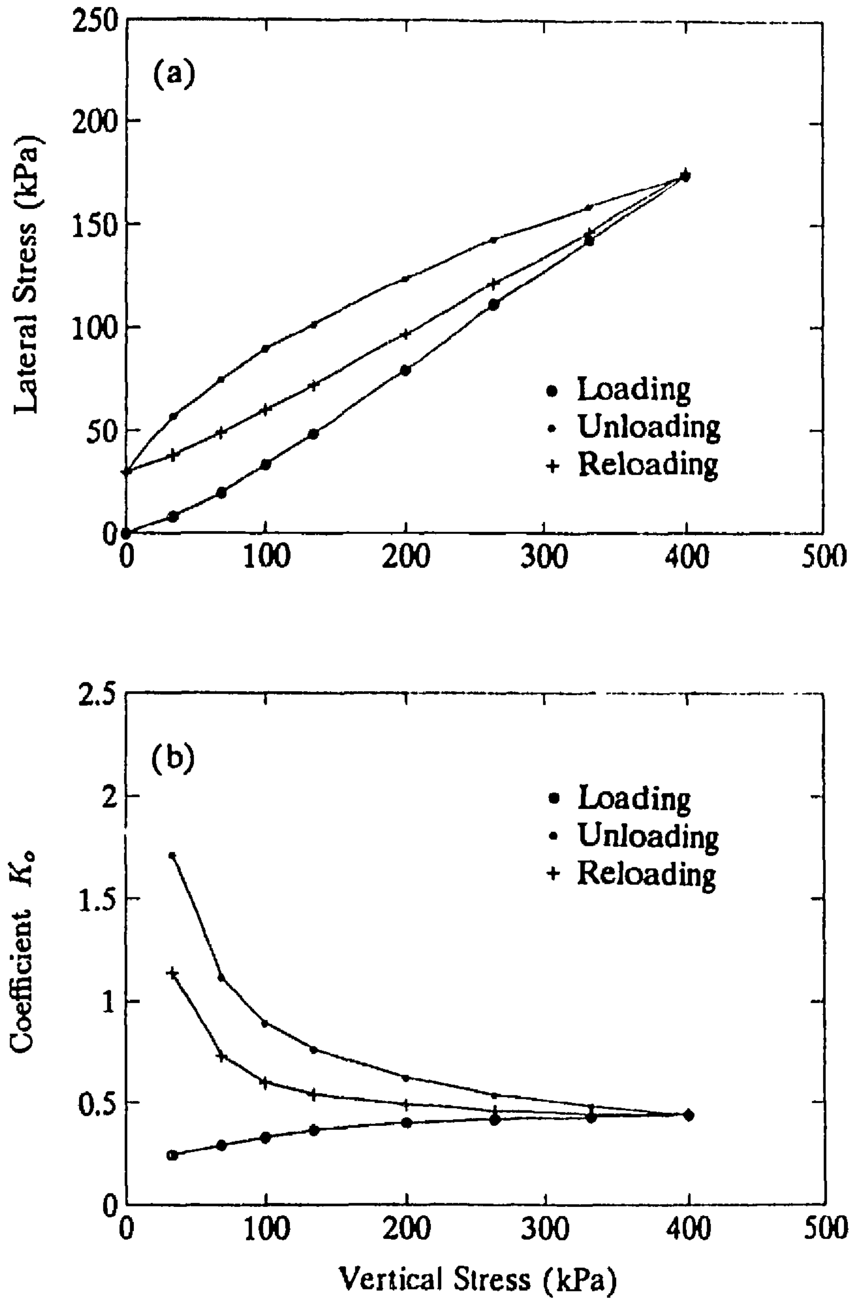
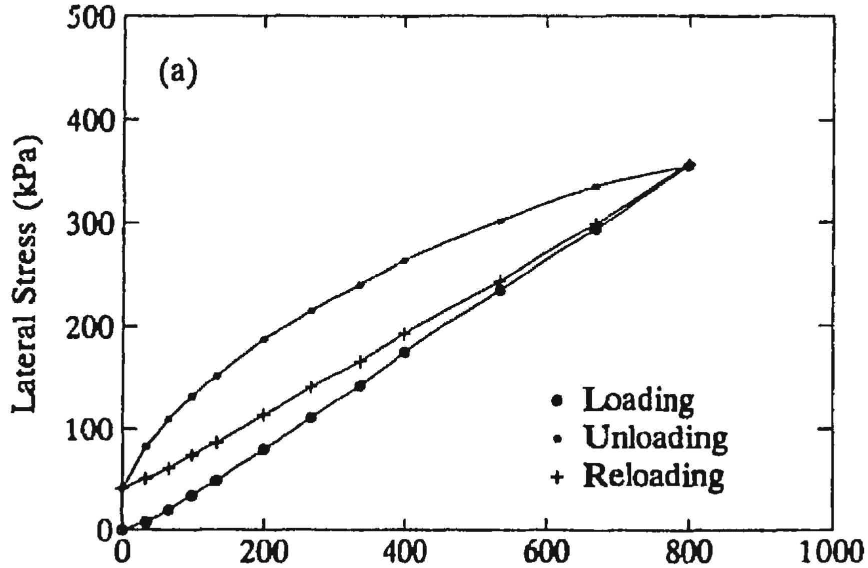


Figure A.6: Lateral Stress (σ_h) and K_o (CC = 0.5%, $\sigma_v \leq 400$ kPa)



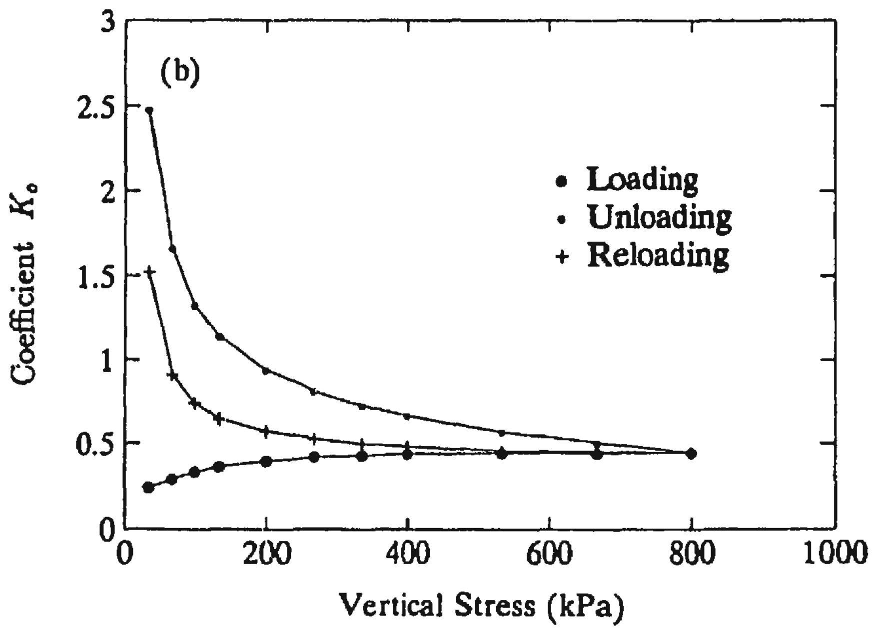


Figure A.7: Lateral Stress (σ_h) and K_o (CC = 0.5%, $\sigma_v \leq 800$ kPa)

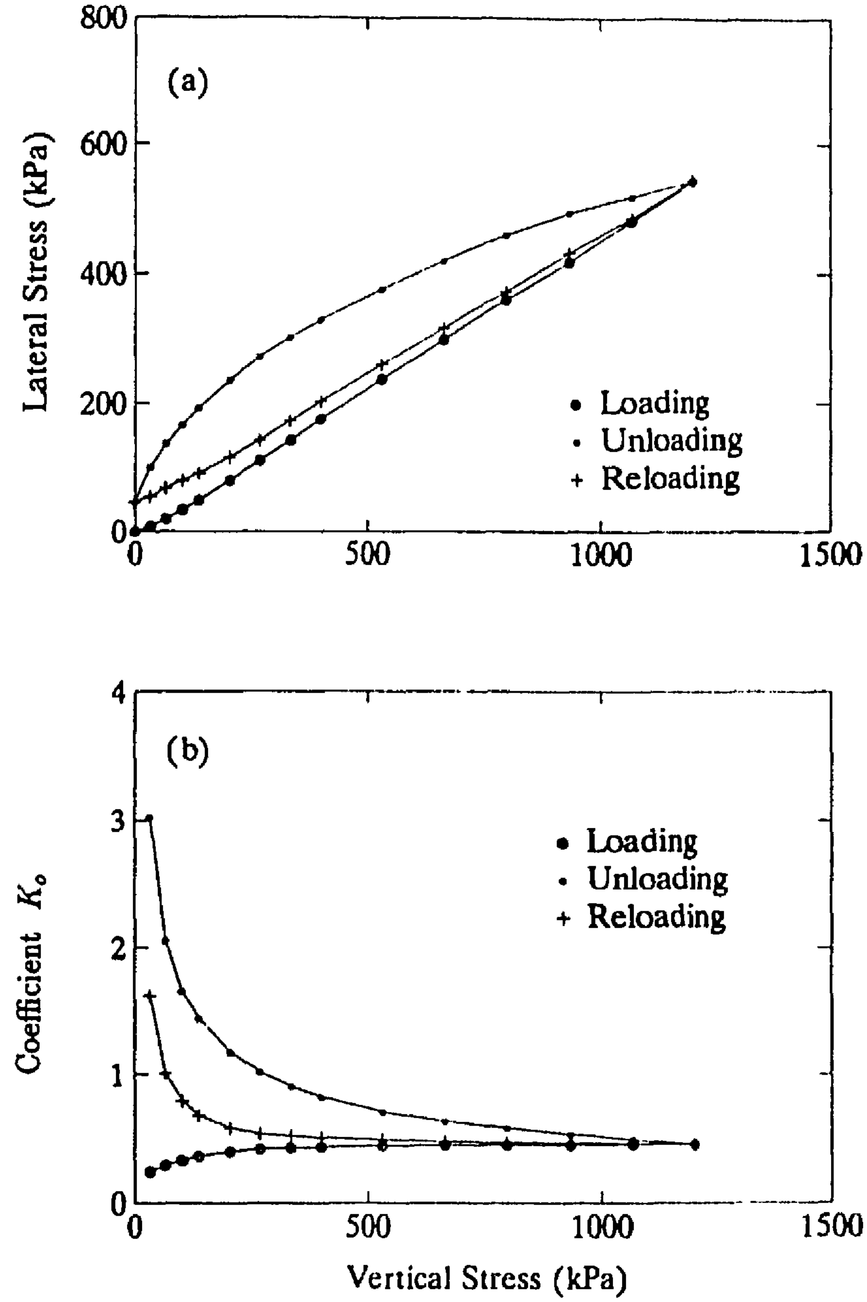


Figure A.8: Lateral Stress (σ_h) and K_o (CC = 0.5%, $\sigma_v \leq 1200$ kPa)

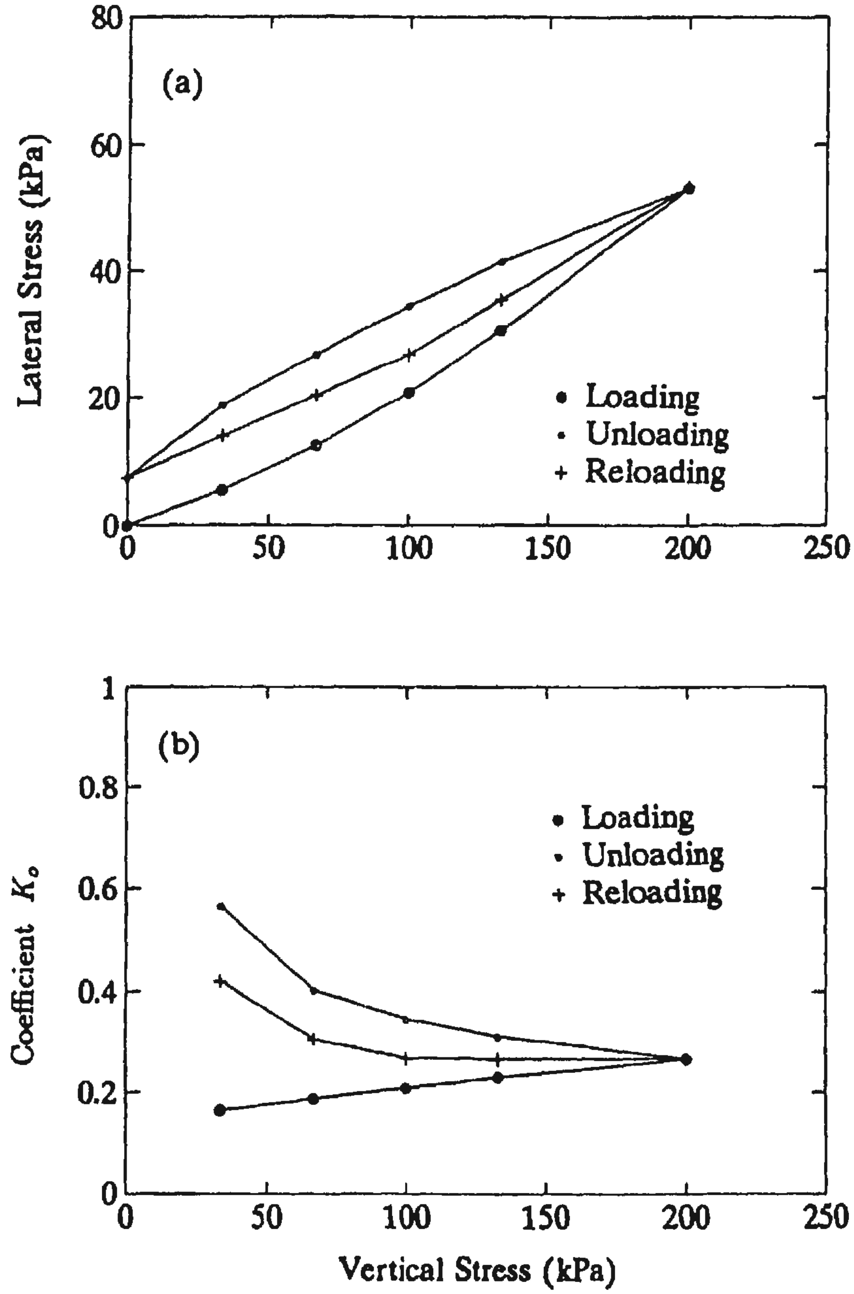


Figure A.9: Lateral Stress (σ_h) and K_o (CC = 1.0%, $\sigma_v \leq 200$ kPa)

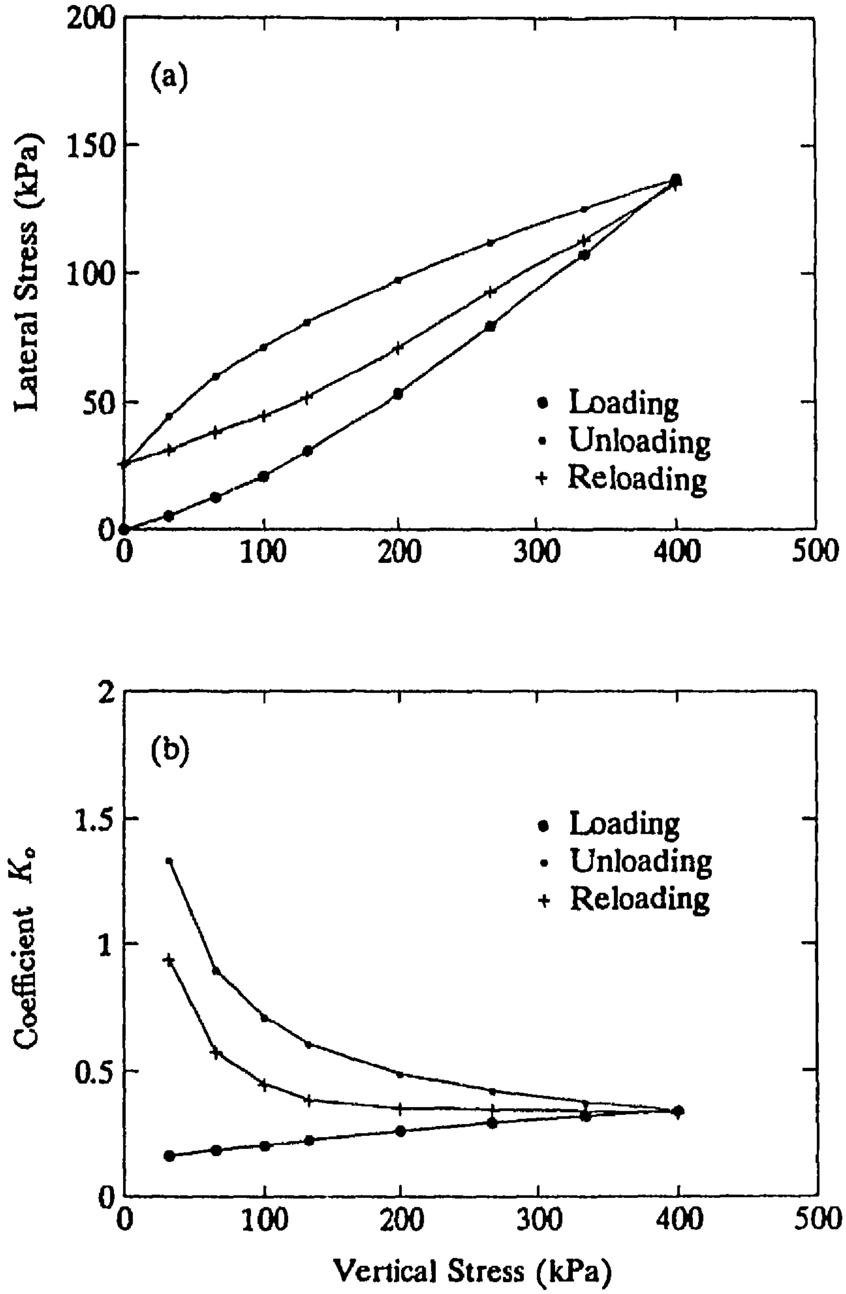


Figure A.10: Lateral Stress (σ_h) and K_o (CC = 1.0%, $\sigma_v \leq 400$ kPa)

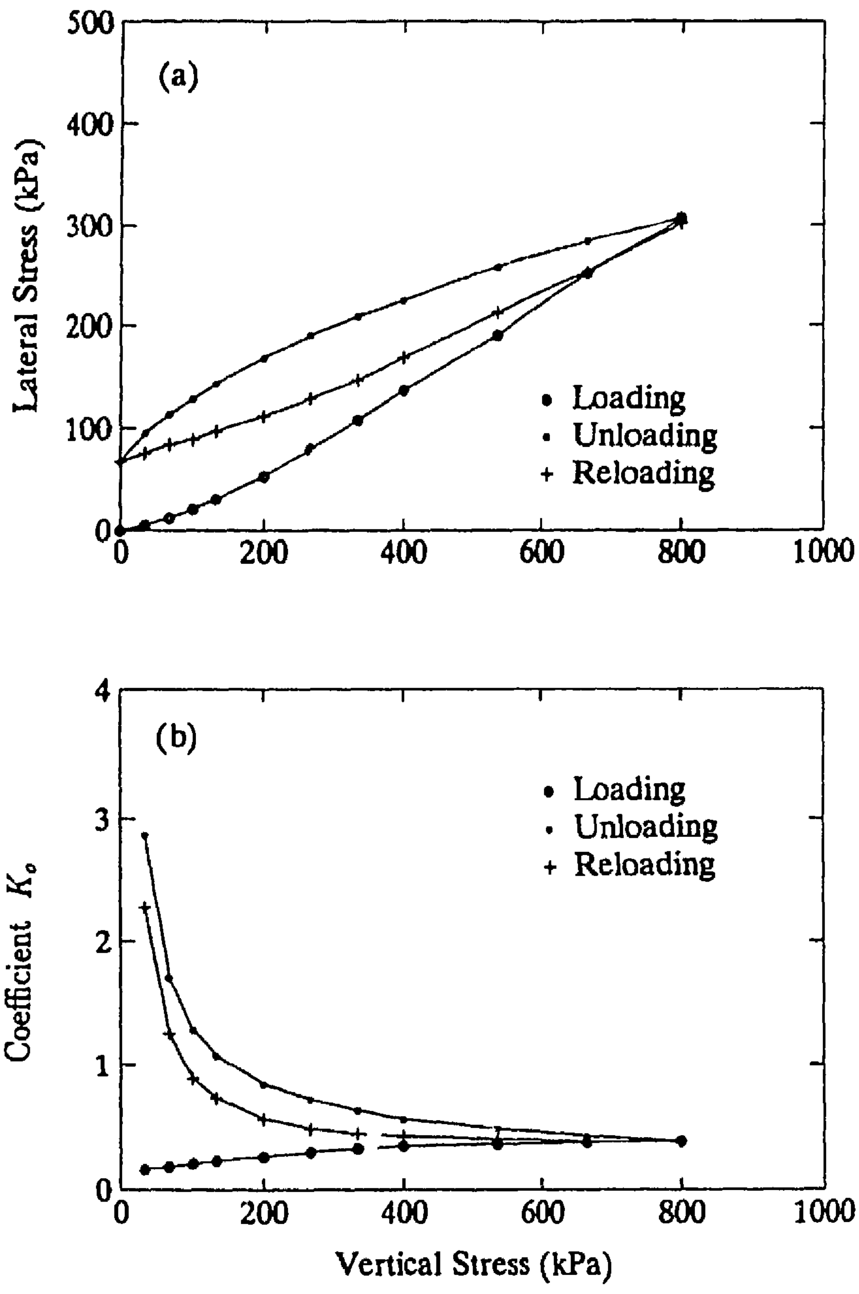


Figure A.11: Lateral Stress (σ_h) and K_o (CC = 1.0%, $\sigma_v \leq 800$ kPa)

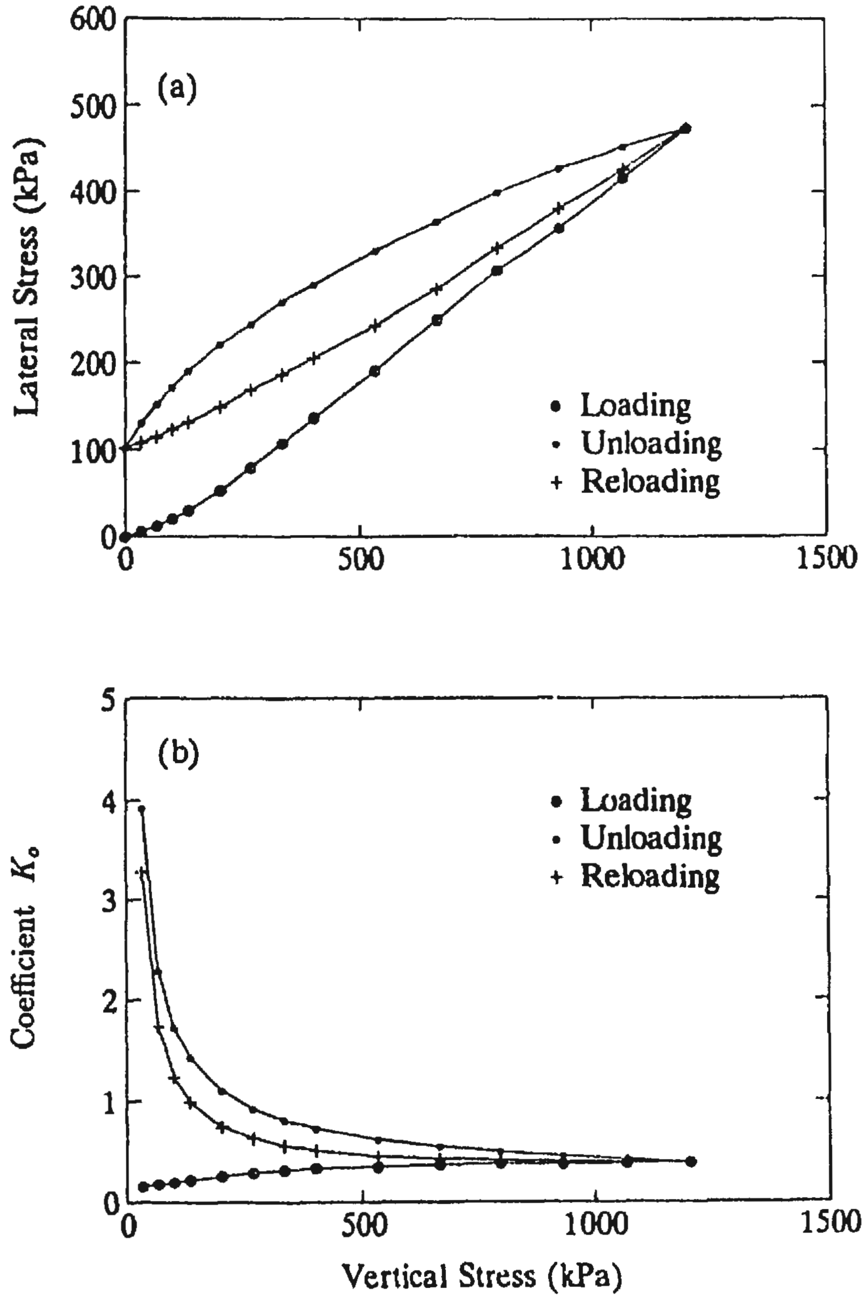
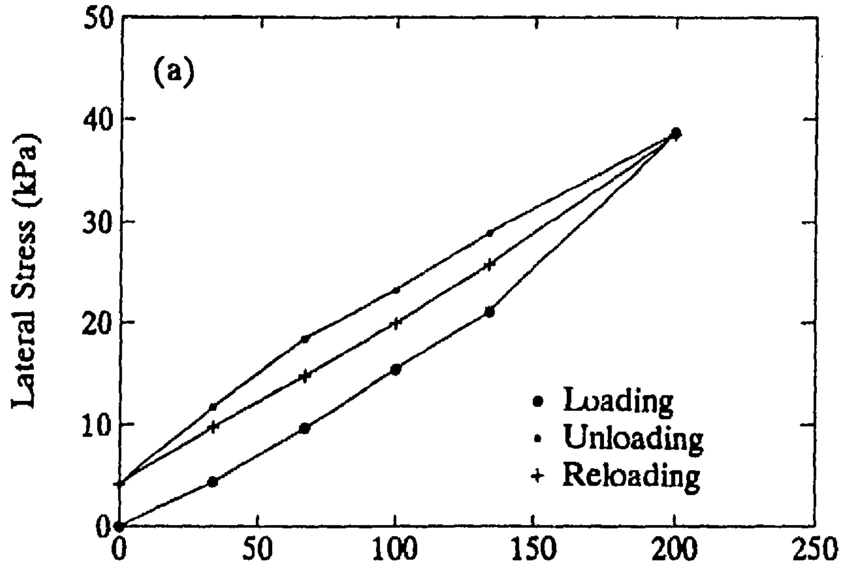


Figure A.12: Lateral Stress (σ_h) and K_o (CC = 1.0%, $\sigma_v \leq 1200$ kPa)



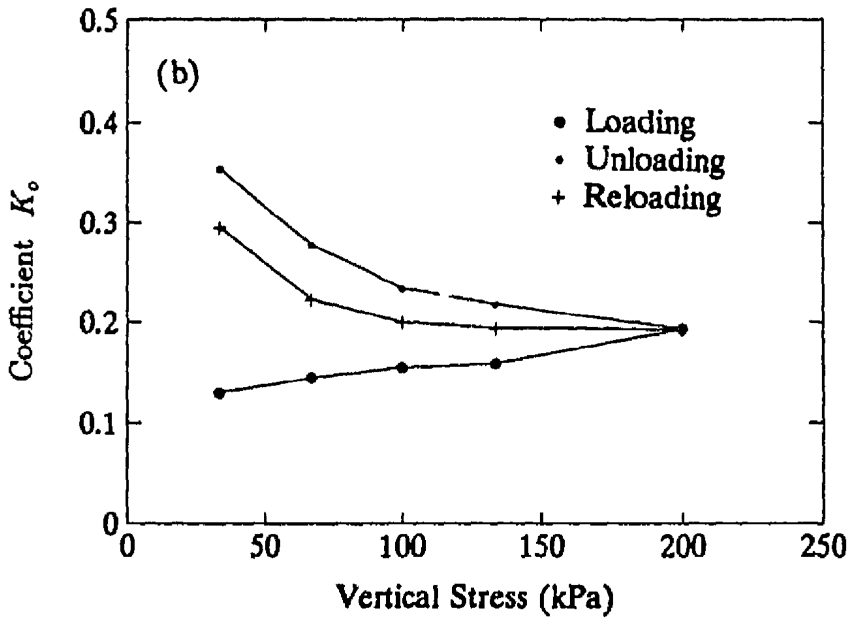
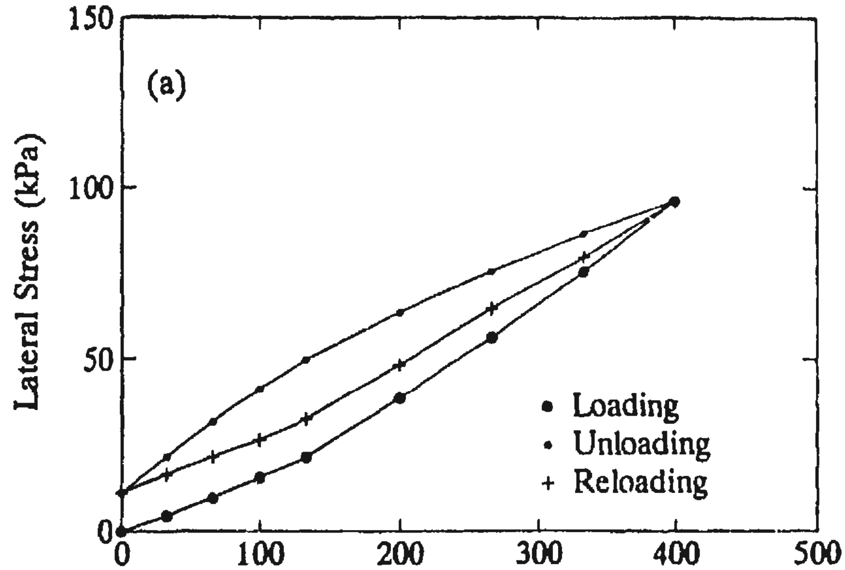


Figure A.13: Lateral Stress (σ_h) and K_o (CC = 2.0%, $\sigma_v \leq 200$ kPa)



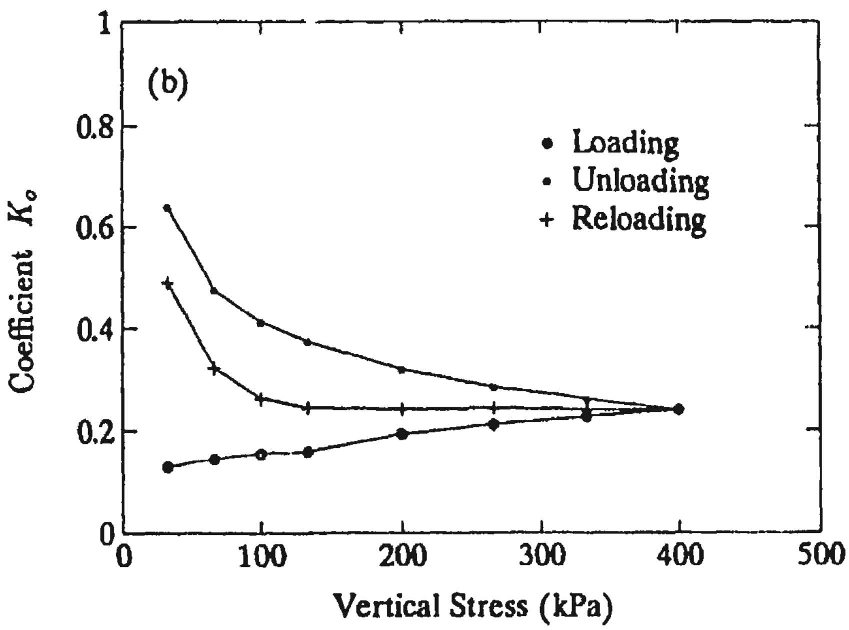


Figure A.14: Lateral Stress (σ_h) and K_o (CC = 2.0%, $\sigma_v \leq 400$ kPa)

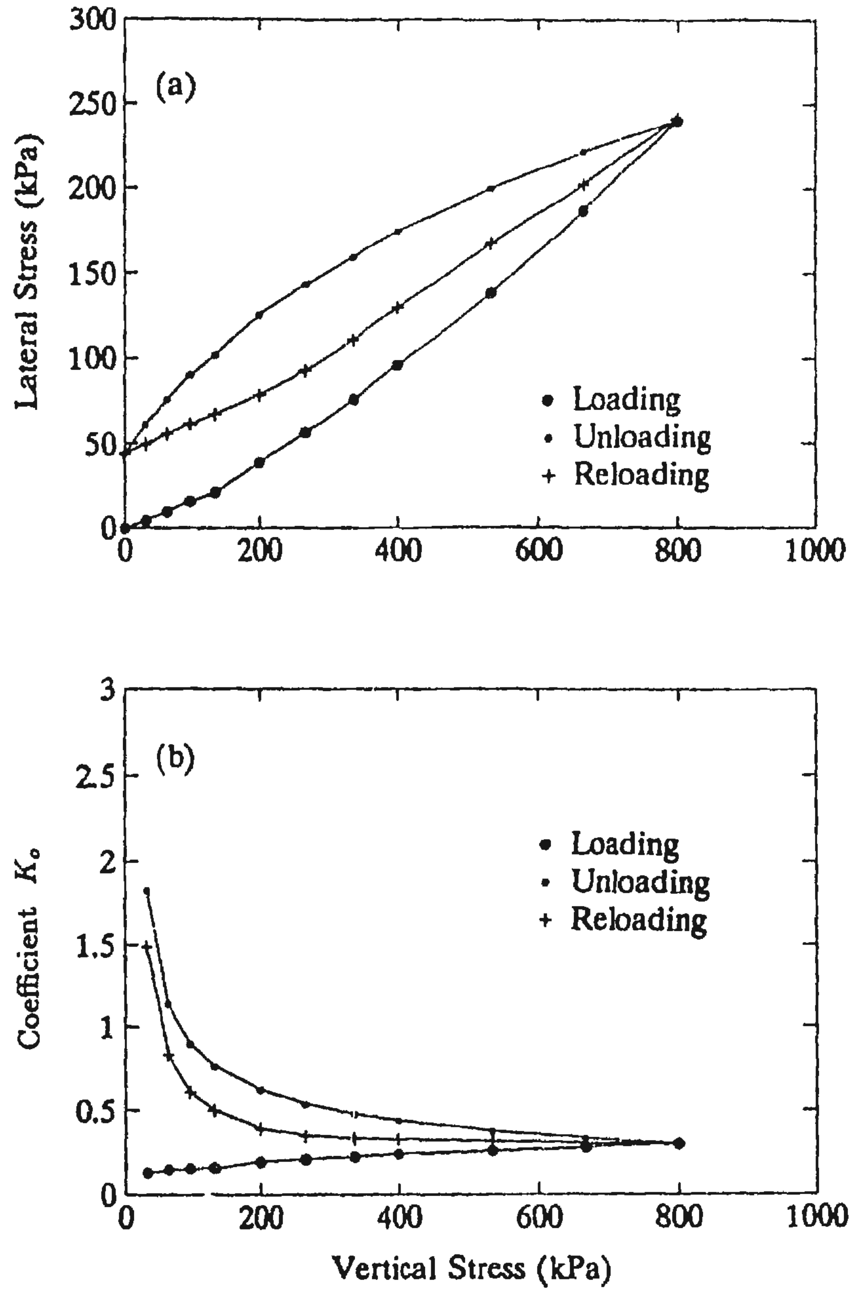


Figure A.15: Lateral Stress (σ_h) and K_o (CC = 2.0%, $\sigma_v \leq 800$ kPa)

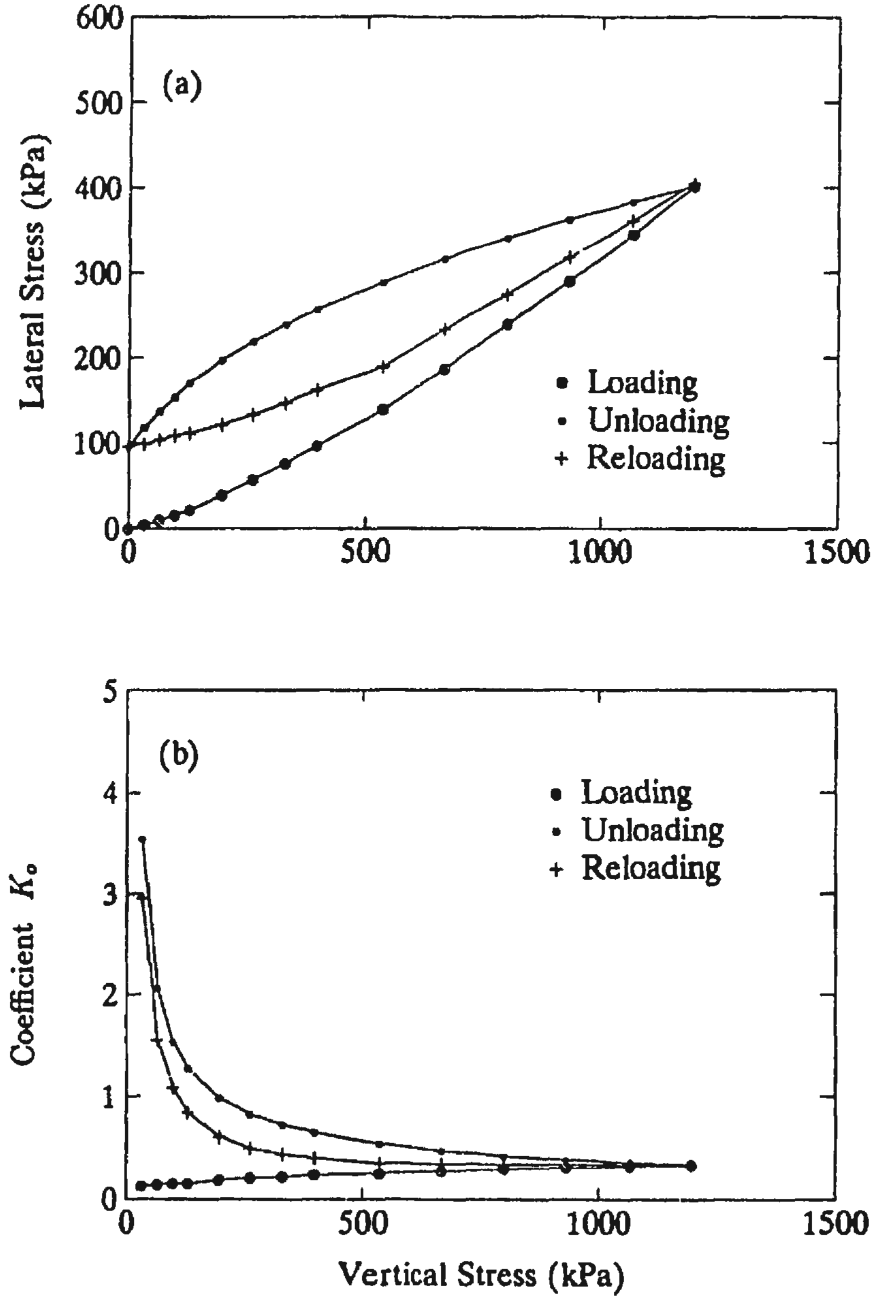


Figure A.16: Lateral Stress (σ_h) and K_o (CC = 2.0%, $\sigma_v \leq 1200$ kPa)

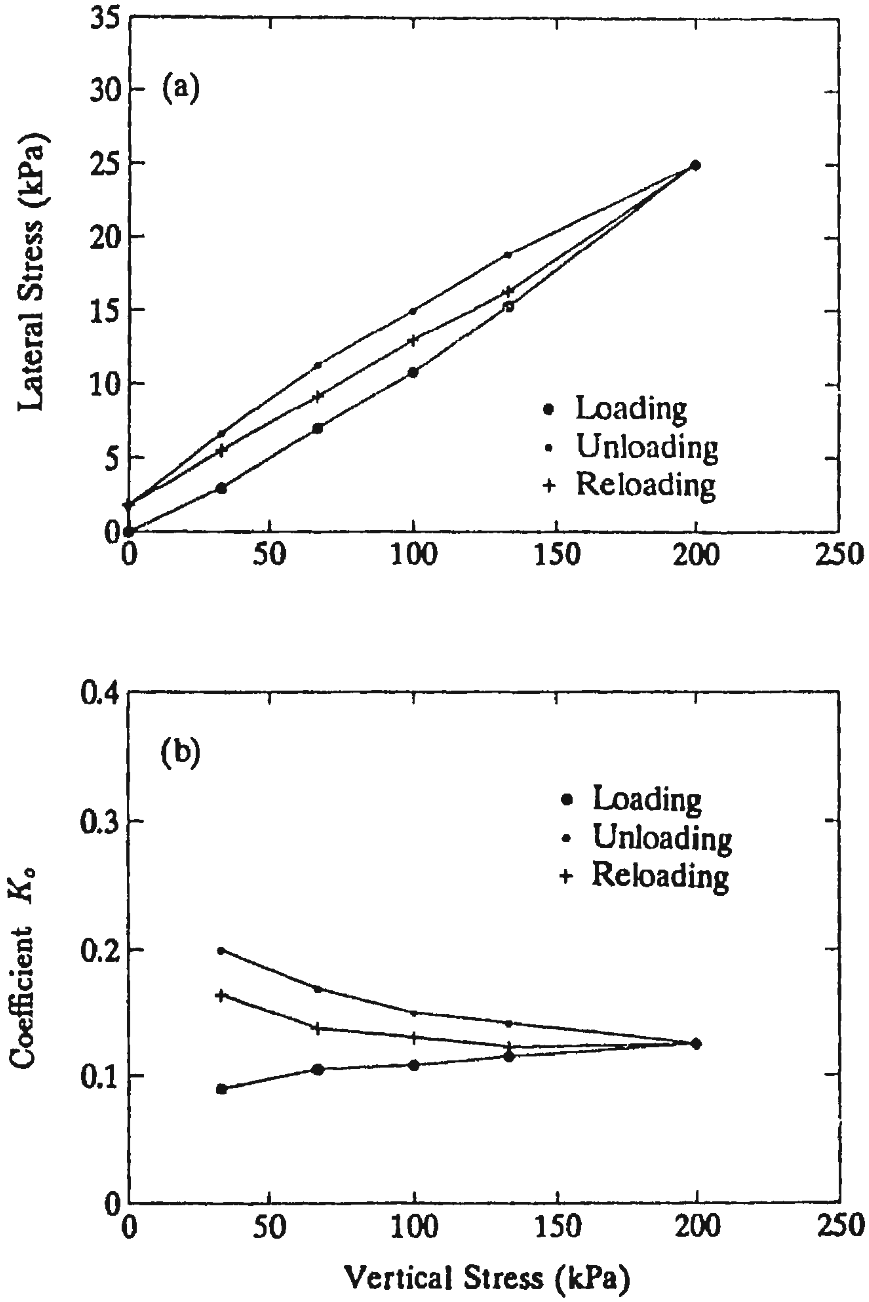


Figure A.17: Lateral Stress (σ_h) and K_o (CC = 4.0%, $\sigma_v \leq 200$ kPa)

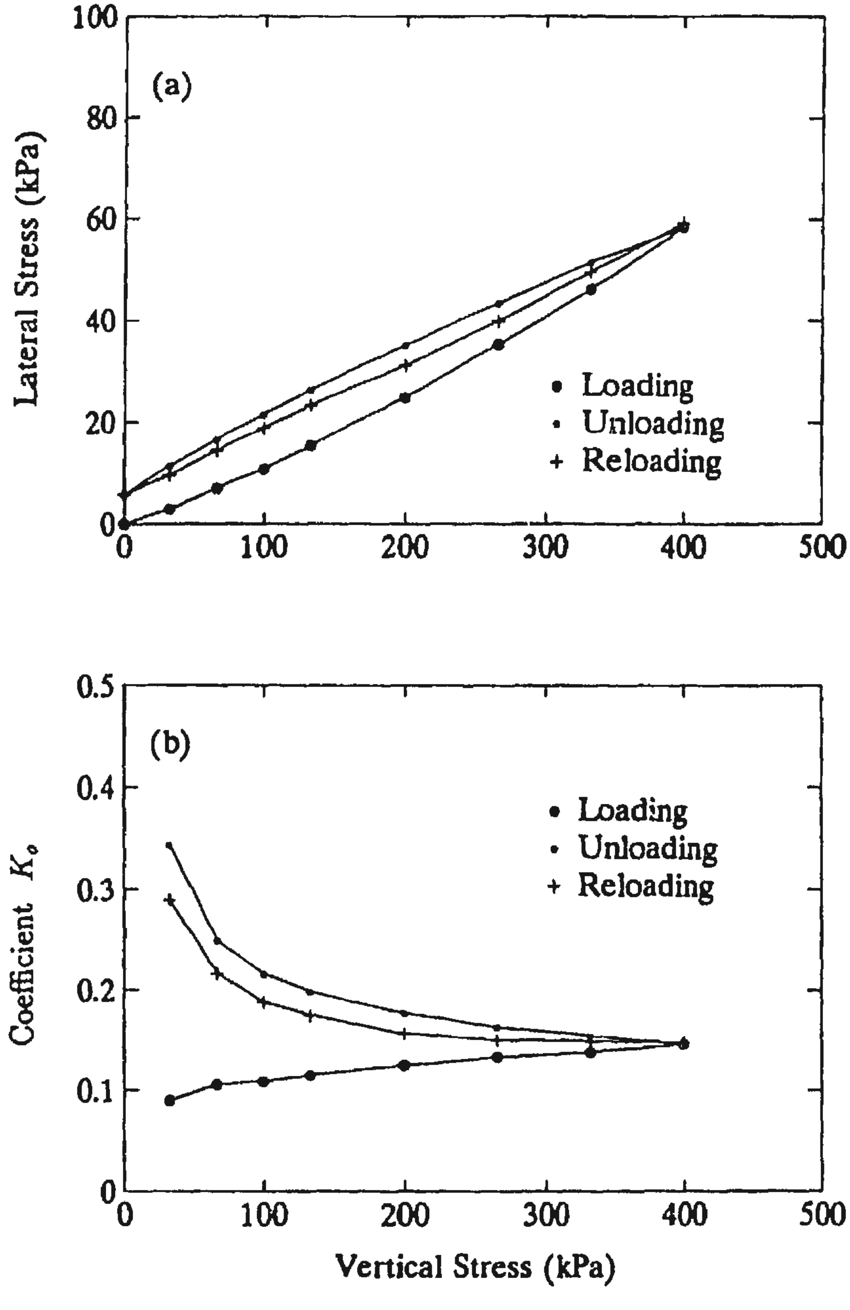


Figure A.18: Lateral Stress (σ_h) and K_o (CC = 4.0%, $\sigma_v \leq 400$ kPa)

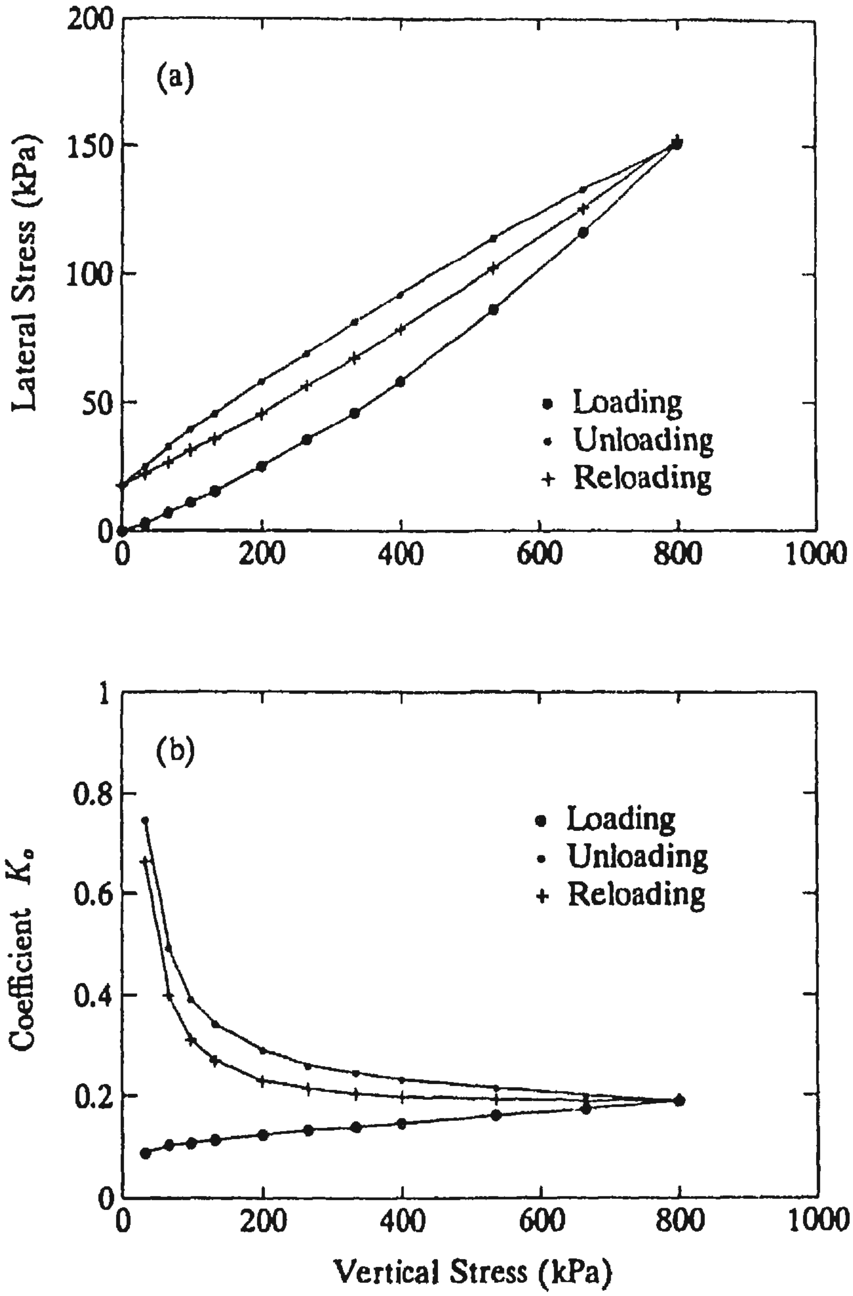


Figure A.19: Lateral Stress (σ_h) and K_o (CC = 4.0%, $\sigma_v \leq 800$ kPa)

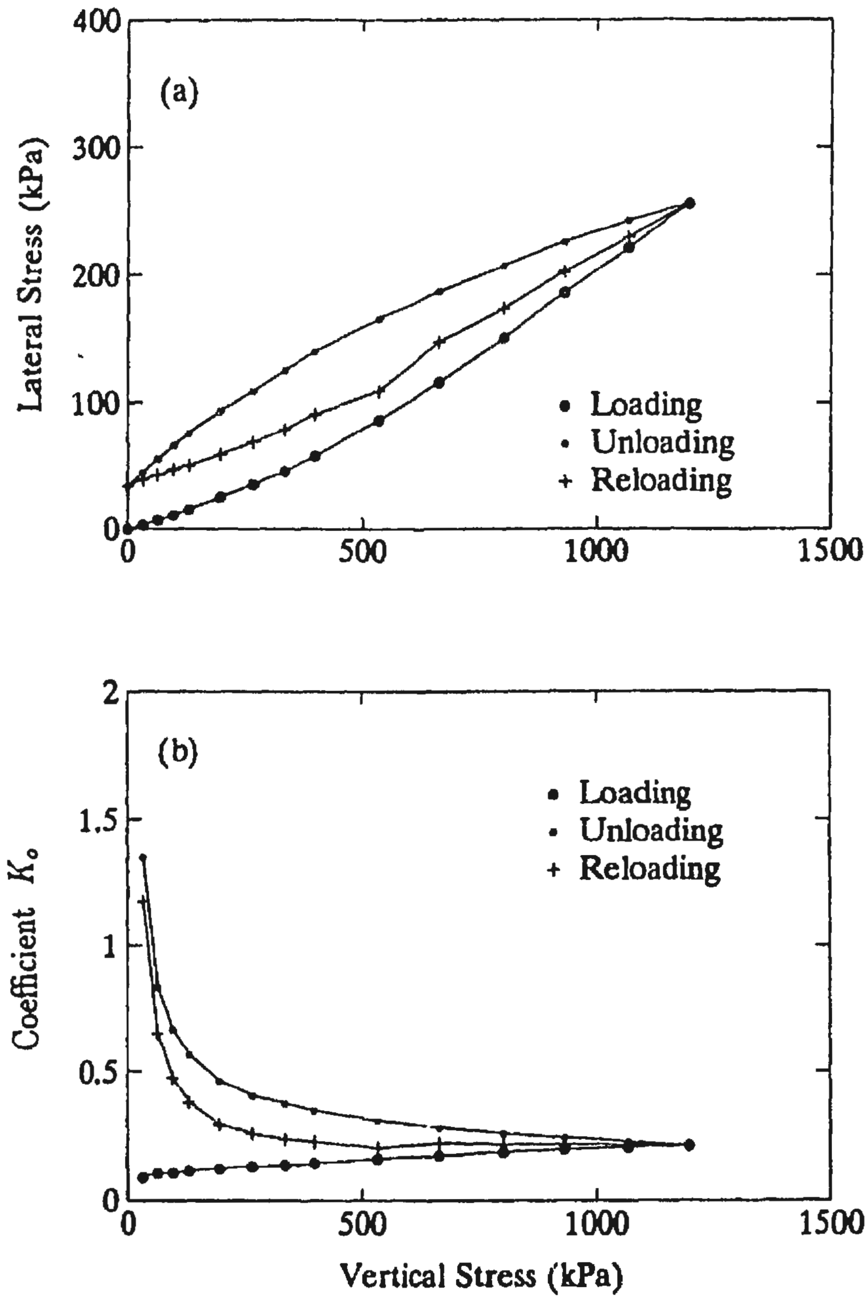


Figure A.20: Lateral Stress (σ_h) and K_o (CC = 4.0%, $\sigma_v \leq 1200$ kPa)

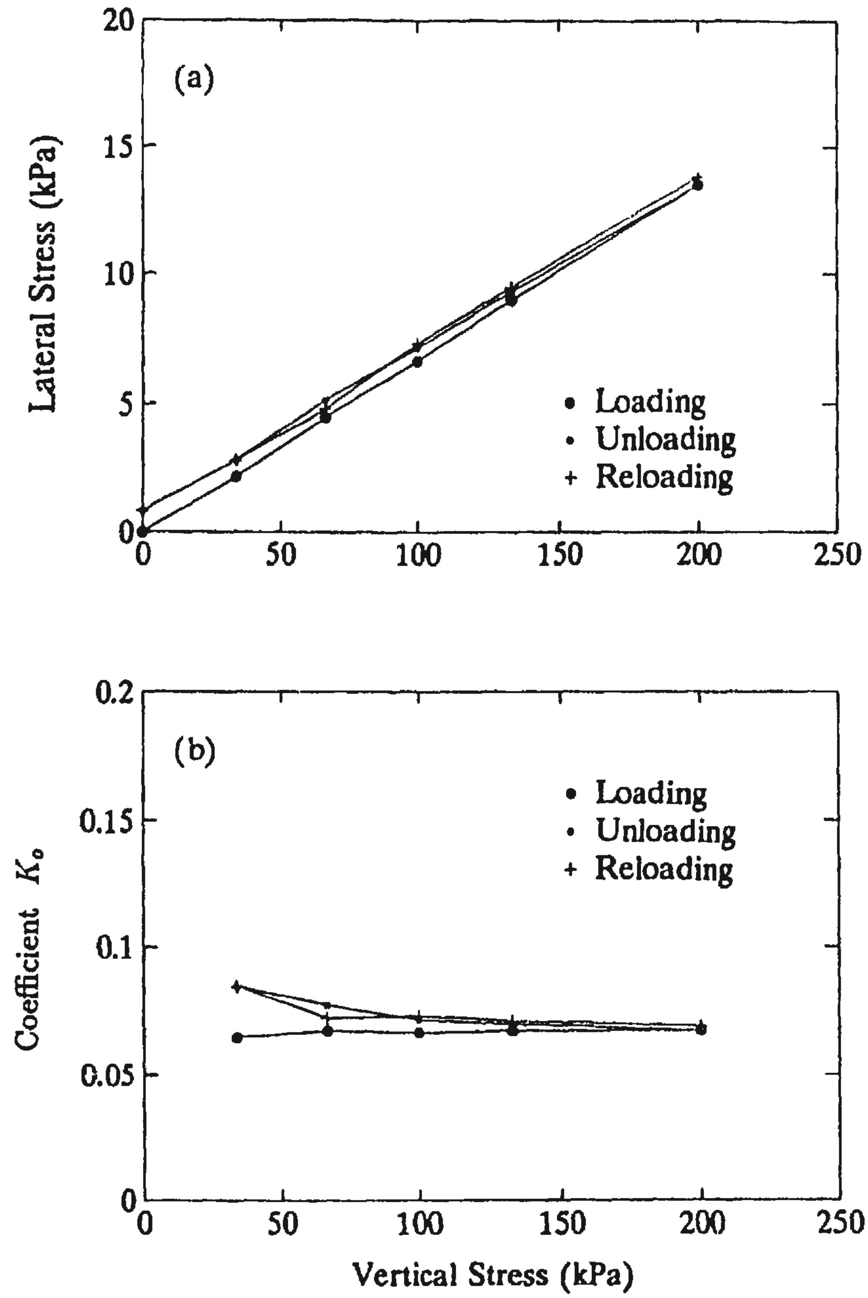


Figure A.21: Lateral Stress (σ_h) and K_o (CC = 8.0%, $\sigma_v \leq 200$ kPa)

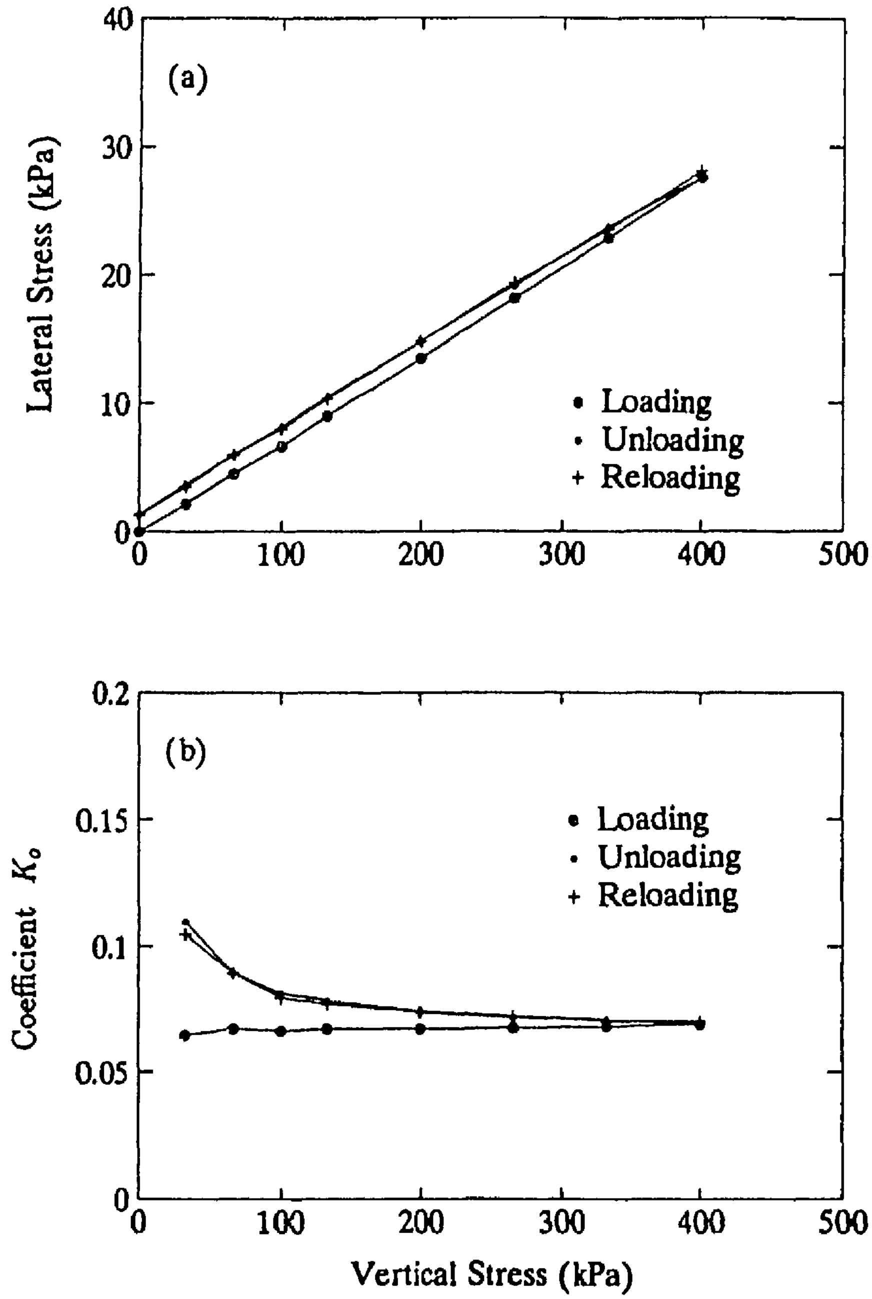


Figure A.22: Lateral Stress (σ_h) and K_o (CC = 8.0%, $\sigma_v \leq 400$ kPa)

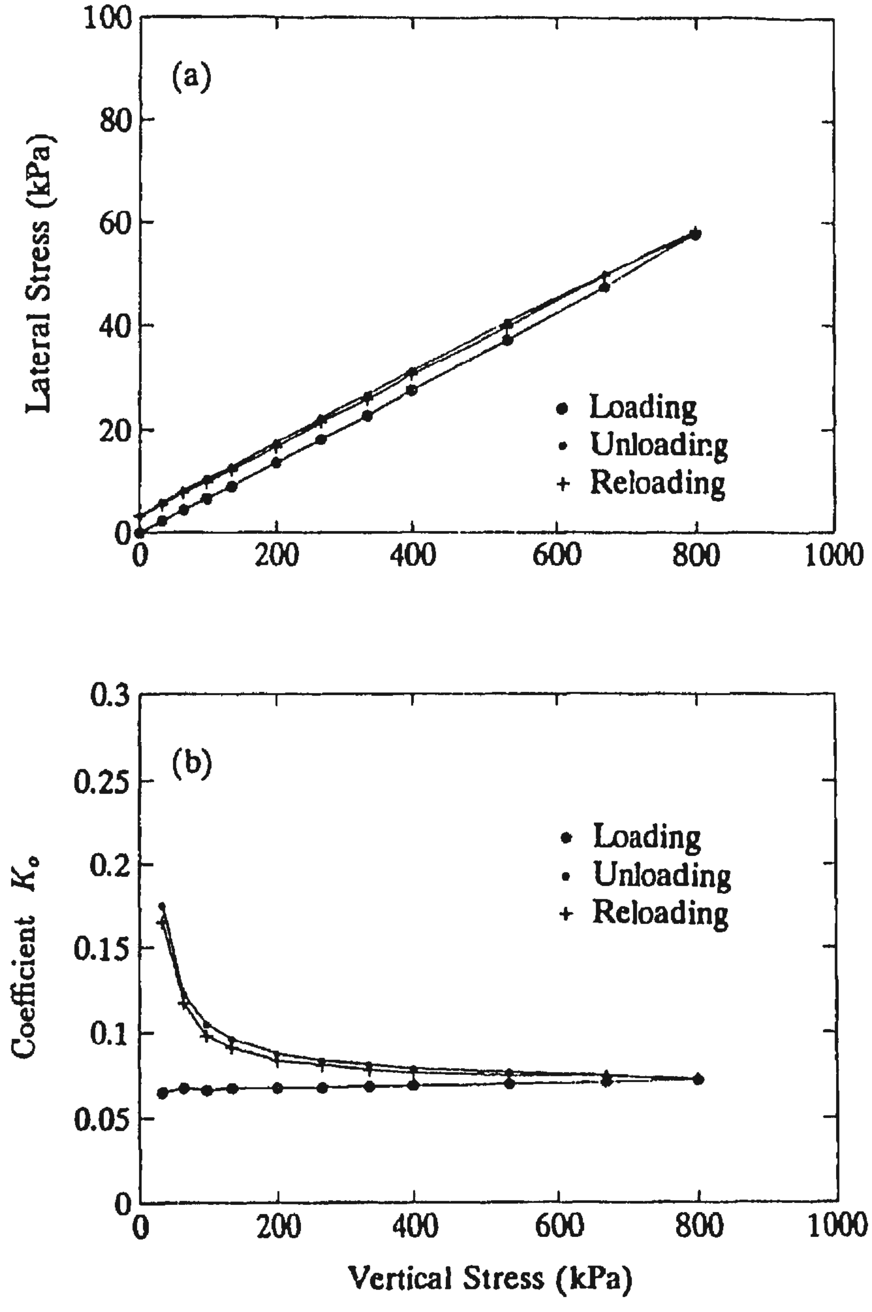


Figure A.23: Lateral Stress (σ_h) and K_o (CC = 8.0%, $\sigma_v \leq 800$ kPa)

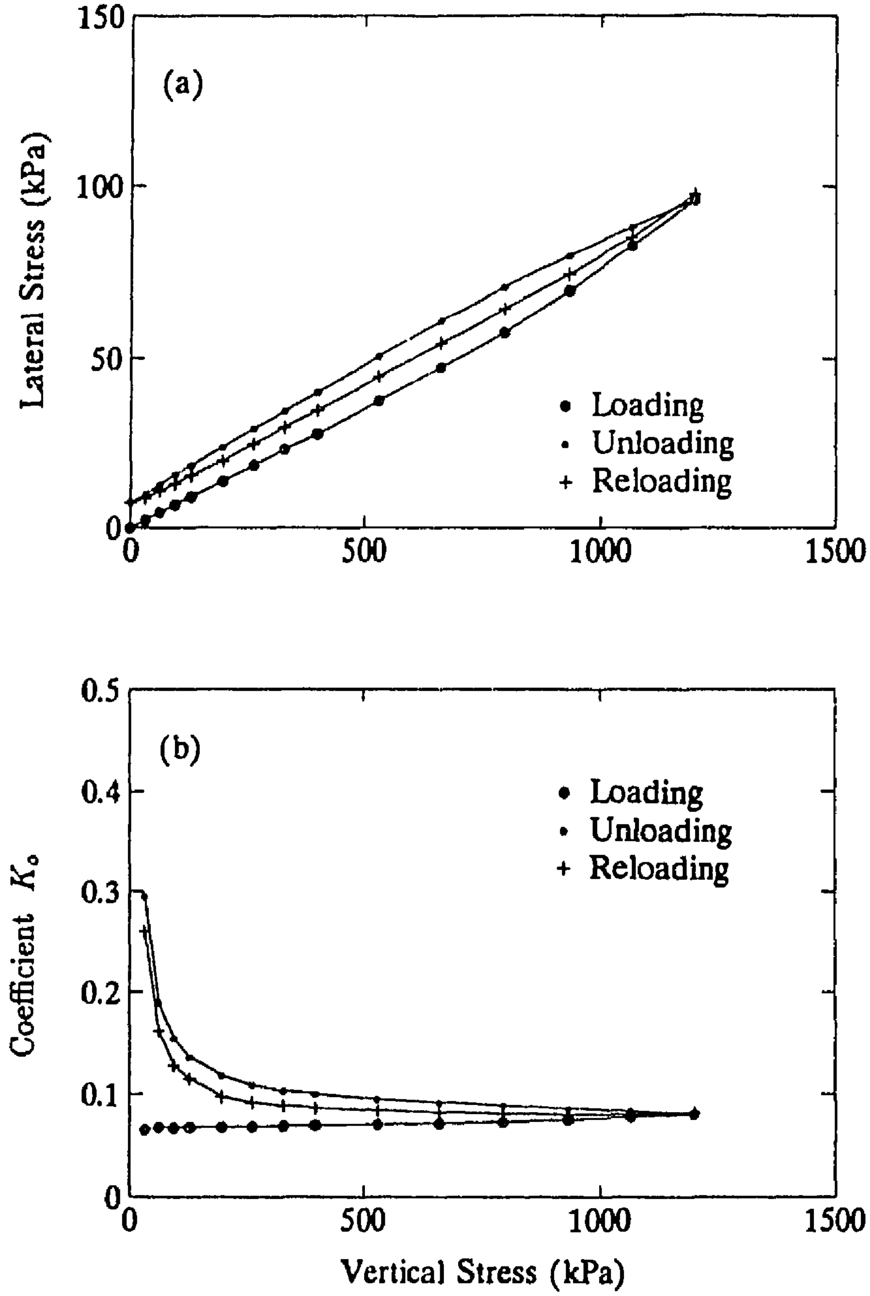


Figure A.24: Lateral Stress (σ_h) and K_o (CC = 8.0%, $\sigma_v \leq 1200$ kPa)