

CONSTITUTIVE MODELLING OF THE BEHAVIOUR OF
A SAND-BENTONITE MIXTURE

BY

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A thesis submitted to the Faculty of Graduate Studies of
the University of Manitoba in partial fulfillment of the requirements
of the degree of

DOCTOR OF PHILOSOPHY

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ABSTRACT

The Canadian concept for disposal of nuclear fuel waste proposes a compacted mixture of sand and bentonite (known as buffer) as one of several barriers limiting radionuclide escape to the biosphere. To ensure acceptable performance of the buffer, it is necessary to understand its stress- strain- time behaviour under rising groundwater pressure up to 10 MPa in the vault. High pressure triaxial laboratory tests have been performed at mean effective pressures up to 9 MPa, and porewater pressures up to 7 MPa at ambient temperatures. The results indicate that the strength of the buffer is dominated by the bentonite ($\phi' = 14^\circ$), and the material exhibits strain- softening behaviour in shear.

Three different approaches for constitutive modelling of the buffer behaviour are examined in this thesis. A three-modulus anisotropic hyperelastic model is proposed for the small strain range. This model accounts for the anisotropic nature of the buffer and permits coupling of mean pressures with shear strains, or deviator stresses with volume strains. A second three-function hypoelastic model is also developed to describe constitutive relationships for straining- to- failure. The third elastic- plastic model (belonging to the Cam clay family) accounts for non- reversibility, non- linearity and dilatancy in the plastic range.

In addition to these predictive models, a conceptual model is proposed based on critical state soil mechanics to provide a coherent framework for describing the behaviour of buffer compacted to different densities. Finally, the interactions between buffer, container, rock

and backfill are examined in non-linear finite element analyses using the proposed elastic-plastic model for the buffer. The preliminary results suggest that swelling of the buffer against compressive backfill could potentially produce large shear strains in the buffer.

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LIST OF SYMBOLS

A	total interparticle repulsion divided by total interparticle area
A, B	Skempton porewater pressure coefficients $\Delta u / q$, $\Delta u / \sigma'_{cell}$
a_m	fraction of the total interparticle contact area that is in mineral to mineral contact
C_c	ion concentration midway between two parallel clay particles (mol/l)
C_o	ion concentration in the free water (mol/l)
C_{ijkl}	complementary constitutive tensor
c	cohesion
$\bar{\sigma}$	modified (true) effective stress
D_{10}, D_{85}	particle diameter for which 10%, 85% of the material has smaller diameter
E	compression modulus
f	yield function
G	shear modulus
G^*	shear modulus in coupled hyperelastic model
g	plastic potential
G_c	clay specific weight
G_{sec}	secant shear modulus
G_t	tangent shear modulus
H, H^*	hardening parameters
J	coupling modulus
J^*	coupling modulus in coupled hyperelastic model

K	bulk modulus
K^*	bulk modulus in coupled hyperelastic model
K_{sec}	secant bulk modulus
M	invariant stress ratio at critical state
m	porewater pressure parameter u/p
N	equilibrium specific volume corresponding to unit effective mean stress
p, q	mean stress $(\sigma_1 + 2\sigma_3)/3$, deviator stress $(\sigma_1 - \sigma_3)$
p'_s	swelling pressure
R	total interparticle repulsion divided by total interparticle area
R	universal gas constant
T	consolidation duration
T	absolute temperature
u	porewater pressure
v	volume strain
W	strain energy function
V_c	clay specific volume - the volume of clay and void space occupied by unit volume of clay particles, excluding the volume occupied by sand
V_{ci}	initial compaction specific volume
CSL	critical state line
NCL	normal consolidation line
SEL	swelling equilibrium line
δ	increment
γ_c	effective clay density
k	swelling (recompression) index

σ	normal stress
σ_{mn}	stress tensor
$\bar{\sigma}$	final consolidation pressure increment
$\bar{\sigma}$	mineral to mineral contact stress
ϵ	shear strain
ϵ_{ij}	strain tensor
η	invariant stress ratio
λ	compression index
λ_c	clay compression index
ν	Poisson's ratio
Ω	complementary energy function
$\emptyset$	angle of shearing resistance
π	osmotic pressure

Subscripts and Superscripts:

$(')$	effective stress (osmotic pressure, tensor difference between the total stress and porewater pressure)
cell, cons,	cell, consolidation, failure
f	failure
e, p	elastic, plastic
1, 3, 50	major, minor, at 50% of undrained strength

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CHAPTER ONE

INTRODUCTION

1.1 Background

This study has direct application in the characterization of buffer material for the Canadian nuclear fuel waste management program. It is also relevant to engineering of pavements on expansive soils.

The Canadian concept for disposal of nuclear fuel waste proposes a compacted mixture of sand and bentonite as one of several barriers limiting radionuclide escape to the biosphere. The conditions under which the sand-bentonite mixture (buffer) must function will vary significantly with time. To ensure acceptable performance it is necessary to understand the stress-strain-time behaviour of the buffer.

Expansive soils have been recognized for many years as a source of significant engineering problems. For instance, the major cause of pavement failures on Manitoba and Saskatchewan highways and airfields is permanent deformation of subgrade due to high-volume-change characteristics of underlying soils. Research work for the nuclear waste containment program has shown that the addition of sand can significantly improve the compaction, permeability and shrinkage properties of expansive clays (Gray et al. 1984; Dixon et al. 1985). However, little information is available on stress-strain and strength properties of swelling clay-sand mixtures.

1.2 Canadian Nuclear Fuel Waste Management Program (CNFWMP)

Spent fuel from nuclear power plants is strongly toxic and a hazard to the social environment. Today Belgium, Britain, Canada, France, Italy, Japan, Netherland, Sweden, United States and West Germany have extensive nuclear waste management programs (Olivier 1986). The Canadian program was initiated by a Federal Government - Ontario Agreement, under which Atomic Energy of Canada Limited (AECL) is responsible for coordination, management and development of a program for the immobilization and safe disposal of nuclear waste.

1.2.1 General Concept

The Canadian program is based on the concept that the waste can be isolated effectively and permanently by disposal underground in deep stable geological formations (Bird and Cameron 1982). The disposal vault will consist of a number of mined chambers 500 metres to 1000 metres below the surface. A compacted mixture of sand and bentonite (known as buffer) is proposed as an effective barrier to limit radionuclide escape to the biosphere. The sand-bentonite material will fill annular spaces between corrosion-resistant fuel-waste containers and the walls of boreholes in the floor of the disposal vault as shown in Figure 1.1 (Rummery and Rossinger 1983).

The conditions under which the buffer must function will vary significantly with time. During emplacement, water pressures around the vault will be low, and temperatures at the surface of the container could approach 100°C. After vault closure, water pressures in the rock

and the pores of the buffer will approach 10 MPa (1000m of water head), a value that is high in normal geotechnical engineering terms. Under these very different conditions, the buffer must maintain low hydraulic conductivity and diffusion coefficient, support the container without excessive deformation, conduct heat effectively into the host rock, and avoid hydraulic fracturing induced by rising groundwater pressures in the rock.

The Canadian concept will be explored by conducting large-scale, in-situ experiments in the Underground Research Laboratory (URL), currently under construction at Lac du Bonnet, Manitoba. It will be the first such test facility built below the water table in previously undisturbed granitic rock. The URL will provide an appropriate environment for experiments to determine the mechanical response of the rock due to excavation and the thermal loading that would take place in the disposal vault. Beginning in 1990, a series of experiments will also be conducted to examine the interaction between a heater simulating the fuel waste container, buffer, rock and groundwater. These experiments are primarily designed to assess the behaviour and performance of the buffer material under in-situ, full-scale borehole emplacement conditions (Kjartanson and Gray 1987).

1.2.2 Vault Sealing Program

The vault sealing program includes research and development of engineered barriers required to retard the migration of radionuclides from a breached waste container to the biosphere. The development of the research program has been described by Bird and Cameron (1982). Figure 1.2 shows the areas of activity in this program. The main features are :

development

- (1) Buffer development
- (2) Backfill development
- (3) Grout, shaft and drift sealing development
- (4) Borehole sealing development
- (5) Underground Research Laboratory

Progress in the vault sealing program has been summarized recently by Lopez (1987).

1.2.3 Development of the Buffer Material

The objectives for development of a buffer material are: (1) protection of the container from mechanical damage during and shortly after emplacement; (2) provision of an optimum container environment; and (3) inhibition of mass transport of water and radionuclides in the vicinity of the waste container (Shemilt 1985). Clays have many properties suitable for achieving these objectives, including acceptable ductility and thermal conductivity, good bearing capacity, high sorption capacity, and availability. Bentonites with their swelling property of absorbing water into their crystalline structure, can potentially provide a chemically and mechanically stable

environment around the waste canisters.

Extensive research has been conducted on a range of candidate materials in the Whiteshell Nuclear Research Establishment at Pinawa, Ontario Hydro Research Division in Toronto, and various universities and institutions in Canada. Material testing has led to recognition of sodium bentonite as an appropriate clay buffer material (see Section 2.3.2), with the Avonlea deposits in Saskatchewan as a potential source. Laboratory compaction, shrinkage, and free swell tests on different clay-sand mixtures (Gray et al. 1984; Dixon et al. 1985) indicated that the addition of sand to bentonite provides a high density material with low sensitivity of compaction density to moisture content, and decreases the shrinkage potential of the mixture. Consequently, a 50/50 mixture by dry mass of crushed silica sand and sodium bentonite was selected as the reference buffer material (RBM).

Bentonites and sands form under very different depositional conditions. They will be mixed in controlled proportions to form the reference buffer. Because this is an engineered mixture, there is little existing geotechnical experience of how it will behave in the repository. To ensure acceptable performance it is necessary to understand the stress-strain-time behaviour of the buffer and how it varies with temperature and groundwater chemistry. The properties of the buffer have been studied recently by various researchers. Quigley (1984) determined the mineralogical composition; Radhakrishna (1984) the thermal properties; Dixon and Gray (1985) the engineering properties; Dixon et al. (1985) the compaction properties; Gray et al. (1984, 1985) the swelling properties; Cheung et al. (1985) the hydraulic and ionic diffusion properties; Yong et al. (1986) the creep

properties and Selvadurai et al. (1985) the container-buffer-rock interaction. Sun (1986) and Wan (1987) presented MSc. theses on work done in the geotechnical laboratories at the University of Manitoba on the stress-strain properties of the compacted buffer (Graham et al., 1986a, 1986b).

1.3 Pavements on Expansive Subgrades

Transportation in Western Canada faces severe engineering problems in dealing with expansive clay-shale deposits such as the Bearpaw shale and expansive lake-bottom clays such as those found at Winnipeg, Regina and Edmonton. The sediments that were carried into ancient glacial lake Agassiz were derived from the Cretaceous Bearpaw shale formation. This shale is rich in the clay mineral montmorillonite. The structure, the large surface area and the electrical charges on its surface give montmorillonite the capacity to attract large amounts of water causing it to swell.

In the United States of America, the Federal Aviation Administration, FAA, has recognized for many years that expansive soils are a major cause of premature airport pavement failures. In 1975, FAA initiated an extensive review of airport pavement design procedures on expansive soils and proposed development programs in various areas. Nevertheless, current FAA design procedures do not adequately treat the design of pavements over expansive soils, and further research is required to improve their performance. In south Africa, Israel and other countries such as India, problems with pavements underlain by expansive clay soils have led to a number of experimental studies of various pretreatment methods for road construction.

Recent studies have indicated that the addition of sand can significantly improve the compaction, permeability and shrinkage properties of expansive clays. However as noted previously, little information is available on geotechnical properties of swelling clay-sand mixtures.

1.4 Objectives of the Research

Physical and chemical properties of the buffer have been investigated extensively by a number of researchers in collaboration with Atomic Energy of Canada Ltd. However, little information is available on the mechanical behaviour of the compacted buffer as it supports the waste container in the changing vault environment. Understanding of the swelling properties of the buffer is essential for assessing long term deformations in the disposal vault.

The conditions under which the buffer must function vary significantly with time as discussed in Section 1.2.1. Buffer mass tests at the Stripa mine in Sweden (Pusch et al. 1985) and small-scale laboratory experiments (Radhakrishna 1985; Selvadurai and Au 1988) have indicated that the buffer could be expected to become fully saturated a few years after emplacement. It is therefore necessary to understand the constitutive behaviour of saturated buffer under in-situ stresses. The main objective of this study is to investigate the stress-strain-time properties of compacted buffer under combinations of high confining pressures and porewater pressures up to 10 MPa. The research will focus on various aspects of the buffer behaviour in both elastic and plastic ranges. The specific objectives of this research

are as follows:

- (1) To examine the validity of the effective stress principle for the buffer
- (2) To develop a conceptual model to describe the mechanical behaviour of the buffer in general, that is,
 - (a) To examine the pre-yield stress-strain relationships with time in terms of "volumetric" and "shear" behaviour
 - (b) To identify the first-time compression line and the swelling equilibrium line for the buffer in the high pressure range, that is, equilibrium swelling pressure - effective clay dry density relationship
 - (c) To evaluate the strength parameters of the buffer upon yielding and at the critical state conditions
 - (d) To investigate the post-yield behaviour of the buffer
 - (e) To study the influence of initial density and microstructural fabric on various aspects of the buffer behaviour
- (3) To establish constitutive relationships for the buffer behaviour
- (4) To conduct preliminary analytical studies of the rock - buffer - container - backfill problem using finite element method.

1.5 Scope of the Investigation

This research is one of a number of studies currently in progress in collaboration with Atomic Energy of Canada Ltd. in the vault sealing research program. To achieve the objectives of this investigation an extensive triaxial testing program has been undertaken consisting of four series. Series (I) was designed to study the effect of stress

level on the buffer behaviour. This series complements and extends research conducted by Sun (1986) on buffer compacted to 1.5 Mg/m^3 dry density. Series (II) formed triaxial specimens at different water contents corresponding to equilibrium states under the applied isotropic compression stress to study the influence of compaction density on the microstructure of the buffer. This complements similar tests at lower pressures reported by Wan (1987). Series (III) examined the properties of buffer specimens compacted to 1.67 Mg/m^3 dry density, corresponding to the proposed emplacement buffer density in the waste repository. Finally, Series (IV) investigated the applicability of the important "effective stress principle" to the buffer.

The testing program provided data for developing the necessary framework for describing the constitutive relationships of the saturated buffer under high confining pressures at ambient temperature. Later in the program the data were developed into elasto-plastic and hypoelastic constitutive models for compacted buffer that can be used in computer modelling. A preliminary numerical study of the proposed rock - buffer - container - backfill installation has been conducted using the finite element method (via a Critical State Program "CRISP") to provide better understanding of the mechanical interaction between different barriers in the vault.

This thesis begins with a comprehensive literature review in Chapter Two. The development of a computer-aided high pressure triaxial system for performing the necessary tests is discussed in Chapter Three. This is followed by the descriptions of experimental procedures and the testing program in Chapter Four. Test results are presented in detail in Chapter Five. The results are further discussed

in Chapter Six leading to the development of a conceptual model based on the critical state. Chapter Seven deals with various approaches to constitutive modelling of the buffer behaviour. The results of some preliminary finite element analysis of the container- buffer- backfill- rock problem will be discussed in Chapter Eight using an elasto-plastic soil model to represent the buffer behaviour. The summary of the research work, conclusions and suggestions for further research are presented in Chapter Nine.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Evidence suggests that expansive clay-based barriers can be effective in limiting the movement of water and air in an underground environment over long periods of time (Lee et al. 1985). Due to their low hydraulic conductivity and resistance against mineralogical changes bentonites have been used as liners to impound solid and liquid waste for several decades (Lundgren and Soderblom 1985). Swelling bentonites such as sodium bentonite have gained prominence in waste isolation projects in the last 30 years (Alther 1987). For example, as discussed in Section 1.2.3, the Canadian Nuclear Fuel Waste Management Program (CNFWMP) has selected swelling sodium bentonite as the clay constituent of the buffer material. Experimental studies of the buffer behaviour (Dixon et al. 1985; Graham and Saadat 1987) have shown that the principal engineering characteristics of the buffer are governed by the clay component. This chapter therefore begins with a general review of the properties of expansive clays known generically as smectites. Then, the behaviour of the clay-sand mixtures and their potential use in the geotechnical industry are outlined. Finally, different approaches to material characterization and modelling of the soil behaviour are discussed in detail.

2.2 Properties of Expansive Clays

The term clay refers to naturally occurring fine grained sedimentary deposits usually resulting from chemical weathering of igneous or metamorphic rock (Grim 1962). However we must distinguish between clay "size" and clay "mineral". A clay sized material in the engineering sense is one in which the constituent particles can be approximated by an equivalent sphere of less than 2 microns (0.002 mm) diameter. The equivalent sphere is a sphere of similar material with the same sedimentary velocity in water as the particle in question. However it should be noted that because of their mineralogy, clay particles are platy or flaky in nature (Yong and Warkentin 1975). Clay particles smaller than 0.001 mm are commonly known as colloidal size particles. Due to the size and shape of the colloidal particles their surface energy in a solution has a greater effect on their behaviour than normal body forces. For example, it is the surface effects that keep colloidal particles dispersed in solution contrary to the effects of gravitational body force, which would normally cause sedimentation. It is commonly believed that colloidal phenomena affect the behaviour of most clay particles.

This section begins with an overview of the mineralogy and fabric structure of clays, and then discusses the phenomenon known as diffuse double layer theory that governs the stability of colloidal clay particles. Finally, the applicability of the effective stress principle in expansive clays is examined.

2.2.1 Mineralogy and Fabric Structure

The shape of most clay crystals is like thin sheets. Rosenqvist (1955) made an analogy between the shape of a clay crystal and a razor blade, but perhaps a more useful analogy for the montmorillonites in this study would be corn flakes - the plates are not fully planar, but curved. They therefore form loose packing arrangements. The basic crystalline structure of clay minerals consists of combinations of silica tetrahedra (a silicon ion tetrahedrally coordinated with four oxygens) and alumina octahedra (an aluminum ion octohedrally coordinated with six oxygens or hydroxyls) (Yong and Warkentin 1975; Mitchell 1976). The silica tetrahedra combine to form sheets in a hexagonal network in which the oxygens at the base of the tetrahedra are each shared by the two adjacent silicon atoms. Similarly, alumina octahedra in combination form sheets composed of two layers of closely packed oxygens or hydroxyls with the aluminium atoms embedded in octahedral coordination. Clay minerals are formed by the stacking of combinations of these basic sheets with different types of bonding between them. The substitution of other ions of approximately the same size for aluminum and silicon (known as isomorphous substitution) results in different minerals.

Figure 2.1 shows the there are three major groups of clay minerals, kaolinite, illite and montmorillonite. Kaolinite has a basic and repetitive structure composed of a silica sheet combined with an alumina sheet. These basic layers are stacked on top of each other to provide a strong composite layer structure. Typical crystals may be 70 to 100 layers thick (Yong and Warkentin 1975). The layers are held

together by van der Waals forces and strong hydrogen bonds. This bonding is of sufficient strength that there is no interlayer swelling.

The basic structure of illite minerals is a sheet of alumina octahedra sandwiched between two sheets of silica tetrahedra. In the silica sheet some of the silicon atoms are replaced by aluminium atoms giving rise to a net negative charge in the layer. This charge is commonly balanced by potassium ions which fit into the holes in the external oxygen layers and provide a bond between the individual layers. This interlayer bond is not as strong as the hydrogen bonding in kaolinite, but is considerably stronger than the bonding in montmorillonite.

Montmorillonite has the same basic structure as illite, and consists of one alumina sheet held between two silica sheets. However, extensive substitution for aluminium and silicon within the lattice sets the montmorillonite minerals apart from illite. Bonding between neighboring layers of montmorillonite minerals is by van der Waals forces and by cations that may be present to balance charge deficiencies in the structure. These bonds are weak and can be easily separated by cleavage or adsorption of water or other polar liquids.

Figure 2.2 shows a schematic of possible structural arrangements of clay particles. In fresh-water conditions, for example, the net forces between the clay platelet surfaces are repulsive and the positively charged edges of particles are attracted to the negatively charged platelets to form a cardhouse structure illustrated in Figure 2.2 (b). If the pore water contains a high concentration of dissolved salts, the net force between the platelet surfaces can become positive. This results in the formation of the flocculated structure shown in

Figure 2.2 (c). This structural arrangement is stronger than the cardhouse structure in Figure 2.2 (b) and when a soil of this type is mixed with water the particles tend to remain held together in units called flocs (Keedwell 1984). If the pore water contains a low concentration of dissolved salts the particles will tend to the dispersed structural arrangement shown in Figure 2.2 (a). Dispersion can be achieved artificially by treating a soil with a deflocculant solution or by mechanical means. An example of the latter is the process of compaction where large deformations imposed on the soil tend to destroy the original structural arrangement of the particles and impose the parallel configuration shown in Figure 2.2 (a). In these conditions, the soil is typically sub-divided into randomly oriented domains with parallel orientations of platelets within the domains.

2.2.2 Diffuse Double Layer Theory

A colloidal clay particle suspended in water usually has an overall negative charge (Lambe and Whitman 1969). This charge can originate from the following situations:

- (1) The most common origin is the isomorphous substitution of one ion for another with similar size but different charge in the crystal lattice. For example, the substitution of aluminium for silicon in illite, or magnesium for aluminium in the alumina sheet, results in a charge deficiency of -1 for each substitution.
- (2) Imperfections within the crystal lattice including broken bonds at crystal edges.
- (3) The adsorption of ions on the particle surface arising from

chemical bonding or physical bonding, such as Van der Waals attraction and hydrogen bonding.

(4) Disassociation of hydroxyl ions from solution.

(5) Presence of amorphous organic matter.

Large negative surface charges associated with clay platelets provide the basis for the formation of the "diffuse double layer" of ions illustrated in Figure 2.3. Aqueous solutions contain positive cations and negative anions in solution. The negative charges of the clay particles attract the cations in the solution towards the clay surface. However, the cations are not simply adsorbed on to the surface of the clay particles due to the thermal energy of the cations which tends to counteract the electrical attraction of the clay particles. The result is that at any time the clay particles will be surrounded by an "atmosphere" of both cations and anions (Lambe and Whitman 1969).

The electric diffuse double layer in Figure 2.3 consists of one layer formed by the negatively charged particle surface and a second layer (the diffuse layer) formed by an excess of oppositely charged ions in solution near the particle. The excess ionic concentration in the overlapping diffuse double layers (Figure 2.2) sets up long range repulsive forces between clay particles which is identified by many researchers as the source of swelling pressure in montmorillonitic clays. The magnitude of swelling pressure depends on the distribution of charge density and electric potential in the diffuse double layer (described by the Poisson-Boltzman equation), and hence on the separation of the particles. Theoretical descriptions of the diffuse double layer were first developed by Gouy (1910) and Chapman (1913) and later modified by Stern (1924). Detailed developments of diffuse

double layer theory are provided by Mitchell (1976).

2.2.3 Osmotic Pressure Concept

Osmosis is the term used to describe the process by which a solvent passes from a solution of lower concentration through a semi-permeable membrane into a solution of higher concentration (Figure 2.4). A membrane is described as semi-permeable if it allows the passage of solvent but not solute. In order to prevent the flow of water in Figure 2.4 (a) a pressure imbalance equal to the osmotic pressure difference between the two solutions would have to be present. The osmotic pressure in clay-water mixtures can be determined from thermodynamic principles by the Van't Hoff equation:

$$\pi = R T (C_c - 2C_o) \quad [2.1]$$

where π = osmotic pressure

R = universal gas constant

T = absolute temperature

C_c = ion concentration midway between two parallel clay particles
(mol/l)

C_o = ion concentration in the free water (mol/l)

Numerous studies in geology have demonstrated the existence of osmosis in deep confined clay aquitards. Imperfections in the clay crystal lattice and isomorphous substitutions within the lattice produce large negative surface charge densities which are balanced by a diffuse layer of cations sorbed in a layer of "bound water" to the clay mineral crystal surfaces. The osmotic pressure concept (Bolt 1954; Mitchell 1976) is a technique used to calculate the repulsive stress between

clay particles. The overlapping diffuse double layers between two clay platelets are considered to behave as a semi-permeable membrane (Mitchell 1976). The higher (excess) ionic concentration in the bound water (Figure 2.3) causes an osmotic pressure or a suction (negative porewater pressure) on free water with which it is in contact. Therefore, the magnitude of swelling pressure depends on the distribution of charge density and electric potential in the diffuse double layer which are functions of surface charge density, surface potential, electrolyte concentration and valence, dielectric constant of the fluid, and temperature. The swelling pressure may also be influenced by the soil fabric and aggregate content. As the clay swells, the inter-particle distance increases, and the suction pressure decreases. At equilibrium, when the specimen is neither expanding nor compressing, the external pressure will balance the osmotic pressure.

Bailey (1965) and Mitchell (1976) reviewed extensive studies of compressibility and swelling of clays of varying pore fluid concentrations conducted during 1950's and 1960's. These works attempted to use the osmotic pressure concept to predict the void ratio versus pressure relationship for clays containing low concentrations of homoionic pore fluid solutions. The osmotic pressure concept has been found, however, to provide only a qualitative description of soil compressibility (Barbour 1987). The major limitation of this technique was the assumption of perfectly dispersed parallel arrangements of individual clay particles, and the assumption that the dominant interparticle force was the physicochemical repulsive force. Although these assumptions may be largely true in dilute, gel-like clay-water mixtures, they are not strictly valid for hard clays and dense

clay-sand mixtures.

2.2.4 Applicability of the Effective Stress Principle

The deformation of clays results mainly from the relative movement of particles rather than from the deformation of the particles themselves. The factors controlling such movements are: (1) the stress system applied to the soil element; (2) pore water pressure; (3) soil fabric structure and (4) properties of the contact zone between particles. Geotechnical engineering uses the concept of "Effective Stresses" to describe the behaviour of saturated soils. The concept states that when the area of contact between soil particles is small, the total stress at a point is the tensor sum of effective stress plus porewater pressure. This implies that the particles transfer forces at "points" or "edges" of contact, and that the void spaces are interconnected with ordinary unbound water or so-called "interstitial" water. However in bentonite, the clay particles are very small. Attractive and repulsive electrical forces exist between the particles and they are of similar or greater magnitude to body-weight forces. Attraction is caused mainly by Van der Waals' attractive forces between closely spaced neighbouring crystal structures. Arising from the previous discussion, the repulsive forces are primarily attributed to interaction between adsorbed double layers surrounding the particles (Lambe 1960; Seed et al. 1960). The magnitude of the attractive and repulsive forces depend on the valencies of the ions in the electrolyte, on its ionic concentration, and on the dielectric constant of the medium. These influence the distance of separation between the

clay particles; the balance between the attractive and repulsive forces; and hence the equilibrium voids ratio of the clay under given externally applied stresses (Section 2.2.2).

2.2.4.1 Modified or True Effect Stress Concept

Lambe (1960) expanded on Terzaghi's effective stress concept and related externally applied total stresses to the internal stresses in a particulate expansive soil system as follows:

$$\sigma = \bar{\sigma} a_m + u + (R - A) \quad [2.2]$$

and therefore

$$\sigma' = \sigma - u = \bar{\sigma} a_m + (R - A) \quad [2.3]$$

where σ = total stress

$\bar{\sigma}$ = mineral to mineral contact stress

a_m = fraction of the total interparticle contact area that is in mineral to mineral contact

u = equivalent pore pressure

R = total interparticle repulsion divided by total interparticle area

A = total interparticle attraction divided by total interparticle area

Based on this equation, the conventional effective stress σ' increases with repulsive forces and decreases with attractive forces. Experimental evidence presented by Balasubramonian (1972) and Chattopadhyay (1972) also indicates that the changes in the conventional effective stress ($\sigma - u$) is equal to change in $(R - A)$ when the volume of the sample was maintained constant. However, experimental studies by Sridharan et al.

(1973) have shown that increasing the dielectric constant of the pore fluid increases the net electrical repulsive stress ($R - A$) and apparently decreases the shear strength. In other words, they postulate that the conventional effective stress increases but the shear strength decreases. To resolve the apparent dilemma, Sridharan et al. (1973), Balasubramanian (1972) and Chattopadhyay (1972) suggested a modified or true effective stress concept expressed as

$$c = \bar{\sigma} a_m = (\sigma - u) + (A - R) = \sigma' + \sigma'' \quad [2.4]$$

where c is the modified (true) effective stress that controls strength, and compressibility; σ' is the traditional effective stress associated with the interparticle contact; and σ'' is the intrinsic effective stress ($A - R$) or net electrical attractive stress derived from the electrochemical charge fields round the particles. The shear strength is postulated to be directly dependent on the net attractive stress, i.e., the modified effective stress c .

This postulation appears at the first sight to be in some conflict with the common understanding (for example Mitchell 1976) that montmorillonite particles are not in mineral-to-mineral contact ($c = 0$) and that electro-chemical forces between the particles (osmotic pressures) balance the difference between externally applied loads and the free water pressure in the clay. The difficulty stems from the understanding of the nature of interparticle contacts in montmorillonitic clays and this is discussed in the following section.

2.2.4.2 Nature of Interparticle Contacts

Clay particles in their natural state are immersed in water and the positively charged ends of the water molecule dipoles (Yong and Warkentin

1975) are attracted and held to the negative ions at the clay surfaces by hydrogen bonding, resulting in the highly structured arrangement of water molecules shown in Figure 2.5(b), and known as the adsorbed water layer. This relatively strong hydrogen bonding restricts easy movement of water molecules, and therefore the adsorbed water possesses higher viscosity and density than those of the free water in interparticle spaces (Yong and Warkentin 1975). The layer of water molecules tightly bound to the particle surfaces by inter-atomic forces can not be removed by oven drying of clay at 105°C temperature. When the soil is not being subjected to change in stress level, the adsorbed water layer is believed to have mechanical properties similar to ice. In clays, contact zones may be either entirely composed of adsorbed water, or "hybrid" contact zones may exist which are neither wholly of adsorbed layer type contact nor the mineral- to- mineral type contact (Keedwell 1984). A mechanical property which distinguishes the adsorbed water layer contact from mineral- to- mineral is that the former is likely to possess thixotropic properties. That is, the tendency to regain strength with time. In a typical adsorbed water layer contact zone one would expect a rapid change in stress level to temporarily destroy the highly ordered arrangement of hydrogen and oxygen atoms causing this previously ice-like material to temporarily adopt fluid properties more like free water. In the mineral- to- mineral contacts the behaviour is likely to be viscous in nature due to the very high stress levels induced in all contact zones.

At the densities proposed for the buffer (Dixon and Gray 1985), it is likely that most of the water in the soil will be electrochemically bound to the mineral particles (Pusch 1982). The nature of the bound water (water-mineral contact) is very complex, transmitting both water and

mineral pressures. In montmorillonitic clays at water content equilibrium, there is little particle-to-particle contact (Mitchell 1976), and therefore $c = 0$ in [2.4]. In this case, the effective stress ($\sigma - u$) can be considered to be equal to the electro-chemical repulsion pressure ($R - A$). As in the case of more common clays with particle-to-particle contact, the total external pressure is the sum of the effective stress (now equal in magnitude to the electro-chemical repulsive pressure) and the free water pressure. The validity of the effective stress concept at water content equilibrium in sand-bentonite buffers is discussed in more detail by Dixon et al. (1986) and Graham et al. (1989).

In buffer mixtures where sand is added to the bentonite, effective stresses will in principle result from a combination of interparticle forces between sand grains and net repulsive forces between clay particles. Electron micrography on the Reference Buffer Material used in this program has shown (Figure 2.6) that the sand grains are dispersed through the soil matrix and do not contribute to the effective stresses that can be generated. It will be shown later that the strength of the buffer depends on the clay mineralogy, and the density of the the clay-water phase. It is not influenced by the sand content.

2.2.4.3 Influence of Porewater Chemistry

Osmotic consolidation of clays may occur as changes in pore fluid chemistry alter the electrostatic interactions between the clay particles (Figure 2.7 (a)). Barbour (1987) in developing an osmotic consolidation theory for clays treated the diffuse double layer as a separate phase in a three phase saturated system. The equilibrium void ratio e was

consequently related to two independent stress state variables (Fredlund and Morgenstern 1977), the conventional effective stress ($\sigma - u$) and the osmotic pressure of the pore fluid π , in a three-dimensional constitutive surface shown in Figure 2.7 (b). The figure summarizes one-dimensional consolidation data from tests on Na-montmorillonite with various concentrations of pore fluid (Mesri and Olson 1971) in one coherent framework. For a given pore fluid chemistry (and hence osmotic pressure), however, the equilibrium void ratio is only a function of the conventional effective stress ($\sigma - u$).

2.3 Behaviour of Clays and Clay-Sand Mixtures

Clay soils typically undergo volume changes due to environmental effects. For example, shrinkage and cracking occur due to drying and swelling takes place if rewetting occurs subsequent to drying. Soil movements in a climate with alternate wet and dry seasons leads to many serious engineering problems. These problems associated with volume changes arise not only from overall volume change, but also from uneven shrinkage or swelling of soils supporting loads. The compressibility of clays under pressure is influenced by the plasticity of the soil. High plasticity soils tend to be more compressible than soils with low liquid limits as shown in Figure 2.8 (McClelland 1967). Skempton (1953) presented approximate relations between void ratio and overburden pressure for clay and silt sediments as a function of the Atterberg limits. Figure 2.9 shows that the compressibility of clays and silty-clays increases with increase in Atterberg limits.

Experimental studies by Dixon et al.(1985) have shown that the addition of sand significantly improves the compaction, permeability and shrinkage properties of expansive clays. Earlier, Dumbleton and West (1970) studied the volume change and strength characteristics of different sand-clay mixtures. They used kaolinite or montmorillonite mixed with sand in various proportions. Figure 2.10 shows the relationship between the moisture content and undrained shear strength of both mixtures with different clay contents. It can be seen that the compressibility of the mixture increases with the increase in the clay content. As expected, montmorillonitic mixtures have higher equilibrium moisture contents.

The behaviour of sand-clay mixtures is examined here as related to two different applications. First, an overview of earlier research on the buffer at Atomic Energy of Canada Limited (AECL), University of Manitoba, and ISMES in Italy will be presented. Then, the problems associated with pavements and embankments built on expansive subgrades will be discussed along with conventional stabilization measures and advantages of sand mixtures.

2.3.1 Behaviour of Clay-Sand Mixtures

Each component of the multi-barrier system in the Canadian Nuclear Fuel Waste Management Program (CNFWMP) has been studied in research projects coordinated by Atomic Energy of Canada Limited (AECL). This thesis focuses mainly on the sand-bentonite buffer used to fill the space between the waste container and the rock.

2.3.1.1 Buffer Research at AECL

Research on the buffer at AECL has concentrated on engineering properties such as the compaction, swelling, hydraulic, and diffusion characteristics of the material. Before the selection of the Reference Buffer Material (RBM), extensive research was conducted on a range of candidate materials. Important features of the compaction and swelling behaviour of different candidate sand-clay mixtures will be discussed in following paragraphs.

Experimental studies of the compaction properties of sand-clay mixtures indicated that the addition of sand significantly increases the achievable compacted density (Dixon et al., 1985; Yong et al., 1986). Figure 2.11 (from Dixon et al. 1985) shows the dry density - water content relationships for both Na-bentonite and illite mixtures with different sand contents. For both clays, as the sand content increased, the maximum attainable dry density increased and the corresponding optimum water content decreased. However, for a given sand content illitic mixtures had higher maximum dry densities and lower optimum water contents than the bentonite ones. Unlike the illitic clay-sand mixture distinct optimum water contents and maximum dry densities were not always evident in Na bentonite - sand mixtures (Oldfield 1974; Pusch 1983). This unusual dry density - water content relationship is likely related to the high affinity of smectites for water. If the attached water has higher viscosity than free water, the compaction data indicate that, with bentonite contents of 50% and higher, the compaction energy is insufficient to overcome the high affinity of the adsorbed water for the mineral surfaces. In mixtures with less than 50% bentonite, a distinct optimum water content for the

maximum dry density was observed (Figure 2.11).

The shape of the compaction curves for the bentonite - sand mixtures has practical implications. If the water content (mass ratio of water to solids) is less than the optimum, the attainable dry density is not significantly sensitive to water content. During in-situ compaction, stringent control of water content would therefore not be required. This contrasts with the illitic clay - sand mixtures for which stringent control of water content would be needed to attain the highest possible in-situ densities.

The addition of sand also affects the hydraulic conductivity and mass diffusion through the clays. The effective porosity n_{eff} of a sand - clay mixture is the total pore space per unit volume of the mixture less the pore space occupied by surface or bound water. Oscarson and Cheung (1983) suggested that the mass diffusion coefficient depends strongly on this parameter. Figure 2.12 (from Dixon et al. 1985) shows the influence of sand content on the effective porosity of both bentonitic and illitic mixtures at their optimum water contents. For both mixtures, n_{eff} decreases as the sand content increases up to 50%. Increasing the sand content above 50% affects only marginal changes in n_{eff} . Therefore, addition of sand should decrease the mass diffusion coefficient of in-situ compacted clays and thus decrease the frost susceptibility of the mixture.

The influence of sand content on swelling and shrinkage characteristics of Na-bentonite - sand mixtures was studied by Gray et al. (1984) and Dixon et al. (1986). The tests were carried out using two different testing procedures, namely confined swell and one-dimensional compression tests. In confined swell tests, the

specimens were 32 mm diameter and 30 mm high. They were formed with different densities and water contents, and tested at constant volume. These specimens were given access to distilled water (at constant volume), and developed swelling pressures p'_s that could be related to the initial density of the mixture. The swelling pressures of sand-bentonite mixtures were found to be independent of the sand content and depend primarily on the effective clay dry density γ_c of the mixture with the sand acting apparently as a filler (Figure 2.13). Figure 2.14 (from Dixon et al. 1986) shows the influence of sand content on γ_c for compacted bentonite - sand mixtures with optimum water content. γ_c does not vary significantly until the sand content exceeds about 50%, above which γ_c (and presumably swelling pressure) decrease markedly. Figure 2.15 shows additional data from one-dimensional compression tests on different sand-bentonite mixtures with clay contents ranging from 25% to 100%. It is clear from this data that the relationships are approximately linear in double-logarithmic plotting (see also Wroth and Houlsby 1985). If the same data are plotted (Figure 2.15 (b)) in terms of the clay specific volume V_c rather than the overall specific volume V , then the four different lines in Figure 2.15 (a) simplify to a single relationship with a good combined coefficient of correlation, $R = 0.97$. Figure 2.15 (b) also shows the swelling pressure line established in Figure 2.13 from the other series of confined swell tests. (Note that $\gamma_c = G_c/V_c$ where V_c is the clay specific volume and G_c is the clay specific weight). The discrepancies between the two lines are attributed to the differences in clay microstructure resulting from two different testing procedures.

Figure 2.16 (from Dixon et al. 1985) shows volume changes in the

compacted Na bentonite - clay mixtures after drying at 105°C . The volume change is related to the effective clay water content, W_c (mass ratio of water to clay), in the mixtures. The data show that mixtures with more than 50% clay shrank considerably and that the amount of shrinkage for a given clay-sand mixture increased with W_c . Mixtures with less than 50% clay exhibited little volume change on drying. The results presented in Figure 2.16 from Dixon et al. (1985) are from dynamically compacted specimens. Results for statically compacted specimens were similar. It has also been found that shrinkage is virtually independent of dry density for specimens with the same moisture content.

2.3.1.2 Buffer Studies at the University of Manitoba

Research on the stress-strain-time properties of the buffer began at the University of Manitoba in mid-1984. It involved triaxial tests of the buffer compacted to 85% ASTM Modified Density at ambient temperature. The testing program (Sun 1986) identified (1) the pressure - volume strain relationship at pressures up to 1.0 MPa; (2) the shear stress - shear strain relationships at different confining pressures; and (3) the strength envelope for the buffer at confining pressures up to 1.0 MPa.

Later Wan (1987) showed that the procedure used for specimen preparation has a strong influence on the compressibility and shearing behaviour of the buffer. The triaxial test specimens in this program were compacted at different moisture contents in equilibrium with the test confining pressure. The testing pressure was also extended up to 1.5 MPa. Results from these two studies (Graham et al. 1986; 1987)

indicate that the behaviour of the buffer is nonlinear, stress level and time dependent, and slightly strain softening in shear. The strength of the mixture (Figure 2.17) is dominated by the bentonite constituent ($c' = 0$, $\phi' = 13^\circ$). However because of the limitations of the laboratory facilities at the time, the tests could not be performed under conditions similar to the anticipated in-situ conditions of 10 MPa pressure and 100°C temperature.

A computer-aided high pressure triaxial system (see Chapter 3) has been developed as a part of the author's research work to allow testing of the buffer at confining pressures up to 10 MPa. The work has led to the development of constitutive models for the buffer at ambient temperature in this thesis. Further development of this system is currently in progress to permit investigation of the buffer behaviour at temperatures up to 100°C at the University of Manitoba (Graham et al. 1988).

2.3.1.3 Research on Temperature Effect on Clays at ISMES

The Italian program has selected hard clay as a candidate host medium because of nuclear waste disposal in the Italian program for its favorable characteristic as a water migration barrier, its high capacity of ion sorption, and its excellent plastic properties and ductility. Expansive clay is self healing in the case of thermal or mechanical cracking. However, the very low permeability of hard clay (about 10^{-11} m/sec) and the clay mineral - water interactions may give rise to potentially adverse conditions in the clay deposit caused by heating. The nuclear waste acts as a heat source underground. Little is known about the effect of elevated temperature on the clay behaviour (see,

however, Campanella and Mitchell 1968; Yong 1967). In the range of temperatures (100 - 150 °C) and the depths of interest for nuclear waste disposal, no porewater vaporization occurs once the groundwater level has been re-established and the clay can be treated as a two-phase medium (Heremans et al. 1981). Recently Baldi et al. (1985; 1986) presented new experimental evidence of increases in porewater pressure caused by thermal expansion of water. The porewater pressure is enhanced by the unusual tendency of clay-water to reduce in volume at temperatures less than 120°C (Heremans et al. 1980) through changes in the thickness of the diffuse layer. For example, considerable excess porewater pressures were generated during heating of Orte clay in undrained condition under constant total isotropic stresses as shown in Figure 2.18. This effect has been found for overconsolidated clays and more appreciably for normally consolidated clays. In principle, the high porewater pressure build up in the field around the waste container could potentially result in large hydraulic gradients, loss of shear strength due to a reduction in effective stresses in clay, and to hydraulic fracturing. The excess porewater pressure requires a long time to dissipate due to the low permeability of the clay.

Baldi et al. (1985; 1986) reported the results of a series of tests on overconsolidated and normally consolidated remoulded Kaolin, remoulded Pontida, silty clay and intact Boom clay. Tests were performed using the high pressure and high temperature triaxial system developed at ISMES. Figure 2.19 (from Baldi et al. 1985) shows a typical relationship between volumetric strain of the clay and temperature during an isotropic loading cycle. The cycle consisted of

isotropic loading (0-1) at room temperature, heating under constant pressure (1-2), further loading (2-3) and unloading at constant elevated temperature, cooling at constant low pressure (4-5), and finally an isothermal reloading involving normal consolidation. The following conclusions have been drawn from the test results in drained and undrained conditions:

- (1) the thermally induced volumetric strains in clays were contractive in drained conditions
- (2) the negative thermal expansion coefficient of clays was found to be temperature dependent, and strongly stress level dependent
- (3) the magnitude of irreversible thermal volumetric strains was larger in the normally consolidated specimens than the overconsolidated cases
- (4) the limit of the normally consolidated range is sensitive to heating-cooling cycle in the drained conditions
- (5) in undrained conditions a significant porewater pressure build-up was observed

The experimental work by (Baldi et al. 1985; 1986) has also led to the development of a constitutive coupled hydraulic-mechanical model for the heated clay-water medium. The model has been implemented into a computer finite element code for both transient flow and undrained conditions, corresponding to long term and short term analyses respectively.

It is important to note that the research at ISMES is the only extensive triaxial testing program conducted on clays at elevated temperature. However, the clays tested at ISMES differ from the sand-bentonite buffer proposed for the Canadian program. Limited model

heater tests have been performed in the laboratories of Carlton University in Ottawa and Ontario Hydro in Toronto (Dr. J. Graham, personal communication).

2.3.2 Pavement Construction on Expansive Subgrades

This section commences with discussion of the engineering problems associated with pavement subgrades and embankments due to environmental effects. Then, conventional methods of pretreatment of expansive subgrades are reviewed. Finally a new approach for stabilization of expansive subgrades is proposed.

2.3.2.1 Problems Associated with Pavement Subgrades and Embankments

The potential strength and resiliency of the subgrade soil greatly influence the type and design thickness of pavement required to sustain the load. Particular attention should be paid to expansive subgrades which exhibit high swelling strains upon saturation and shrinkage when dried out. The two controlling distress zones in flexible asphaltic pavement are at the bottom of the asphalt layer and at the top of the subgrade (Yoder 1967). These distress features are:

- (1) Compressive strain at the surface of the subgrade which controls the permanent deformation (rutting) of the subgrade
- (2) horizontal tensile strain in the asphalt layer, generally at the bottom which controls the cracking of the asphalt layer (fatigue)

The predominant distress mechanism experienced on Manitoba and Saskatchewan highways and airports (Van Cauweberghe, 1985; Sauer, 1978)

is rutting of the subgrade. Since the subgrade strength is directly dependent on soil type and moisture content, performance of the pavement is therefore greatly influenced by in situ subgrade moisture content. In Manitoba, pavement subgrades are normally placed and compacted at or near their plastic limit which is about 25% . However, moisture contents under paved surfaces a few years after construction have been found to be as high as 40% (Ganapathy, 1986). This is the main reason behind permanent deformation of subgrades which results in pavement failures. Numerous field studies have been conducted which demonstrated the differential heaving of pavements and its correlation with subgrade moisture conditions. Results from the University of Manitoba field study of concrete road surfaces have been reported by Hamilton (1980).

Premature failure of airport pavements, manifested as surface irregularities, affects operational limitations, accelerates aircraft fatigue, and reduces safety. A major cause of such pavement failures in United States has been found to be the underlying expansive soils (McKeen, 1980). In most cases, volume changes in swelling soils produce unacceptable differential movements in pavement resting on the soil and cause surface irregularities and cracking. Cracking usually leads to increased differential movements in the pavements. Pavement failures under construction traffic loading has also been reported in parts of USA (Wahls, 1967) and Canada (Sauer, 1978). The objective of pavement design should be to incorporate methods in the construction process that prevent or reduce the effects of soil volume changes.

2.3.2.2 Environmental Effects on Pavements

One of the most important factors affecting pavement performance is that of environment, particularly water and frost action. In situ moisture contents under pavements are greatly influenced by precipitation, variation of surface water drainage and temperature condition. Variation of in situ moisture content can cause severe problems when dealing with expansive subgrades. Similarly, frost action could lead to frost heave and loss of subgrade support during frost-melt period.

2.3.2.2.1 Effect of Volume Change Related Soil Movements

When a paved surface is constructed on expansive clay soils, the normal sub-aerial evaporation of water from the soil is prevented, the moisture content increases and the soil expands. This results in the following functional and structural problems (Van der Merwe et al., 1980):

- (i) A general unevenness of the pavement surface which adversely affects riding quality
- (ii) Significant deformations occurring at certain areas of moisture content concentration such as culverts
- (iii) Shear and flexural failures due to excessive deformation where the road cover is insufficient

Unevenness of the paved surface is due to the following factors:

- (1) The expansive properties of the clay soil vary from point to point

- (2) The thickness of the clay is not constant
- (3) Variation of moisture content takes place along the edges of the pavement resulting in shrinkage and swelling of the clay causing longitudinal cracks near the outer edges of the pavement

In general, specific surface microtopographic features of the clayey subgrade soils are affected by clay content, mineralogical composition, climate, and site drainage conditions. Following construction of the pavement, the soil structure adjusts as the soil equilibrates under the new boundaries established by the pavement. The soil continues to respond to climatic and drainage conditions as modified by the pavement. The responses of three existing airport pavements to a wide range of climatic conditions have been reported by Mckeen (1980). The study included; a) comparison of field and laboratory suction measurements, b) relation of soil differential heave to pavement roughness.

Highly expansive (or compressive) soils occur extensively in Western Canada. The clay-shale bedrock in the Prairies are rich in the clay mineral montmorillonite and have produced recent glacial and postglacial deposits with liquid limits typically between 90 and 600; plastic limit between 20 and 25; and clay fractions greater than 80% . Figure 2.20 shows the swelling capacity of the Lake Agassiz clays at Winnipeg based on clay fraction and plasticity index. Figure 2.21 shows the swelling pressures that could be developed in these soils. Normally, pavement subgrades are placed and compacted at or near their plastic limit which is about 25% . Moisture contents under paved surfaces some years after construction have been found to be as high as 40% (Ganapathy, 1986). Given this large water intake in these clays

and the relatively light downward pressures exerted by the pavements, significant swelling and heaving occurs.

2.3.2.2.2 Effect of Temperature and Frost Action

Air temperature has a direct influence on the performance of the pavement and supporting medium in both flexible and rigid pavements. Sudden temperature variations accompanied by fluctuations in soil moisture cause deformations in the pavement system, which may result in cracking, spalling or even the blow out of some slabs. Theoretical and experimental evidence has confirmed that moisture flow can be set up in soils under temperature gradients. This flow, which occurs both in the vapor and fluid phases in the direction of decreasing temperature, can cause appreciable moisture transfer under certain conditions. Aitchison and Holmes (1961) have analysed the effect of vertical flow on suction profiles due to a net evaporative loss at the surface.

The effect of frost action on pavements is one of the most severe of the environmental factors. In its broadest sense, frost action includes both frost heave and loss of subgrade support during the frost-melt period. Frost heave may cause a portion of the pavement to rise, due to ice crystal formation in a frost susceptible subgrade or base course. Thawing of the frozen soil and ice crystals during the spring period may cause pavement damage under loads. This damage usually results in high maintenance costs. The mechanics of the frost-heaving phenomenon are extremely complex and include many factors. For ice lensing to occur three conditions are essential; 1) frost susceptible soil 2) freezing environment 3) continuous supply of

water. If any of the conditions is not present or can be controlled or eliminated then ice lensing will not occur. Results of studies conducted by the U.S. Corps of Engineers have indicated that frost-susceptible soils include all inorganic soils which contain greater than 3 percent by weight particles finer than 0.02 mm. Figure 2.22 shows the frost susceptibility of various types of soils. The percentage shown on this diagram is the recommendation of the Canadian MOT as to the reduction in strength that should be considered in the design of pavements on such subgrades. It can also be seen that the fine sand to clayey silt type of soils are the most susceptible due to strong capillary action. Such soils can draw up water from 3 to 4 metre below the pavement surface and cause the ice lenses to grow.

2.3.2.2.3 Practical Methods Of Minimizing Pavement Distortions

The compaction of soils which will eventually support pavement surfaces is one of the essential elements in the construction of embankments and subgrades. The pavement surface requires a foundation which does not settle or slump and a subgrade which does not heave or depress. The embankment must be constructed of suitable materials, on a stable foundation, compacted at a consistent level to prevent differential settlements and have stable side slopes. A local embankment subsidence of as little as 10 mm can have a dramatic effect on the safe operation of a vehicle travelling at 90 km/h. Therefore, the construction of a highway embankment demands a high level of quality control right from foundation level to subgrade level. The development of transverse bumps and ruts can usually be attributed to

deficiencies in the subgrade; the major portion of any rut usually occurs in the subgrade and is normally as a result of inadequate pavement structure or inconsistent subgrade.

Proper compaction of subgrades and base courses increases density with a consequent lower potential of moisture content, even in the event of subsequent saturation. Both of these factors result in an increase in strength. However, it has been found by field experience that it is impossible to compact some plastic clay (and therefore expansive) soils to 100 percent modified ASSHO maximum dry density, using conventional compaction equipment. Laboratory studies indicate that expansive soil samples compacted at relatively low moisture content will swell more, with subsequent losses in strength, than those compacted at higher moisture contents (Yoder, 1967). Swelling potential decreases as moisture content increases to about the moisture contents greater than optimum. Therefore, when dealing with highly expansive soils it is important to compact them at moisture contents near or slightly (about 1%) in excess of the optimum moisture content.

Other methods used to prevent pavement distortion and cracking were classified by Kassiff et al. (1969) into three main categories :

- (i) Replacement or improvement of the subgrade over the full active depth where severe climatic effects are experienced
- (ii) Increase of surcharge on the clay subgrade
- (iii) Minimization of moisture changes in the subgrade by prewetting, extensions of the surfacing period, construction of suitable impervious coverings and the judicious use of properly designed drainage and shoulders.

The replacement of the clay subgrade is generally not economically

feasible. The method of increasing the surcharge is mostly not practical, apart from the fact that the effect of the fill is inherently small since the pressure is about 17 kPa per metre of fill and the swelling pressures of expansive clays are in the order of 200 kPa to 1000 kPa. Blight and De Wet (1965) reported on a trial pavement section where a deep clay soil was successfully prewetted by introducing the water by means of 20 ft (6 m) deep boreholes. A method of prewetting the clay subgrade suggested by Williams and Simons (1963) is to clear the road reserve of vegetation and leave it fallow for a year before construction. Research carried out by De Bruyn (1973) showed that desiccation of the soil could be prevented to a large extent by removal of the vegetation. Furthermore, if the bare soil was covered by a layer of sand, about 150 mm thick, more rainwater is trapped and seeps into the soil. The sand also forms a very effective barrier against evaporation of moisture from the soil. Full-scale field studies conducted by Van der Merve (1980) indicated that the initial high variation in moisture content of the expansive roadbed material could be eliminated by prewetting process, thereby limiting road irregularities to movements caused by seasonal variations.

2.3.2.3 Conventional Soil Stabilization Measures

In many instances subgrade soils that are unsatisfactory in their natural state can be altered by admixtures, by the addition of aggregate or by proper compaction. Such stabilization measures could render the soil suitable for highway construction. In a broad sense, soil stabilization improves the soil so that it can be used for

sub-bases and bases. Yoder (1967) discussed various types of stabilization according to the properties imparted to the soil, for example, increased strength or improved workability through reducing the plasticity. Types of admixtures include cementing agents, modifiers, water proofing, water-retaining agents, water-retarding agents and miscellaneous chemicals. Each of these admixtures has its particular use and limitations. The cementing materials include portland cement, lime, a mixture of lime and fly ash, and sodium silicate. Mixing lime into fine-grained soil is probably the oldest of all site-improvement techniques. The amount of lime is usually 3 to 6 percent by weight to be mixed with soil thoroughly and quickly, otherwise the reaction will be incomplete.

Field experience on Saskatchewan highways has shown that the addition of small amounts of lime to subgrade soil considerably improves its resistance to deformation under dynamic loading (Sauer, 1978). Because of this phenomenon, lime has been used successfully on construction projects where the water content of the subgrade material has been considerably in excess of untreated optimum and the costs and difficulties involved in drying it have made the use of lime economically feasible.

The improvements in the engineering properties of a soil as lime is added are attributed to two basic reactions (Leonards, 1962):

(1) Colloidal-type reactions which occur almost immediately and result in immediate changes in soil plasticity, workability, and swelling properties (Thompson, 1968). It involves one of the following : a) ion exchange of calcium for the ion carried by the soil; b) depression of the double layer on the soil colloids as a result of the increased

cation concentration in the porewater; and c) expansion of the double layer of the soil colloids from the high pH of lime.

(2) Pozzolanic reaction (cementation) between the lime with the available reactive alumina or silica in the soil, or both.

When lime is added to a fine-grained soil, it generally increases the plasticity of low plasticity soils and decreases the plasticity of cohesive soils. They become friable and easy to work (Mateous, 1964; Marks, 1970). Lime reduces the maximum dry density and increases the optimum moisture content of clayey soils at a given compaction effort (El-Rawi, 1972; Marks, 1970, Ladd, 1960). The swelling characteristic of the soil is reduced and the shrinkage limit is increased when lime is added (Mateous, 1964).

2.3.2.4 Advantages of Sand-Clay Mixtures

It has been well established that portland cement or lime treatment improves the plasticity and strength properties of swelling soils. However, it increases hydraulic conductivity and potential for frost heave. In addition, such stabilization measures for pavement sub-bases and subgrades are quite costly. Thus, a more economical method of stabilization for expansive soils is needed.

Adequate compaction of subgrades and base courses for highways and airports is essential for satisfactory pavement performance. Compaction increase density and consequently the strength. However, it has been found by field experience that it is impossible to compact some clay soils to 100 percent modified AASHO maximum dry density, using conventional compaction equipment (Yoder, 1967).

As discussed in Section 2.3.1.1, experimental studies by Gray et

al. (1984); Dixon et al. (1985); Yong et al. (1986) have shown that the addition of sand significantly improves the compaction, permeability and shrinkage properties of expansive clays. The increase of the achievable compacted density improves the strength and stiffness characteristics of pavement subgrades. The presence of sand also reduces the mass diffusion through the clays (Gray et al. 1984), and therefore reduces the frost susceptibility, the magnitude of frost heave, and shrinkage characteristics of expansive subgrades.

2.4 Conceptual Framework of Soil Behaviour

Computers and modern analysis provide an important capacity for solving problems with complex geometries and material behaviour such as the rock-buffer-container problem in the Canadian Nuclear fuel waste Management Program. The principal difficulty in using these tools at the present time is to find constitutive models of the soil behaviour that are applicable and reliable. Theories of elasticity and plasticity have been widely used in geotechnical engineering for problems involving small and large strains, respectively. Eringen (1962), and Saleeb and Chen (1980) have reviewed various approaches commonly used for constitutive modelling. The principal effort in this thesis has been to collect data that could be used in existing, or newly developed, constitutive models. Thus, relatively little effort has been devoted to the detailed critical review of the state of the art constitutive modelling, or to the research leading to the development of new models. This section will concentrate on three types of constitutive models for soils, namely hyperelastic, hypoelastic, and elastic - plastic models. Finally, an overview of the critical state concept based on the theory of plasticity will be presented.

2.4.1 Hyperelastic Soil Model

A rational approach in formulating nonlinear elastic secant stress-strain models for soils can be developed on the basis of hyperelasticity theory (Chen and Baladi 1985). Here, the constitutive relations are based on the assumption of the existence of a strain energy function W in terms of strain invariants or a complementary energy function Ω in terms of stress invariants such that:

$$\sigma_{ij} = \partial W / \partial \epsilon_{ij} \quad \text{and} \quad \epsilon_{ij} = \partial \Omega / \partial \sigma_{ij} \quad [2.5]$$

in which W and Ω are convex functions of the current components of the strain and stress tensor respectively. Eq. [2.5] yields a one-to-one relationship between states of stress and strain. In addition to the reversibility and path independence of stresses and strains in hyperelastic type of elastic models, thermodynamic laws are always satisfied, and no energy can be generated through load cycles.

Simplified forms of hyperelasticity can be considered rational extensions of the linear theory of elasticity. For instance, a linear anisotropic elastic model has been proposed by Graham and Houlsby (1983) which involves only three material constants. To construct a secant constitutive relation for coupled or uncoupled volumetric and deviatoric stresses and strains, strain- or stress-dependent bulk and shear moduli has been widely used (Domaschuk and Valliapan 1975; Hanrahan 1985). More complicated models based on the classical theory of hyperelasticity, have been formulated by Evan and Pister (1966) and subsequently used by Ko and Masson (1976), and Saleeb and Chen (1980), among others, in soil mechanics.

The hyperelastic formulation can be quite accurate for soils straining in proportional loading. However, hyperelastic type models

exclude any inelasticity in solid behaviour and thus it is incorrect to use them to describe the unloading. Another problem is that too many parameters are needed to form a nonlinear hyperelastic model, even when initial isotropy is assumed.

2.4.2 Hypoelastic Soil Models

An obvious shortcoming of the hyperelastic (nonlinear elasticity) models discussed in the previous section is the path-independent behaviour implied in the formulation. An improved description of the soil behaviour is provided by the hypoelastic formulation (Truesdell 1955 ; Treuesdell 1965) in which the incremental stress and strain tensors are linearly related through variable material response moduli that are functions of the current state of stress or strain state.

2.4.2.1 Hypoelastic Formulation in General

Coon and Evan (1971) suggested the following formulation for time-independent applications with monotonic loading where creep and strain rate effects may be neglected:

$$d\epsilon_{ij} = C_{ijkl}(\sigma_{mn}) d\sigma_{kl} \quad [2.6]$$

Here C_{ijkl} are complementary constitutive tensors (or moduli) which depend on the current stress levels. Eqn.[2.6] gives a general linear relationship between incremental strains and incremental stresses which may then be integrated to determine a required stress-strain relationship.

These incremental stress-strain relations provide a mathematical description for materials with limited "memory" of previous loading history. The general hypoelastic model depends therefore on stress history or stress path. In the linear case for which $C_{ijkl}(\sigma_{mn})$ is a constant, the hypoelasticity degenerates to hyperelasticity (Chen and Baladi 1985). In hypoelastic formulations the tangential stiffness $[C]$ is identical in loading and unloading. This reversibility requirement, only in the infinitesimal (or incremental) sense, justifies the use of the term hypoelastic or minimum elastic.

There are two important features associated with hypoelastic modelling (Chen and Baladi 1985). First, in the nonlinear range, the hypoelasticity models exhibit stress-induced anisotropy. This anisotropy implies that the principal axes of stress and strain are different, introducing a coupling effect between normal stress and shear strain. The second feature is the distinction made between loading and unloading under uniaxial stress conditions. However, under multiaxial stress conditions, the hypoelastic formulation provides no clear criterion for loading or unloading. Thus, a loading in shear may be accompanied by an unloading in some of the normal stress components. Assumptions are therefore needed for defining a suitable loading-unloading criterion.

Two different types of hypoelasticity models have been developed from the general framework given by [2.6]. One type expresses C_{ijkl} in a tensor stress series (Coon and Evans 1971; Yin 1984). Obtaining the resulting material constants is difficult, sometimes impossible. The second type makes a further assumption to reduce the required number of stress level dependent parameters C_{ijkl} in [2]. These parameters can be determined by differentiating curves fitted to experimentally

observed stress-strain data. Examples of this type are: simple incremental models by Kondner (1963), Kulhawy et al. (1972); the E, ν model of Duncan and Chang (1970); the K, G model of Domaschuk and Valliappan (1975); the three modulus model of Yin and Yuan (1985a,b); and models with more than three moduli, for example by Darve et al. (1986). Models of this type are attractive from both computational and practical viewpoints (Chen and Saleeb, 1982). Other examples of the classical formulation and applications of the first-order hypoelastic models can be found in the papers by Tokuoka (1971), Davis and Mullenger (1979) and Desai (1980). However, the practical usefulness of the hypoelastic models is limited by the nature and the number of tests required to determine the material constants.

2.4.2.2 KGJ Hypoelastic Model

A new hypoelastic model has recently been developed by Yin et al. (1988) in connection with the buffer study presented in this thesis. The model incorporates non-reversibility, nonlinearity, dilatancy, and the related phenomenon which produces shear strains from mean stress changes. It uses three stress dependent modulus functions that will be referred to subsequently as "moduli". These are (1) the bulk modulus K, (2) the shear modulus G, and (3) a coupling modulus J. The constitutive relationships can be expressed in a simple incremental form as follows:

$$\begin{cases} d\epsilon_v = \frac{1}{K} dp' + \frac{1}{J} dq \\ d\epsilon_s = \frac{1}{J} dp' + \frac{1}{3G} dq \end{cases} \quad [2.7]$$

The bulk modulus K represents the volumetric stiffness of the clay with respect to dp' ($K > 0$). The shear modulus G controls shear deformations with respect to dq ($G > 0$). The coupling modulus J

accounts for the volumetric strain produced by an increment dq in deviator (shear) stress, and the shear strain produced by an increment dp' in mean effective stress. The formulation assumes that the $dp', d\epsilon_s$ -coupling and the $dq, d\epsilon_v$ -coupling are controlled by the same J -modulus. Positive dilatancy, that is, expansion during shearing, is associated with $J < 0$. Compression during shearing produces $J > 0$. If there is no dilatancy or no anisotropy, $J = \infty$, and [2.7] has the same form as the K, G model described by Domaschuk and Valliapan (1975). Eqn. [2.7] can also be extended for a general 3-D stress state.

When the current stress p', q is lower than previous stresses $p'_{\max}, q_{\max}$ (that is, during unloading or reloading), the same form as [2.7] is used but the moduli K, G and J are replaced by K^e, G^e and J^e . Note that in this case $p'_{\max}$ and $q_{\max}$ are not the failure stresses p'_f, q_f , but rather the highest values attained during earlier stressing. Appropriate moduli (K, G and J , or K^e, G^e and J^e) are then chosen depending on whether the stress changes are for first-time loading or for unloading / reloading.

Dr. T. Huckel, Duke University, has commented in personal communications that "hypoelasticity has drawn series criticism in the seventies, for its lack of incremental continuity of response to loading along neutral paths (Morz et al. 1979). This defect leads to series numerical instabilities in finite element programs and hypoelasticity has since lost its impact. A remedy to this shortcoming may be sought through a discrete or continuous dependence of incremental matrix on the stress rate direction."

2.4.3 Elastic-Plastic Soil Models

The concept of yielding and the development of irrecoverable plastic strains has now been well established in modern soil mechanics. The incremental elastic-plastic theory applied to soils usually includes the following basic elements (Ko and Sture, 1981):

(1) The strain increment $\delta\epsilon_{ij}$ can be decomposed into the elastic and plastic parts, $\delta\epsilon_{ij}^e$ and $\delta\epsilon_{ij}^p$.

(2) The elastic strain increment can be calculated as $\delta\epsilon_{ij}^e = C_{ijkl} \delta\sigma_{ij}$, where C_{ijkl} is the elastic constitutive tensor.

(3) There exists a yield surface $f(\sigma_{ij}, H)$ representing the boundary of all states in stress space that are attainable from the current stress state elastically.

(4) When the stress state reaches the yield surface, the plastic strain increments are defined by a plastic potential $g(\sigma_{ij}, H^*) = 0$ such that

$$\delta\epsilon_{ij}^p = \lambda (\partial g / \partial \sigma_{ij}) \quad [2.8]$$

where λ is a scalar multiplier, whose determination will be based on the basic elements (1) to (4) and a consistency relation. The yield function f and the plastic potential g depend on the hardening parameters H and H^* , which reflect the plastic strain history. If such dependence is absent, then ideal plasticity is obtained.

(5) The consistency condition, $\delta f = 0$ ensures that the stress state does not leave the yield surface when stress states follow the expansion of the yield surface.

(6) Strain or work-hardening materials require a hardening rule.

The flow rule in [2.8] can be classified as either associated or nonassociated. In the case of an associated flow rule, with $f = g$, the plastic strain increments calculated from [2.8] are normal to the yield surface, and Drucker's postulation (Drucker, 1950) for a stable material applies.

Based on the pioneering work of Drucker et al. (1957), several hardening plasticity theories have been advanced to describe the consolidation and yielding behaviour of soils. The basic differences among these models lie in the description of the yield surface and the manner in which the hardening parameter H enters into the yield function. The isotropic hardening plasticity models, for instance, specify that the yield surfaces expand or contract isotropically about the hydrostatic axis in principal stress space. A well known example of isotropic hardening plasticity models is the Cam-Clay model proposed by Roscoe and Schofield (1963) based on the critical state concept.

Roscoe and Schofield derived the Cam-Clay model based on the assumption that there is no recoverable component in the shear distortion of wet clay. The original Cam-Clay constitutive equations overpredicted the observed values of the strain increments at small shear stress levels. This was presumably due to the original assumption regarding zero recoverable shear strain. Moreover, the original bullet shaped cap (yield locus) predicted shear strains in isotropic compression.

Later, Roscoe and Burland (1968) suggested a modified version of the Cam-Clay model with elliptical yield loci. This model took account of a recoverable component of shear strains accompanying any changes in deviatoric stress q . The elastic and plastic constitutive relationships in this case can be summarized as follows :

$$\begin{bmatrix} \delta v^e \\ \delta \epsilon^e \end{bmatrix} = \begin{bmatrix} K/Vp' & 0 \\ 0 & 1/3G' \end{bmatrix} \begin{bmatrix} \delta p' \\ \delta q \end{bmatrix} \quad [2.9]$$

where G' is the elastic shear modulus, λ is the slope of virgin compression line and K is the slope of the swelling (or recompression) line.

$$\begin{bmatrix} \delta v^p \\ \delta \epsilon^p \end{bmatrix} = (\lambda - K)/Vp' \begin{bmatrix} (M^2 - \eta)/(M^2 + \eta) & 2\eta/(M^2 + \eta^2) \\ (2\eta/(M^2 + \eta^2)) & 4\eta^2/(M^2 - \eta^2)(M^2 + \eta^2) \end{bmatrix} \begin{bmatrix} \delta p' \\ \delta q \end{bmatrix} \quad [2.10]$$

where $\eta = q/p'$. In Critical State theory the isotropic normal consolidation line (NCL) and Critical State Lines are assumed to be straight and parallel in $V - \ln(p')$ plot with the slope λ . The normal consolidation line is expressed as follows:

$$V = N - \lambda \ln(p') \quad [2.11]$$

The parameter N , the equilibrium specific volume of the soil corresponding to unit effective mean stress, is a constant for a particular soil. It is evident that a total of five parameters λ, K, G', M, N are therefore needed to form a Cam-Clay model.

2.4.4 Critical State Concept

Application of the theory of plasticity to soil mechanics led to the development of the critical state concept by Roscoe, Schofield and Wroth (1958) and Schofield and Wroth (1968). The state of a soil sample can generally be described by three parameters. These are mean principal effective stress p' , deviator stress q and specific volume V .

The critical state is an ultimate condition in which plastic shearing could continue indefinitely without changes in volume or effective stresses. This condition of perfect plasticity can be expressed by:

$$\partial p' / \partial \epsilon = \partial q / \partial \epsilon = \partial V / \partial \epsilon = 0 \quad [2.12]$$

The critical state is reached with an effective stress ratio

$$(q / p')_{c.s.} = \eta_{c.s.} = M \quad [2.13]$$

In drained or undrained triaxial compression tests on normally consolidated (or lightly overconsolidated) soil yielding first occurs with stress ratio $\eta < M$ as shown in Figure 2.23. Continued loading is associated with plastic hardening, expansion of yield loci, and increase of stress ratio until ultimately the effective stress state reaches the point on the current yield locus where the plastic strain increment vector is aligned with the critical state line ($q / p' = M$).

In drained and undrained triaxial compression tests on heavily overconsolidated soil yielding first occurs with $\eta > M$ as shown in Figure 2.24. Continued deformation is associated with plastic softening, contraction of yield loci, and decrease of stress ratio until ultimately the effective stress state reaches the point on the current yield locus where the plastic strain increment vector is again aligned with the critical state line ($q / p' = M$).

The data collected in the testing program for the buffer (Chapters 5 and 6) will be used later to form hyperelastic, hypoelastic and critical state models of the behaviour.

CHAPTER THREE

DEVELOPMENT OF COMPUTER-AIDED HIGH PRESSURE TRIAXIAL SYSTEM

The proposed experimental program for the geotechnical behaviour of buffer involved pressures up to 10 MPa. This range of pressures is about an order of magnitude higher than the pressures covered by conventional laboratory testing facilities. Therefore, a High Pressure Triaxial System (HPTS) was specially developed in the geotechnical laboratories at the University of Manitoba for testing buffer. The low hydraulic conductivity (10^{-12} m/sec, Cheung et al. 1985) of the buffer was seen to require long periods of time for the material to reach equilibrium. This warranted the need for an automated data acquisition system to facilitate long term triaxial testing. The details of Computer-Aided High Pressure Triaxial System (CAHPTS) including instrumentation and software development will be discussed in this chapter. The system is one of three such facilities in Canada, the others being at the University of Alberta and in CANMET in Ottawa.

3.1 Design of High Pressure Triaxial Cell

The high pressure triaxial cell with submersible load cell in Figure 3.1 was specially designed and manufactured in the University of Manitoba workshops. The cell has been used to test samples 50 mm in diameter and 100 mm in height at confining pressures up to 10 MPa and

back pressures up to 8 MPa. Other sample sizes up to 100 mm in diameter may also be tested by fitting an appropriate pedestal and loading cap. The cell was constructed of steel (hot rolled - 1020) and was later nickel plated to provide protection against corrosion. A unique feature of the cell design is that the tie rods are placed inside the cell to provide support for internal instrumentation and to speed up the cell assembly after the specimen has been installed. The cell consists of three separable parts :

(a) The mild steel cell base provides adequate embedment length for internal tie rods. Confining pressure and back pressure drainage ports access the interior of the triaxial cell through pressure sealed fittings in the base.

(b) The cylindrical sleeve is a 12.7 mm thick seamless mild steel pipe (hot rolled - 1020) capable of withstanding at least three times the allowable design pressure prior to yielding. The sleeve slides down into the base plate once the specimen installation is complete. The sleeve placement is facilitated by a jacking system specially designed to avoid jamming of the sleeve against the cell top.

(c) The cell top guides the loading ram with the submersible load cell attached to it. It is supported and secured by four rods and nuts which tie the cell to the base plate. Careful machining and assembly of base plate, cell top and tie rods ensures alignment of the loading ram with the cell axis throughout the testing program. To prevent possible uplift when pressurized, the cylindrical sleeve is restrained by four small plates attached to the cell top by Allen screws.

Hydraulic seals between parts are provided by a wiper seal and several 'O' rings. For instance, a wiper seal (25 mm in diameter) is

used to provide seal between the loading ram and its guide as shown in Figure 3.1. The loading ram guide is covered by teflon to reduce friction. Nevertheless, the axial load is measured inside the cell at the top of sample by submersible load cell fixed to the loading ram as shown in Figure 3.2. The ball bearing assembly in the figure ensures alignment of the axial load applied through the load cell button on to the loading cap at the top of the specimen. Details of the load cell are given in Section 3.3.3. The electrical wires from the load cell exit the cell top through a pressure sealed bulkhead connector. A clamping system was also designed for holding the load ram as shown in Figure 3.3.

3.2 Triaxial Testing System

Three different triaxial cells with various pressure capacities were used at different stages in the program. Conventional triaxial cells were used for testing up to 1.75 MPa cell pressure. For confining pressures less than 3.5 MPa, commercially produced "high" pressure Brainard-Kilman cells with aluminium sleeves were utilized as shown in Figure 3.4. The fabrication of the specially designed high pressure triaxial cell, described in Section 3.1, allowed testing up to 10 MPa confining pressure as shown in Figure 3.5.

The schematic layout of the computer-aided high pressure triaxial system is illustrated in Figure 3.6. Fluid pressures were supplied from compressed nitrogen tanks controlled by self-bleeding high capacity pressure regulators. The accumulator provided physical separation between the inert nitrogen gas and silicone oil used for the

cell fluid by means of a rolling bellofram pressure diaphragm. The accumulator kept the cell pressure constant by allowing compensation for the small cell fluid losses encountered in long term testing. A short length of small diameter, high pressure, fabric-covered hydraulic hose was used to connect the accumulator to the high pressure triaxial cell. The use of silicone oil with 500 CP viscosity as the cell fluid, and the physical separation of gas and fluid by the accumulator reduced the possibility of osmosis and air-diffusion through the specimen membrane, especially in long term triaxial tests (Pollard et al., 1977). De-aired water was used in the back pressure lines.

At the start of the experimental program, the instrumentation was partly mechanical and partly electrical. Axial deformations and volume changes in the specimens were measured by a displacement dial gauge and a graduated burette respectively. The axial load was measured by a mechanical load (proving) ring. The measurements were recorded manually at this stage. Later, exclusively electrical instrumentation was used and this allowed automatic data acquisition. Detailed description of the measurement devices and instrumentation will be discussed in Section 3.3. A Linear Variable Differential Transformer (LVDT) was used to measure axial deformation (channel 1) in Figure 3.6. A high capacity submersible load cell was used to measure axial load applied to the specimen (channel 2). An LVDT-based volume change device was utilized to record specimen volume changes (channel 3). Pressure transducers were used to measure porewater pressure at the cell base (channel 4) and cell pressure (channel 5). Figure 3.7 shows the high pressure triaxial cell with instrumentation attached.

3.3 Measurement Devices and Instrumentation

3.3.1 Fluid Pressure Transducer and Calibration Device

The wide range of fluid pressures involved in the testing program required the use of a number of different pressure transducers with desired capacities and accuracies. In the low and intermediate stress ranges up to 3.5 MPa, Data Instruments pressure transducers (Model AB) with three different capacities 0.7, 1.4, 3.5 MPa were used. The accuracy of the measurements including non-linearity, repeatability and hysteresis is better than 1% of full-scale range. In the high stress range up to 10 MPa, Micro Gauge pressure transducers were used with a capacity of 14 MPa. The accuracy of the measurements is within 0.1% of full-scale in this case. These transducers were used to measure cell pressures and porewater pressures during the tests. The transducers were mounted directly on the cell base to be as close to the pressurized environment as possible and to avoid compressibility associated with pipe connections.

The pressure transducers were calibrated using model 8130 Ashcroft portable dead weight tester (Lucas Industrial Management) which was calibrated to National Standard. The accuracy of the dead weight tester in Figure 3.8 is 0.03% of the reading with a maximum capacity of 7 MPa. A typical calibration curve is shown in Figure 3.9. Linear regression analysis of the calibration data indicated excellent linearity ($R^2 = 0.999999$) of transducers in the measurement range in general agreement with the manufacturer's specifications. All transducers were calibrated regularly before use to confirm the recorded calibration factor.

3.3.2 Axial Deformation Transducer

A Linear Variable Differential Transformer (LVDT) is a reliable and very sensitive instrument for measurement of axial deformation. The displacement of the LVDT core is linearly proportional to the variation of output voltage (Figure 3.10) and produces the calibration factor also shown in the Figure. The DC-type LVDT used has 25 mm total travel in the linear range. The accuracy of full scale measurements is 0.5% (0.12 mm) which means 0.12% strain accuracy full scale for a 100 mm long specimen.

3.3.3 Submersible Load Cell

In the low and intermediate stress ranges up to 3.5 MPa, the axial load was measured either by a load ring or an external load cell at the top of the loading ram. In the high pressure testing system (HPTS), on the other hand, the axial load applied on a specimen was measured directly using an internal submersible load cell (Interface 1221) attached to the loading ram as shown in Figure 3.2. The internal load cell eliminated the necessity to provide a frictionless bearing for the loading ram. The capacity of the Interface load cell is over 110 KN operating under 10 MPa cell pressure without appreciable zero shift due to the variation of cell pressure.

A typical calibration curve including a load-unload cycle for the load cell under the maximum confining pressure of 10 MPa is illustrated in Figure 3.11. The calibration was repeated with zero cell pressure for comparison. No appreciable difference was observed due to the presence of confining pressure. "No zero shift" performance of the

load cell was deemed an essential requirement for accurate measurement of the axial load. The combined maximum error due non-linearity, hysteresis, non-repeatability and static error band is less than 0.13% full scale. The maximum error would then be 0.07 MPa in axial stress accuracy full scale for a 50 mm diameter specimen. In order to improve the sensitivity of measurements the load cell input supply was doubled from 10V to 20V. Figure 3.2 shows the load cell attached to a very carefully machined mounting plate, ground flat to 0.0002 T.I.R. with a loading button acting against a ball bearing resting at the top of the specimen. The loading ram - load cell - ball bearing assembly was especially designed to ensure coincidence of the loading axis and the axis of symmetry of the specimen throughout the testing program.

3.3.4 Volume Change Device

The burette system used in conventional tests was replaced by an Imperial College volume change transducer from Shape Instruments. An schematic diagram of the device is shown in Figure 3.12. It consists of a hollow brass cylinder and a floating frictionless piston with two Rolling Bellofram seals attached. Water flowing into or out of the chamber causes the piston to move, and this movement is measured externally by means of an LVDT mounted on the outside of the cylinder. The capacity of the device is 100 ml with a 25 mm travel LVDT. The calibration factor for the device was found to be insensitive to changes in back pressure from 0.2 MPa to 1.0 MPa. A typical calibration curve is shown in Figure 3.13. According to the manufacture's specifications, the device is designed to withstand a maximum back pressure of 1.4 MPa and the sensitivity of the measurements is 0.02 ml.

For triaxial tests with elevated back pressure up to 8 MPa, a thick wall cast acrylic back pressure burette has been designed and manufactured. To allow automatic data acquisition a high pressure volume change device with a capacity of 10 MPa is now being designed for future work at elevated temperatures up to 100°C.

3.4 Loading Systems

Two different loading systems were used for strain controlled and stress controlled tests in the program. The strain controlled shear tests were run by moving the platen in a 10 KN or 50 KN compression frame at a constant pre-set strain rate. The generated axial load was measured by either load ring or electrical load cell. Stress controlled tests were performed incrementally by applying known dead weights on a hanger. A lever arm system was utilized to magnify the applied load by a factor of 7.9858 where large dead weights were required. This system was utilized for incremental drained tests with cell pressures up to 3.5 MPa.

3.5 Data Acquisition System (DAS)

A Data Acquisition System (DAS) has been built and developed in the University of Manitoba as part of this program to monitor triaxial tests and collect data. After discussion, it was decided to develop small, stand-alone units dedicated to a small number of tests and individual graduate students, rather than a larger centralized unit.

This was done by attaching a channel selector and a digital voltmeter to a microcomputer which was then also available for data reduction, word processing and graph plotting.

Instrumentation readings are taken at selected time intervals and recorded systematically. The response of a test specimen can also be monitored on the computer video screen at any time while the test is in progress. The details of interfacing devices, software for data acquisition, reduction and plotting have been presented separately in a DAS User's Manual (Nordien and Saadat 1988). An overview of the system will be discussed here briefly.

3.5.1 Interfacing Devices

A schematic diagram of various interfacing devices in the Data Acquisition System is illustrated in Figure 3.14. The system consists of an IBM-PC (or compatible computer) connected to a HP-PC Instrument Digital Multi-Meter (DMM), its accompanying IBM-PC adaptor card and three different power supplies (5V, 10V and 20V). Analog (output) signals from the measuring instruments are converted to digital signals by the Digital Multi-Meter, then sent to the IBM-PC. The DMM has 8 I/O channels whose ranges are set at 200 mV or 20 V depending on the output voltage range of the measuring instrument, as given in Table 3.1. One or two HP-PC Instruments Relay Multiplexers (with 8 output channels each) are used as channel selectors. The HP-PC Instruments software for controlling DMM and relay multiplexers has been incorporated into a program for triaxial testing. The original version was written by Indrawan (1986). Additional software were also developed at the University of Manitoba for data reduction and plotting using LOTUS

123.

3.5.2 Software Development for Data Acquisition and Monitoring

An interactive computer program in BASIC language written by Indrawan (1986) for IBM-PC / TecMar LabMaster Computerized Triaxial System at the University of Auckland has been modified to communicate with the PC / HP-PC Instruments DAS developed at the University of Manitoba. The HP-PC Instruments routine for controlling DMM and relay multiplexer replaced the routine for TecMar LabMaster. Some other modifications were necessary such as (1) the introduction of an extra channel for each test to monitor and record the confining pressure which had to be adjusted manually during the test; (2) accomodation of long term consolidation and the capability to change the reading intervals during shear; (3) improvement of video display management; (4) minor changes to the disk files used by the program; and (5) monitoring capability on the screen at anytime during the test. A subroutine CALIB was also written to facilitate the calibration of measurement devices. The data acquisition program is called TestMast for Test Master. TestMast now requires 5 channels to run a complete triaxial test. A 16 channel data acquisition system with the modified version of the software can run three tests simultaneously as shown in Figure 3.14. The capacity of the system can be easily expanded to handle more tests if desired.

TestMast was designed (Indrawan, 1986) to distinguish various stages of a triaxial test. For instance, TestMast can handle simulataneously, test #1 at the saturation stage, test #2 at the consolidation stage and test #3 at the loading stage. TestMast can

take measurements at any given interval for any test. This capability allows each test to be controlled independently. TestMast reads data in volts through the DMM and writes the raw data to a disk file. The organization of Testmast and step-by-step procedures for running TestMast are discussed in detail in the available User's Manual (Nordien and Saadat 1988).

3.5.3 Computer Programs for Data Reduction and Plotting

To convert the recorded voltage readings into engineering units two analysis programs called "Cons" (for Consolidation stage) and "Triaxial" (mainly for undrained shear) are used. These are modified versions of the analysis programs included in Testmast by Indrawan (1986). These programs are special cases of Undrained and Drained Triaxial Consolidation Shear Data Analysis Programs for PC developed at the University of Manitoba by Homenick and Saadat (1987). These interactive programs were developed earlier to handle the data which were collected manually.

"Cons" and "Triaxial" produce two diskfiles each. The first diskfile records the converted data in engineering units, and the second diskfile contains the stresses, strains and other results of the analysis. The plotting capability of earlier version of "Triaxial" has been replaced by a subroutine written to create a Lotus 123 import file. A standard worksheet template was created to produce the curves needed for interpreting the collected data, in addition to a menu designed to facilitate loading new data, viewing the test curves, and saving the curves. The structure of DAS worksheet menu is given in the User's Manual along with the detailed explanation of plotting

procedures. Lotus 123 is used to produce publication quality plots of the test results on a pen plotter.

3.6 Contributions to Development of Geotechnical Laboratories

At the beginning of this project the capability of the triaxial testing laboratories at the University of Manitoba was limited to 0.6 MPa pressure, with manual data collection, simple data reduction, and no plotting facilities. As a result of the work described in this chapter, the Geotechnical Laboratories has now developed a capability for performing Computer-Aided High Pressure Triaxial tests to 10 MPa, with back pressures up to 8 MPa. Data can now be collected automatically by a dedicated Data Acquisition System and analysed by Data Reduction Computer Programs. Data manipulation, regression and quality plotting can also be performed on a personal computer work station.

CHAPTER FOUR

EXPERIMENTAL PROGRAM AND TESTING PROCEDURES

This chapter begins with a review of the basic properties of the buffer and its components. The specimen preparation techniques for triaxial testing and quality control tests are then discussed. Finally, the design of the experimental program and testing procedures are described in detail.

4.1 Test Materials

Testing has been done on the 50% - 50% mixture by dry mass of crushed quartz sand and sodium-based bentonite described in Section 1.2.3 as the Reference Buffer Material (RBM) defined by AECL. In subsequent writing it will often be referred to simply as "buffer". The sand fraction of the buffer is a uniformly graded mixture of sub-rounded fine to medium sand particles obtained from crushed single-mineral fine quartz. Detailed specification of the sand has been given by Gray et al. (1984) but can be summarized here as $D_{10} = 0.2 \text{ mm}$, $D_{85} = 1.3 \text{ mm}$, Curvature Coefficient = 0.91. The bentonite is a sodium-rich bentonite sold commercially as Avonseal by Avonlea Mineral Industries at Regina. The source of the natural material is the Bearpaw Formation of Upper Cretaceous Age in South Saskatchewan.

Avonseal is a grey-white powdered clay produced by controlled drying (less than 120° C) and grinding of the natural soil (Dixon and Gray, 1985). Its composition has been reported previously by Dixon and Woodcock (1986), and by Quigley (1984). The liquid limit is 250% and the plasticity index 200%. Distilled water has been used throughout the testing program. Since these are tests on artificially prepared materials it is more appropriate to use the term "specimen" for the cylinders of compacted buffer that were tested. Occasionally "sample" may be used synonymously with "specimen" (Graham et al. 1986).

4.2 Specimen Preparation and Quality Control

The techniques for specimen preparation are consistent with the procedures used earlier by Sun (1986), Wan (1987) and Yarechewski (1988) for specimens compacted to 85% ASTM Modified Standard Density. The improved compaction frame (Figure 4.1) commissioned by Yarechewski (1988) was employed to produce higher density specimens, mainly at 95% ASTM Modified Standard Density.

Before experimental studies could start, it was necessary to develop specimen forming techniques which would allow preparation of consistent specimens. The sand and bentonite were first dried in an oven for 24 hours at 105°C. After cooling to room temperature, the weights of distilled water and sand needed to make a specimen were mixed together. The required quantity of dry bentonite was then stirred carefully into the mixture until the sand grains were bounded uniformly by clay particles. The mixture of sand, bentonite and water was then cured in an airtight container at 6°C for three days. Before

compacting the specimen, the cured mixture was vacuum deaired for 5 minutes at room temperature to reduce the quantity of air dissolved in the water phase. The resulting mixture was then compressed in five equal layers into a standard-sized cylindrical form to produce specimens approximately 50 mm in diameter and 100 mm high. Following this, the specimens were extruded, weighed and measured to determine their bulk unit weights. Surplus material was used for water content determination before and after compaction.

Table 4.1 shows the relationship between "targetted" and "measured" values of dry density and water content. In Table 4.1(a), the aim was to produce specimens that were compatible with the earlier specimens tested by Sun (1986) with average dry density $1.50 \pm 0.03 \text{ Mg/m}^3$ at water content $27.9 \pm 0.5\%$. The purpose of this work was to continue the investigation begun by Sun to higher stresses. The compaction density corresponded to 85% ASTM Modified density. The quality control tests QC901-QC908 in Table 4.1(a) suggested a practice of targetting the specimen preparation towards a design dry density of 1.54 Mg/m^3 at 29.5% water content. The quality control specimens QC901-910 in Table 4.1 were split into 5 slices to determine their variation in water content along the length of the specimen. Each slice was divided into two halves to check the ability to measure water contents consistently. The average water contents and standard deviations of the variations of water contents along the length of the specimens are given in Table 4.1. Typical results are shown graphically in Figure 4.2.

The resulting average values of density and water content actually obtained in each of the tested specimens in Series (I), T901-T913,

T915-T919, T922-T923 and T925 compacted to 85% ASTM Modified density are shown in Table 4.2, together with some other specimens compacted to higher densities. The overall average dry density and water content that were achieved in the actual test specimens are $1.50 \pm 0.02 \text{ Mg/m}^3$ at $28.4 \pm 0.9\%$ water content. These agree closely with the earlier results by Sun (1985).

Similar quality control tests were later performed by Wan (1987) for specimens compacted at a range of initial densities and water contents. The data show that these specimens are also uniform and consistent. Therefore, the same procedure for specimen preparation can be utilized to form specimens at other densities of interest in this program. To confirm this conclusion two more quality control tests were performed and the results are given in Table 4.1(b). More recently, Yarechewski (1988) has improved the specimen formation techniques further using a variable lift thickness during compaction. As a result of this work, specimens can now be formed in five layers with variations in moisture content and dry density of $\pm 0.11\%$ and $\pm 0.003 \text{ Mg/m}^3$, respectively.

4.3 Testing Program

The objectives and scope of the experimental investigation were discussed in detail in Sections 1.3 and 1.4. It will be recalled that the main purpose was to examine the mechanical behaviour of compacted buffer at different densities and pressures at room temperature. The testing program was especially designed to complement the results from earlier investigations by Graham et al. (1985, 1986a) mostly on buffer

compacted to 85% ASTM Modified Standard Density (1.50 Mg/m^3 dry density) in a low range of stresses up to about 1 MPa.

The program is divided into four main series of tests. In Series (I), specimens were compacted to 1.5 Mg/m^3 dry density corresponding to 85% ASTM Modified Density. This series is an extension of the work by Sun (1986) on buffer compacted at the same density but now to higher stresses up to 10 MPa. Twenty-two specimens (T901-T919, T922-T923 and T925) were tested in this series. Series (II) involved buffer material compacted to various densities related to the test confining pressure. The primary objectives of this series were to :

- (1) shorten the duration of the initial consolidation phase of the experiment, where it appeared that many weeks were necessary to allow the specimens to compress or swell to equilibrium water contents under the applied cell pressures; and
- (2) confirm the relationship proposed by Graham et al. (1986) between swelling pressure and clay dry density (or clay specific volume) in a higher stress range and compare the results with earlier values from one-dimensional compression tests by Dixon et al. (1986).

This series consisted of four specimens (T920-T921, T924 and T926) compacted at equilibrium moisture content on the basis of progressively updated equilibrium mean pressure vs. clay specific volume relationship. It complements the work reported in the MSc. thesis by Wan (1987).

Series (III) simulates the initial buffer density that is actually proposed for the operation of the repository. The Reference Buffer Material is to be compacted at 95% ASTM Modified Density (1.67 Mg/m^3 dry density). This series included seven specimens T927-T934 tested at

pressures up to 10 MPa. Series (IV) was designed to investigate the applicability of the effective stress principle to the buffer. This series comprised three specimens (T927 and T932-T933) compacted at various densities and tested with different back pressure up to 7 MPa. The detailed schedule of testing program is summarized in Table 4.3 and given in detail in Appendix (A).

In a general sense, the testing program consists of four types of tests performed at different stress levels - (1) isotropic consolidation; (2) undrained triaxial compression with porewater pressure measurements; (3) drained incremental shear loading at constant p' ; (4) conventional drained loading at constant strain rate. In an idealized isotropic condition, the deformation of the buffer along any stress path may be decoupled into "volumetric" and "shearing" deformations. The isotropic consolidation tests provide the necessary information to quantify the volumetric component of deformation, while the constant- p' drained tests evaluate the shearing component separately. The conventional undrained and drained tests are performed to assess the strength and porewater pressure behaviour of the buffer. The principal soil parameters to be examined in the testing program are; (a) the effective stress strength envelope; (b) porewater pressure parameters A_f and m ; (c) undrained Young's modulus E_{50} from the end of consolidation to 50% of the undrained shear strength; and (d) bulk modulus K and the shear modulus G . The testing procedures are discussed in the following sections.

4.4 Testing Procedures

Test procedures adopted in this experimental program basically

follow those developed earlier in the geotechnical laboratories of the University of Manitoba (for instance, see Noonan, 1980). However, some modifications have been found necessary due to the nature of the buffer material being tested. These are discussed in detail in following sections.

4.4.1 Isotropic Consolidation Tests

4.4.1.1 Installation Procedures

After being compacted to the required unit weight and water content, the specimens were extruded from the mould and installed on a deaired triaxial cell base. The triaxial test arrangement generally included:-

- (1) Lateral filter paper drains, now installed in a spiral arrangement to reduce their contribution to sample stiffness
- (2) Bottom end drainage only - this represents a change from earlier testing arrangement used by Sun (1986), where top drains were also used. The simplification aimed at reducing recurring problems with leakage of cell water into the drainage lead connections. The differences in drainage time between single-ended and double-ended drainage is small provided lateral drains are used (Bishop and Henkel, 1964).
- (3) Two latex membranes separated by a layer of silicone oil, with the cell pedestal and top cap carefully covered with silicone grease. The thickness of the membranes was increased as the confining pressures increased so that the diffusion rate through the membranes was minimized in long term tests. Membranes of three different thicknesses

of 0.36 mm, 0.64 mm and 1.27 mm were used in this program.

- (4) The test cell filled with either; (a) deaired water beneath a covering layer of viscous mineral oil used to reduce piston friction and leakage in Wykeham-Farrance, and Brainard-Killman cells; or (b) silicone oil with a viscosity of 500 CP in the high pressure triaxial cell manufactured for this project.

Installing specimens in triaxial cells is commonly accompanied by strenuous efforts to minimize air trapped in the cell base or between the specimen and the inner membrane. This is usually done by flushing water from the bottom to the top of the installation, and washing air bubbles past the top cap. However, early in the testing program it was appreciated that bentonite close to the outside of the specimens could swell rapidly at the low (atmospheric) pressure during this process. This would then be accompanied by subsequent re-compression during the early stages of consolidation and would introduce uncertainty into calculations of water content, clay effective density, and V_c . In addition, the procedure was not totally effective in removing air from around the specimens or from inside the specimens, although the average degree of saturation at the beginning of testing was $96.7 \pm 1.4\%$ (Graham et al. 1986). Air bubbles were observed almost daily in the drainage leads and had to be removed by the flushing procedures described for example by (Lew, 1981). Once again however, this caused some uncertainties in V_c -calculations and porewater pressure measurements.

Both of these difficulties have been largely overcome in subsequent tests by using a "back pressure" of 200 kPa (up to 1 MPa at 10 MPa cell pressure) in the porefluid (Head, 1980). In this

procedure, the cell pressure and porewater pressure are both increased by 200 kPa so that the effective consolidation pressure on the sample at equilibrium remains at the desired value. The procedure clearly requires the validity of the effective stress principle outlined in Section 2.2.3. Experience with later specimens in the program in terms of water content control has been fairly good. Air bubbles have not been a major problem; volume changes have been easier to measure and record and agreements between the calculated and measured water contents have been better as will be discussed in Section 4.5. Furthermore, the level of saturation in the specimens at the end of consolidation has been generally satisfactory as will be shown in Section 4.4.1.2.

4.4.1.2 Saturation Tests

The saturation of the specimens is checked by performing B-tests. This is done by closing off both drainage leads to the bottom of the specimens, increasing the externally applied cell pressure σ_{cell} say by 50 kPa when the effective cell pressure is 1 MPa (or 500 kPa at 10 MPa cell pressure) and measuring the resulting value of $B = \delta u / \delta \sigma_{cell}$ in the porewater pressure inside the specimen. For fully saturated soil, the measured B-values depend on the relative stiffnesses of the soil skeleton and the porewater and can be expected to be lower than 100% at high confining stress levels (Bishop et al. 1975). Assuming that the solid particles are incompressible, the relationship between the coefficient B, the compressibility of the soil skeleton C_s and the compressibility of water C_w can be expressed as

$$B_{100} = \frac{1}{1 + n C_w / C_s} \quad [3.1]$$

where n is the porosity of the soil (Bishop and Eldin 1950). Based on the measured long term compressibility of the compacted buffer (Section 6.2), B -values of 0.99 and 0.97 should be obtained from fully saturated specimens when the consolidation pressures are 3 MPa and 9 MPa respectively. In contrast, however, extensive experimental and theoretical studies by Black and Lee (1973) suggested that the B -value could drop to 0.69 and 0.20 for stiff and very stiff soils respectively at saturation levels as high as 99.5% . In addition, the time required to reach a full 100% saturation may be longer than desirable or acceptable especially if high back pressures are not available. Direct measurements of porewater pressures in B -tests produced the values given in Table 4.4 within twenty minutes to one hour after increasing the confining pressure. Except for a few samples where air bubbles appeared to be the problem, the obtained values are generally higher than 0.6, a value that Black and Lee suggest would indicate saturation levels of better than 99.5% . The long-term B -values, however, were generally found to be very close to unity. In addition, the behaviour of specimens in undrained shear (Section 5.3.2.3) indicated that this level of saturation may be considered acceptable provided that the specimens were sheared very slowly to allow enough time for porewater pressure equalization (Head, 1980). Therefore, alternative criteria for determining saturation were adapted in this program ; that is (1) to measure B -values at two back pressures and compare to B_{100} , and (2) to increase the back pressure carefully noting any tendency for water intake.

Figure 4.3 shows the correlation between measured B-values and the initial degree of saturation for specimens with back pressures in excess of 400 kPa. The lower the initial degree of saturation the lower the measured B-value. In the case of specimens with air problems and low initial degrees of saturation, it sometimes took a few hours before the B-value approached acceptable values. This was considered as an indication of lack of full saturation. To overcome this problem the magnitude of back pressure was increased where the capacities of the cell and the transducers allowed. The higher back pressure was maintained for a few days to dissolve small air bubbles in the system before the B-test was repeated. Experimental studies by Black and Lee (1973) showed that small air bubbles (diameter < 2 mm) suspended in a small diameter plastic tube under pressures in the range of 150 kPa to 550 kPa require up to several days to dissolve. Later in the program, high air entry disks were also utilized at the base of the specimen to prevent air entry into the filter stone and the pressure transducer at the cell base.

4.4.1.3 Leakage Tests

Figures 4.4 to 4.7 show results from tests performed to examine the performance of the membranes surrounding the specimens. The membranes were applied over a dummy brass specimen of the same size as the normal specimens of buffer. All-around filter paper drains were used in each case to transfer either water volumes or water pressures transmitted through the membranes or the sealing rings. Figure 4.4(a) and Figure 4.4(b) show results from two 0.36 mm thick membranes at 1.5 MPa and 3.3 MPa cell pressure respectively with back pressures of 0.3

MPa in each case. Total volume changes and especially the measured volume change rates measured were relatively small. In Figure 4.4(b), although the total volume change was about 0.9% of total specimen volume but the steady state leakage rate dropped off rapidly to about 0.02% /day. Similar results were obtained by Wan (1987) in the low stress range. Figure 4.5 shows the results from thicker 0.64 mm membranes used at the same pressures. Similar membrane leakage tests were also performed using thicker 1.27 mm membranes (Figure 4.6) at high confining pressures up to 10 MPa and back pressures from 1 MPa to 7 MPa. The rate of volume change in the latter cases was approximately 0.005% /day, substantially lower than the former case using thinner membranes.

In Figure 4.7 the test was performed at 3.3 MPa cell pressure and 0.3 MPa back pressure with sealed-off drainage, and the gain in porewater pressure was recorded with time. The porewater pressure build-up due to diffusion or leakage past the membrane was estimated about 20 kPa /day using two 0.64 mm thick membranes. This is small compared to the applied cell pressure.

4.4.1.4 Consolidation Procedure

Consolidation tests were undertaken to provide the necessary information on the volume compressibility of the buffer through its bulk modulus, $K = \delta p' / \delta v$, where $\delta p'$ is the change in consolidation pressure and δv is the resultant volume strain. Consolidation tests with swell-compression cycles were also performed to evaluate the recoverable or elastic bulk compressibility of the buffer, $K^e = \delta p' / \delta v^e$, where δv^e is the elastic volume strain.

Two types of consolidation procedures were used in this program; single-stage in which the final consolidation pressure p'_{cons} was added in one step; and incremental consolidation in which a series of steps were added in turn until the final consolidation pressure was reached. The majority of the specimens were subjected to single-stage consolidation (Table 4.5) where the volumetric strain v was recorded with time until equilibrium was reached. The seven remaining specimens were loaded with a series of increments each lasting 5 to 7 days. The load ratio between successive increments was 1.32. The consolidation stage of the test was considered complete when the rate of volume strain was less than 0.1% /day as will be explained in Section 5.3.1.1. The degree of consolidation at the "end of consolidation" was then evaluated using the method proposed by Sridharan and Rao (1973).

4.4.2 Undrained Shear Tests

As indicated in Table 4.6 the majority of tests in the experimental program has been done in strain controlled undrained shear. After the B-test described earlier, the motor drive of the compression test frame was switched on, and the machine run at constant speed with nominal axial displacement rates corresponding to 0.1 - 1.2% of the initial sample height per hour. The choice of the strain rate influences the strength of the soil and the measured porewater pressures measured in undrained tests (Bishop and Henkel, 1964; Graham et al., 1983). Because of the low hydraulic conductivity of the compacted buffer very slow shearing rates have been used to ensure porewater equalization within specimens. The selected strain rate has been progressively reduced from 1.2% per hour to 0.25% per hour and

0.1% per hour as increased consolidation pressures up to 10 MPa produced denser buffer at the end of consolidation. This prolonged the shearing durations significantly so that at the end, undrained shear tests lasted up to one week. The theoretical time required to failure in undrained tests, based on 95% pore pressure equilization within the buffer sample (Bligh 1964), is estimated about three to four weeks. Considering the period of three to eight weeks required for consolidation stage of the test, a slightly lower degree of pore pressure equilization (over 90%) was here accepted as a compromise for practical reasons. Section 5.3.2.4 will show that the associated error in strength measurement will be less than 4%.

Readings of axial displacement, axial load, cell pressure and porewater pressure were taken at 5-20 minute intervals during the first hour, and thereafter at 30 minute to 2-4 hours intervals. Maximum deviator stress (shearing resistance) was reached after about 5-50 hours depending on the strain rate employed. Shearing continued to axial strains of about 11%-15% reached after 15-150 hours. Porewater pressures were measured by electrical pressure transducers housed in stiff mounting blocks attached directly to the base of the cells as discussed in Section 3.3.1.

The effect of strain rate on undrained shear strength of the buffer was also examined in this program. Relaxation tests and step changing strain rate tests with three different rates (Graham et al., 1983) were performed after the peak strength was reached. The relaxation test was conducted by switching off the compression machine and allowing the specimen to strain at a decreasing rate due to the stored energy in the proving ring. The step changing strain rate tests

involved changing gear or the rate of feed of the compression machine during shear. To examine the reversibility of shear deformations unload-reload cycles were incorporated into some tests after the peak resistance had been reached.

4.4.3 Drained Shear Tests

In addition to the strain controlled undrained shear tests, seven specimens listed in Table 4.7 were loaded with a series of deviator stress increments keeping the applied mean stress constant after porewater dissipation. This was done by decreasing the cell pressure by an amount equal to half of the axial stress increase, that is, $\delta p' = \delta \sigma_1' + 2\delta \sigma_3' = 0$. The drainage leads were kept open throughout. Each load increment was held for 7 days and the resulting axial and volumetric deformations measured. With increasing shear (deviator) stress levels, the axial displacement rates increased, until finally the specimens failed by exhibiting large, accelerating displacements. Another specimen T934 was subjected to strain controlled conventional drained shear where the cell pressure was kept constant throughout the test and the resulting volume changes was measured.

4.5 Water Content Control

The determination of the specific volume at various stages of the triaxial test is essential in the formulation of the conceptual model for the behaviour of buffer. At the end of shearing, the drainage leads to the specimen were closed and the cell pressure removed. The cell was then quickly emptied of water and oil, the sealing rings

removed, and the membranes rolled away from the specimen. Several samples were then taken from inside the specimen for determination of the end-of-test water content. By considering the volume changes measured during consolidation and drained shear, the initial water content can then be back calculated from this measured end-of-test water content. Similarly, the end-of-test water content can be estimated from the measured initial water content. Discrepancies between the measured and calculated values usually reflect deficiencies in water content control due to installation procedures and testing techniques. Occasionally, large discrepancies (say in excess of 4%) indicate leakage problems during testing. Small differences in water content control, in spite of well established testing techniques in carefully run tests, are accepted as inevitable.

The measured end-of-test water contents for all the specimens tested in this program are compared with the calculated values in Table 4.8. The agreement between these values is generally satisfactory, except for the first few specimens with leakage problem. For the majority of the specimens the difference between the measured and calculated water content values were less than 0.6%. However, a few specimens indicated discrepancies of about 2% due to (1) the flushing procedure used initially in the program which was later abandoned as explained in Section 4.4.1.1; and (2) lack of initial saturation of the lower filter stone upon sample installation because of earlier limitation of sleeve height in the high pressure cell. In such cases, the measured end-of-test water content was considered more reliable and was therefore used in the calculation of the clay specific volume V_c and density.

CHAPTER FIVE

TEST RESULTS

5.1 Introduction

This chapter presents results obtained from the experimental program conducted by the author. The first section highlights the findings from the quality control tests described in Section 4.1. This is followed by the presentation of the results from the triaxial testing program related to (1) consolidation, (2) undrained shear, (3) porewater pressure generation, (4) undrained stiffness, (5) drained shear, and (6) strength envelope. The volume of the collected data is too large to be presented here in its entirety. Reduced data for all the tests have been thoroughly documented in three annual technical reports to Atomic Energy of Canada Limited. The first two were submitted by Graham et al. (1986), and Graham and Saadat (1987). The final report by Graham et al. (1989) will be submitted shortly. Thus, it has been considered appropriate to concentrate here on typical test results selected to highlight contrasting patterns of observed behaviour in the experimental program. To simplify presentation of the results, the tests are divided into two groups of "low density" and "high density" specimens. Specimens compacted at densities less than 90% ASTM modified density (1.56 Mg/m^3 dry density) will be referred to as "low density" specimens. These specimens were definitely compacted on the "wet side" of their optimum (peak) moisture content (Dixon et al. 1985). Specimens compacted at densities more than 90% ASTM

modified density ("high density" specimens) are compacted close to their optimum moisture content, but generally just on the wet side so that high degrees of saturation were achieved. It will be shown in this Chapter in more detail and particularly in Section 6.2.3 that division into "low density" and "high density" specimens is a rational grouping for the tests based on compacted microstructural fabric. The importance of the microstructural fabric of clay on the behaviour is well established (Young and Warkentin 1975; Seed et al. 1962).

5.2 Quality control Tests

Before beginning the triaxial testing program, ten quality control tests were performed (see Section 4.2) to ensure that specimens formed for the testing program were uniform and consistent. Quality control tests were performed at both 85% and 95% modified ASTM standard density. Small variations of water content ($\pm 0.3\%$) along the length of the specimens (see for example Figure 4.2) demonstrated that the compacted specimens were essentially uniform. Careful control of compaction water contents produced high levels of saturation $96.7\% \pm 1.4\%$ in compacted test specimens as indicated in Table 4.2. In general, the results in Section 4.2 confirmed the repeatability and consistency of the specimen preparation technique for the testing program.

5.3 Triaxial Tests

A total of thirty-five compacted specimens has been tested under

triaxial (axisymmetric) stress conditions. Following the testing procedures outlined in Chapter 4, the specimens were first consolidated isotropically. After consolidation was completed, the majority (twenty-seven) of the specimens were sheared in undrained triaxial compression. The remaining eight specimens were subjected to drained compression at constant mean pressure. Figures 5.1-5.32 show typical test results presented here for completeness. The figures are arranged as follows:-

Figures 5.1-5.9; 5.13-5.14 - volume strain vs. time

Figures 5.10-5.12 - p'/p'_{cons} vs. v , incremental p'

Figures 5.15-5.21 - $q, \Delta u, q/p'$ vs. ϵ_1 , undrained CIU shear

Figures 5.22-5.25 - Δu vs. p , undrained CIU shear

Figures 5.34-5.35 - $\epsilon_1, \dot{\epsilon}_1$ vs. time, incremental q , constant p'

Figures 5.36-5.37 - q/q_{max} vs. ϵ , incremental q , constant p'

The strength envelope of the buffer is shown in Figures 5.33 and 5.34. Summarized results of parameters measured in the tests are presented in Tables 4.5 - 4.7 and 5.1 - 5.3 .

5.3.1 Consolidation

The consolidation results are presented here for (1) single-stage consolidation tests (2) incremental consolidation tests and (3) multi-stage consolidation tests under elevated back pressure.

5.3.1.1 Single-stage Consolidation

Typical results from single-stage consolidation tests are shown in

Figures 5.1 - 5.3 and 5.4 - 5.5 for compressive and expansive specimens, respectively. The results are given in terms of the volumetric strains measured from the absorbed (or expelled) water versus the elapsed time during consolidation. That is, the volumetric strains are measured over the whole sample and not exclusively in the clay-water phase.

Determination of swelling pressure of compacted buffer with different initial densities (Gray et al. 1984) is considered an essential step towards understanding the volume change behaviour. For example, under the applied consolidation pressure of 0.2 MPa high density specimen T926 in Figure 5.5 expanded. In contrast, high density specimen T920 in Figure 5.3 compressed when subjected to 3 MPa confining pressure. The confining pressure under which no volume change takes place corresponds to the swelling pressure. In general, the compacted buffer expands when the consolidation pressure is less than the swelling pressure and it starts to compress once this pressure is exceeded.

Consolidation durations in Table 4.5 ranged from 51 days in T916 (Figure 5.2) down to 9 days in T914 in single-stage consolidation tests. According to Sections 2.4.1 the "state" of the buffer is only determinate if (1) the applied total stresses and porewater pressures are known, (2) the specimen is in equilibrium under these stresses, and (3) the specific volume can be established from the initial compaction conditions (or the final water content) plus the measured volume changes. Therefore, the measured rates as well as the magnitudes of volume change during consolidation are examined here.

"Equilibrium" requires formally, that volume changes should have

ceased. However, it will be seen at once in Figures 5.1-5.2 and Figure 5.3 that some of the samples had not actually reached this condition when the consolidation phase was terminated. At this stage the average rate of volume change was usually less than 0.1% per day, a figure which corresponds to about ten times the membrane leakage rates established in Figures 4.4 - 4.6. Recent studies by Graham et al. (1986a) showed that this was a necessary but non-rational compromise between conflicting requirements (1) to wait until the measured volume change rate was zero; (2) to avoid problems of leakage or osmosis past the membranes and sealing rings in very long duration tests; and (3) to allow sufficient testing to ensure adequate insight into the broad nature of the behaviour of the buffer. The measured total volumetric strains in this program during single-stage consolidation and incremental consolidation tests are summarized in Table 4.5. They range from about -15.9% in T926 (Figure 5.5) to +13.2% in T925 (Figure 5.7). In general, the compressive volume strains increase with consolidation pressure, conversely the expansive volume strains decreased with consolidation pressure. The transition pressure between expansive and compressive is the swelling pressure which depends on the density of the compacted buffer.

Typical consolidation results for Series (II) are shown in Figures 5.3 and 5.4 and are summarized in Table 4.5. It will be recalled that the purpose of this series was to prepare the specimens at their equilibrium water contents and effective clay densities so that the times taken for them to reach equilibrium under the applied consolidation pressures would be small. This was in fact achieved. When compared with the consolidation strains in the Series (I) in Table

4.5, the Series (II) do show significantly smaller volume strains, generally less than 3% . The volumetric strains in the case of specimens T924 and T926 were only -0.24% and -0.15% respectively.

5.3.1.2 Incremental consolidation tests

Seven specimens in this experimental program T905, T912, T915, T917, T925, T929 and T930 were tested with incremental isotropic loading through successive increases in confining pressure. One specimen T929 was also subjected to an incremental unload-reload cycle to examine the elastic component of deformation under isotropic compression. Typical volume strain versus elapsed time responses are shown in Figures 5.6-5.9 . Each of the pressure increments was left in place for periods of 5 to 7 days. While these are shorter than the durations of consolidation in the other specimens where the final consolidation pressures were applied in a single increment, the volume strain rates at the end of the majority of the smaller increments were of the order 0.1% to 0.2% per day. This is just slightly larger than in single-stage consolidation tests in this program. To determine the degree of consolidation achieved at the end of each loading increment the strain-time curves have been examined using the method proposed by Sridharan and Rao (1973). The results generally showed that the specimens were about 80% consolidated at the end of the increments. The degree of consolidation was generally about 95% at the end of final increment before shearing.

The reversibility of volumetric deformations due to isotropic loading and unloading have been examined by specimen T929 in Figure 5.10. This specimen was subjected to four equal increments of

isotropic compression up to 2.28 MPa pressure. The volume strain relationship with elapsed time for the loading increments are shown in Figure 5.9. After the fourth increment the specimen was unloaded isotropically in two equal decrements to 1.14 MPa pressure. The specimen was then reloaded in two equal increments to 2.28 MPa pressure and a final increment increased the pressure to 3 MPa. Each of the pressure increments and decrements lasted for 5 days. Figure 5.10 shows the resultant relationship between volumetric strain (compression or expansion) with elapsed time. It can be seen that the volumetric expansion in unloading is approximately equal to the volumetric compression upon reloading, but considerably smaller than the volumetric compression during first loading. That is to say that the deformation of compacted buffer consists of both elastic and plastic components and only the elastic component is recoverable due to short term unloading. Thus, the swelling characteristics of the expansive buffer does not seem to be able to undo entirely the macrostructural reorientation that had previously taken place in compression. This conclusion agrees with earlier work by Lambe (1958), Seed et al. (1962), and Musa (1982). It is later postulated in Chapter Six that the dispersed fabric structure of the compressed buffer can not be altered enough to allow complete reversal of deformations, unless extensive swelling takes place to the extent that the structure is rearranged to a flocculated structure under very small confining pressure (less than 100 kPa).

Figure 5.11 - 5.12 summarize the isotropic consolidation data given in Figures 5.7 - 5.8 for T925 and T930 respectively. The '1-day', '3-days' and '5-days' contributions to the strain displacements

have been evaluated separately (Graham et al., 1983) and plotted against corresponding p' -values normalized by the final isotropic consolidation pressure σ_{cons} . This allows comparison of the bulk stiffness of the material at different stress levels and times. Comparison of these results with similar tests performed at lower pressures by Sun (1986) will be presented in Section 6.2.

5.3.1.3 Multi-stage Consolidation tests under elevated back pressure

Results from Multi-stage consolidation tests with different back pressures in Series (IV) to examine the effective stress principle are shown in Figures 5.13 and 5.14. According to the effective stress principle, if there is no change in the effective stress then the specimens should exhibit no volume straining. Conversely, any measured volume changes would imply some changes in the real interparticle stresses. Two consolidated specimens T927 and T933 showed small additional volumetric strains less than 0.3% and 0.05% due to changes in back pressures from 0.2 to 1.8 MPa and from 1 to 7 MPa, respectively as illustrated in Figures 5.13 and 5.14. For example, specimen T933 in Figure 5.14 was initially consolidated in stage (I) to 4 MPa confining pressure and 1 MPa back pressure. Consolidation essentially ceased after 8 days when about 1.2% volume strain was measured. Subsequently, the confining pressure was increased in stage 2 to 10 MPa and the back pressure to 7 MPa keeping the nominal effective stress constant. A volume strain of about 0.05% was recorded after seven days. Care has been taken here with calibrating the back pressure burette and cell leads for volume expansions and creep. These results suggest that at high elevated back pressures the effective stress

principle is still valid.

Two other multi-stage tests were also conducted on specimens T923 and T932. However, the recorded volume changes were not consistent with the measured end-of-test water content due to leakage problem later detected under high back pressures. Nevertheless, the discrepancies between the measured end-of-test water contents and the calculated water contents at the end of stage (I) of the consolidation under small applied back pressure were 0.6% and 1.3% respectively. This confirms the earlier hypothesis that no appreciable volume change is expected due to increase in back pressure while the effective stresses are kept constant. Nevertheless, factors such as incomplete saturation of the specimens and slightly different soil fabric microstructures may contribute to small apparent deviations from the expected no volume change behaviour. The results generally confirm earlier work by Wan (1987) under low effective confining pressures. Back pressures of the magnitudes used in this study are exceptionally rare in geotechnical testing and provide an important extension to the range of acceptability of the effective stress principle. The tests are directly relevant to the Canadian Nuclear Waste Management Program where effective stresses in the buffer may remain small but porewater pressures may be very high.

5.3.2 Undrained Triaxial Shear

After consolidation, the majority of the specimens in this program have been sheared undrained in strain-controlled triaxial compression with porewater pressures measured just outside the cell base. These tests are used to define the shear stress-shear strain characteristics

of the compacted buffer.

5.3.2.1 Stress-Strain Characteristics and Porewater Pressure Generation

Figures 5.15 - 5.21 show some typical undrained shear stress-shear strain curves for the buffer compacted to different densities and consolidated to various pressures. Figure 5.18 also presents the stress-strain behaviour of the buffer subjected to unload-reload cycles. The upper part of each figure shows (1) normalized deviator stress q/σ'_{cons} and (2) normalized porewater pressure change $\Delta u/\sigma'_{\text{cons}}$ plotted against axial strain ϵ_1 (or shear strain ϵ_s). The use of the confining pressure σ'_{cons} here allows simpler presentation of the data at consistent scales, and simpler evaluation of the effects of confining pressure on stress-strain behaviour. The lower part of each figure shows the variation of stress ratio $q/p' = 3(\sigma'_1 - \sigma'_3)/(\sigma'_1 + 2\sigma'_3)$ versus axial strain.

The stress-strain curves in Figures 5.15 - 5.21 are reasonably consistent, indicating in general terms a slightly strain-softening material. The porewater pressure response indicates both compressive and dilative behaviour depending on the compacted density and consolidation pressure. "Failure" in the specimens has here been defined as the maximum value attained by the deviator stress $q = (\sigma'_1 - \sigma'_3)$. Values of the principal test parameters at failure are given in Tables 4.6 and 5.1. The axial strains to failure ϵ_{1f} generally lie in a fairly restricted range from 4% to 5%.

The failure deviator stresses and porewater pressures in Table 4.6 can be expected to be influenced by their corresponding consolidation

pressure. The influence of consolidation pressure can be removed if q_f and Δu_f are divided by the appropriate effective confining pressure σ'_{cons} . When this is done in Table 5.1, many of the values of $q_f/\sigma'_{\text{cons}}$ lie between 0.4 and 0.6. Corresponding values of $\Delta u/\sigma'_{\text{cons}}$ are from 0.2 to 0.4. In each test series however, there are marked deviations from these average values, for example T905, T913 and T925. These have resulted in part from differing test conditions; from different end-of-consolidation equilibrium "states" relative to the swelling equilibrium line; or from problems with equipment during the tests. However, two rather different patterns of behaviour can be distinguished which will be discussed in the following chapter.

Values of the commonly used porewater pressure parameter $A_f = \Delta u_f/\sigma'_{\text{cons}}$ (Skempton, 1954) are deduced from porewater pressure response in Figures 5.15 - 5.21 and listed in Table 4.6. Another way of examining porewater pressure generation is based on the relationship between porewater pressure changes Δu and total mean pressure changes Δp (Graham et al, 1986). In quasi-elastic linear soils the Δu vs. Δp relationship is straight (Graham and Houlsby 1983; Wood and Graham 1987). Isotropic soils have $m = \Delta u/\Delta p = 1.0$ whereas in anisotropic soils, $m \neq 1.0$. Moving from elastic behaviour through a yielding process towards rupture produces marked changes in the Δu vs. Δp relationships and these can be used to distinguish easily between compressive and dilative behaviour, respectively on the "wet" and on the "dry" side of the Critical State Line CSL.

The graphs of Δu versus Δp are shown in Figures 5.22 - 5.25. They generally show compressive behaviour at confining pressures higher than the swelling pressure, and dilative behaviour under lower confining

pressures. For example, specimens T928 (Figure 5.23) and T932 (Figures 5.25) show clear evidence of dilative behaviour and conform with the pattern established for the buffer by Graham et al (1986a) and by Wan (1987).

It will be remembered that specimens in series (II) were all formed in such a way that they should have been at swelling equilibrium, and indeed they experienced only small compressive (or expansive) volume strains during consolidation. The buffer would therefore be expected to be compressive in shear. However, some of the specimens were in fact dilative in shear. This question have also been raised by Wan (1987) and will be addressed in detail in Chapter 6. Values of $m = \Delta u / \Delta p$ in the early approximately linear sections of these figures are given in Table 4.7. Most of the m -values are significantly greater than unity, indicating that the compaction and specimen-forming process built an anisotropic fabric structure into the buffer. It is apparent in Table 5.1 that specimens which were compressive during consolidation and shear had high A_f - and m -values. This could be attributed to the anisotropic particle microstructure formed during compaction which was not completely changed during the subsequent consolidation and shearing procedures. Conversely, specimens that expanded during consolidation had significantly lower porewater pressure parameters that indicate some modification of the compaction microstructure towards a more isotropic arrangement (Wan 1987). The data is better demonstrated in Figure 5.26, where the specimens that showed compressive behaviour during testing had larger porewater pressure parameters A_f than the specimens that were dilative during shear.

The magnitudes of porewater pressure drop in shear between peak and 12% axial strain for all dilative specimens tested in this program and reported by Sun (1986) are summarized in Table 5.2. This data combined with the results reported by Wan (1987) are normalized with respect to consolidation pressure and plotted against consolidation pressure and end-of-consolidation water content in Figures 5.27 and 5.28 respectively. It is apparent that the tendency of the buffer to dilate diminishes with increasing consolidation pressure and decreasing end-of-consolidation water content. Careful examination of all the results suggest that the variability of the porewater pressure parameters depended on the confining pressure at which the test was performed. There is a clear relationship between the confining pressure and the amount of swelling or compression experienced by the specimens (Table 4.5) and so the two parameters are alternative ways of expressing the same idea. Understanding the buffer interparticle arrangements, however, is useful since it helps to explain the mechanical response of the buffer as related to details of changes taking place at the microstructural level.

5.3.2.2 Undrained Shear Modulus:

A measure of the shear stiffness of the buffer can be estimated from the undrained test data presented typically in Figures 5.15 - 5.21. Here, the shear stiffness is taken as the secant modulus between zero and $0.5q_{\max}$. Graham et al. (1986a) discussed the relationships between the various moduli that can be interpreted from the observed measurements. In undrained tests the Young's (compression) modulus E_{50} is the slope of the q, ϵ_1 -curves between zero and $0.5q_{\max}$, and $G = E/3$.

Values of E_{50} are given in Table 5.3. These have been calculated as secant-values from the beginning of shear to $q_{\max}/2$. In some cases corrections have been made at the beginning of the stress-strain curves to take account of bedding-in difficulties and zero shift. Table 5.3 also shows values of E_{50} normalized with respect to the undrained strength s_u and the consolidation pressure σ'_{cons} . The average values of $E_{50}/\sigma'_{\text{cons}}$ and $G_{50}/\sigma'_{\text{cons}}$ from all the specimens sheared undrained are 49 ± 21 and 16 ± 7 respectively. As is usual in soil testing the prediction of modulus values is usually associated with a high level of variability. Figures 5.29 and 5.30 show the relationships between $E_{50}/\sigma'_{\text{cons}}$ and E_{50}/s_u and consolidation pressure respectively. The value of normalized elastic Young modulus $E^e/\sigma'_{\text{cons}}$ of the buffer measured from unload-reload cycles in Figure 5.18 is estimated to be 75. This is about 50% higher than the average value obtained for $E_{50}/\sigma'_{\text{cons}}$.

The value of normalized shear modulus interpreted in this way from the undrained shear tests is higher than the 3-day corresponding secant value of 12.5 ± 4.5 determined from the drained, incremental shear tests at constant p' in Section 5.4. It will be remembered that the undrained shear tests incorporate a shorter period of creep straining than the drained tests and it is therefore reasonable that they show higher shear moduli (Wroth et al. 1979).

5.3.2.3 Undrained Effective Stress Paths

Figures 5.26 and 5.27 summarizes the effective stress paths in terms of stress invariants, q and p' , during undrained shear for specimens compacted at low and high densities respectively. The specimens compacted wet of the optimum moisture content (or at low

densities) are shown in Figure 5.26. Except for anomalous stress paths for a few specimens with very low B-values discussed in Section 4.4.1 which were discarded; two different patterns of fairly consistent behaviour was observed. The specimens consolidated to a pressure less than the compaction swelling pressure tend to be dilative in shear, therefore bend towards the right. This corresponds to lightly overconsolidated behaviour in classical soils mechanics terms. A distinct feature of the behaviour of these specimens is the early portion of the stress paths consisting of a relatively linear "elastic" section to over 50% of maximum shear stress. The inclination of the linear stress paths with respect to the vertical (q-axis direction) reflects inherent anisotropy of the specimens due to one dimensional compaction of the mixture in the mould during specimen formation. The inherent anisotropy of the compacted buffer has become evident in the porewater pressure response in Figures 5.23 - 5.26 as discussed earlier in Section 5.3.2.1. Graham and Houlsby (1983) related the degree of inclination of the stress path to the relative stiffness of the specimen in horizontal and vertical directions. For an isotropic elastic material in undrained shear at constant volume, the mean effective stress remains constant, and therefore the stress path rises up vertically in q, p'-plane.

Specimens consolidated to a pressure more than the compaction swelling pressure exhibit compressive behaviour, moving leftwards in q, p-plane in shear and producing large porewater pressures. This corresponds to normally consolidated behaviour in classical soil mechanics terms. The anisotropic "elastic" portion of the stress paths tend to be relatively shorter for compressive specimens. The shape of

the yield surface (see Section 2.4.3) is expected to be non-symmetrical for anisotropically compacted buffer (Graham et al. 1983). That results in the initial stress path being inside the yield loci and the buffer behaviour being initially "elastic" even in the normally consolidated stress range.

Similar patterns of behaviour in shear was observed for specimens compacted dry of optimum (high density specimens) as shown in Figure 5.32. The only difference being the magnitude of the swelling pressure or the transition pressure from dilative behaviour to compressive behaviour. In summary, many of the features commonly associated with anisotropic overconsolidated and normally consolidated soils (Wood 1984) were observed in the behaviour of the compacted buffer.

5.3.2.4 Strain Rate Effects

The effect of strain rate on the undrained shear strength of the compacted buffer was examined by; 1) step-changing the strain rate in strain controlled tests and 2) conducting relaxation tests on four specimens T915, T917, T920 and T921. The resultant stress-strain curve for specimen T915 is shown in Figure 5.15.

Results from the step changing strain rate tests on three specimens T915, T917 and T920 indicate that a tenfold increase in strain rate increases the undrained shear strength by about 6%. Here, the strain rate was step changed from 0.05% / hour to 0.25% / hour and 1.25% / hour and vice versa, allowing steady state conditions to be established at each strain rate. Figure 5.33 shows a typical relationship between undrained shearing resistance s_u and $\text{Log}(\text{strain rate})$ from a relaxation test on Specimen T921. It is clear that the

undrained shearing resistance of the buffer increases approximately linearly with $\text{Log}(\text{strain rate})$. The change in shearing resistance can be described by the strain rate parameter, $\rho_{0.1}$ (Graham et al. 1983a). This parameter is defined as the percentage change in undrained shearing resistance at 0.1% / hour, produced by a tenfold change in strain rate. From all the relaxation tests performed the average $\rho_{0.1}$ was determined to be about 5%. This is slightly lower than the same parameter obtained from step changing strain rate tests. Comparison of these results with earlier work by Wan (1987) on lower density buffer indicates that the $\rho_{0.1}$ parameter is not significantly influenced by the density of the compacted buffer.

5.4 Drained Shear

The drained shear tests were performed (1) to compare the shear strength envelopes from strain controlled and load controlled tests, and (2) to examine the relationship between shear stiffness, shear stress and time. Five specimens T910, T911, T916, T918 and T931 were loaded with a series of deviator stress increments while the mean stress p' was held constant. The drainage leads were open throughout this process. Loading continued until the axial deformations accelerated towards failure. The failure is here defined as the onset of the strain rate increase as shown for instance in Figure 5.34. One Specimen T931 in Figure 5.35 was also subjected to an unload-reload cycle prior to failure to examine the elastic shear stiffness of the buffer. Another Specimen T934 was sheared in a conventional strain controlled drained test to examine the applicability of a three modulus

hypoelastic constitutive model developed for the buffer (Chapter 7) for stressing along a conventional drained stress path.

Typical resulting graphs of ϵ_1 and $\dot{\epsilon}_1$ vs. time in constant- p' tests are shown in Figures 5.34. Since the mean pressure changes were zero in constant- p' tests, the porewater pressures generated at least in the early stages of loading arise only from the anisotropic elasticity of the clay microstructure. This proposition is based on the assumption that end of primary (EOP) consolidation was reached prior to shearing. This is not strictly true for all the specimens tested and some volume change correction may be necessary, as will be discussed in Section 7.3.2, to evaluate the volumetric strain produced by anisotropy of the microstructure. It is important to remember that in the final loading increment that led to failure, fast "creep" strains probably have caused increasing porewater pressures and moved the effective mean pressure leftwards in p', q -space towards the strength envelope. It is not possible to measure these porewater pressure transients inside the specimen and there is therefore some uncertainty about the precise stress state when failure actually occurred. However since the last deviator stress increment was relatively small the uncertainty in the value of p'_f at failure is not large. To reduce the margin of error the size of the final load increment was cut in half where failure was deemed to be imminent.

The purpose of these tests was to develop further information on the shear stress - shear strain - time behaviour of the buffer with a view to developing suitable constitutive relationships (Chapter 7) for use in computer modelling of the container installation. To assist in comparing results from tests with differing mean pressure levels, the

results have been expressed as in terms of $q/q_{\max}$ versus shear strain ϵ (Graham et al. 1986). A typical 1-day, 2-day, 3-day and 5-day shear stress - shear strains relationship is presented in Figures 5.36. Figure 5.37 shows a similar stress-strain relationship for specimen T931 which included an unload-reload cycle. The shear stress- shear strain curves are strongly non-linear. As shear failure is approached then large shear strains are observed. The stiffness of the buffer appears to be both time and shear stress level dependent (Houlsby 1985).

Conventional practice has been to quote secant stiffness which is easy to extract, however, modern constitutive modelling theories are cast in incremental form, and for small increments stiffness approaches the tangent values. Atkinson et al. (1986) proposed a new method for determination of tangent stiffness parameters from soil test data. The stiffness parameters for the buffer are presented in Table 5.3 in terms of normalized tangent and secant shear modulus $G_{50}/\sigma'_{\text{cons}}$ measured from the 3-day curves at, and up to, $0.5q_{\max}$. The measured values of tangent and secant modulus $G_{50}/\sigma'_{\text{cons}}$ range from 2.7 to 9.6, and 6.4 to 18.2 respectively. The average values are 7.0 ± 2.7 and 12.5 ± 4.5 respectively. The normalized shear modulus $G_{50}/\sigma'_{\text{cons}}$ generally varies with σ'_{cons} pressure (Graham et al. 1987; Domaschuk and Valliappan 1975). This will be discussed further in Section 6.3. The value of normalized elastic shear modulus $G^e/\sigma'_{\text{cons}}$ measured from the 3-days unload-reload curve in Figure 5.37 is approximately 37.5. This is three time larger than the average normalized secant shear modulus up to $0.5 q_{\max}$, $G_{50}/\sigma'_{\text{cons}}$, of 12.5 from drained tests; and more than twice as large as the average normalized secant shear modulus up to $0.5 q_f$, $G_{50}/\sigma'_{\text{cons}} =$

16 measured from undrained tests. This indicates that a large portion of the shear deformations in drained tests are unrecoverable as evident in Figure 5.37.

5.5 Coulomb-Mohr Strength Envelope

Figures 5.31 and 5.32 show detailed stress paths for low density and high density specimens respectively at mean effective pressures up to 10 MPa. It can be seen that the majority of specimens in the high pressure range (2.2 to 10 MPa) in Figures 5.31 and 5.32 were compressive in nature and therefore the stress paths move leftwards (towards lower p' -values) in the q, p' -plots shown. The failure envelopes determined from these results up to 3.5 MPa mean effective pressure can be characterized by straight lines (not shown in the figures) : $c' = 46$ kPa, $\phi' = 14^\circ$ in an "overconsolidated" range that is associated with dilative "short-term" specimens; and $c' = 0$ kPa, $\phi' = 14^\circ$ for normally consolidated specimens that were consolidated for longer periods and were compressive in shear. These strength envelopes correspond to the strength of the clay phase of the mixture. The particle structure of the mixture has been examined by electron micrographs such as those shown in Figure 2.6. Figure 2.6 (a) shows the microstructural fabric of the buffer at the magnification of approximately 40 times, where larger angular particles of the quartz sand separated by a matrix of clay and finer sand particles. At larger magnification (330 times in Figure 2.6 (b)) the smaller sand particles are still separated from each other, this time by a matrix of clay particles only. This supports the results presented in Figure 5.38,

indicating that the behaviour of the buffer is controlled by the density of the clay phase, with the sand acting as a filler.

The measured strength of $\phi' = 14^\circ$ in this testing program at pressures up to 3.5 MPa is rather smaller than the corresponding value of $\phi' = 16^\circ$ reported in lower stress range up to 1.5 MPa (Graham et al. 1986a). In addition, in higher stress range from 3.5 MPa up to 10 MPa (Figure 5.38) the end-of-test strength of the buffer appears to increase much more slowly, pointing to a curved strength envelope. At higher pressures the evidence supports lower "local" ϕ -values ($\phi = \arcsin(3M/(6 + M))$) with increasing consolidation pressure p'_{cons} . Therefore, a curved strength envelope in the form of a power law which can represent the whole range of stresses up to 10 MPa is here proposed for compacted low density buffer as follows:

$$q = 2.29 p'^{0.79} \quad [5.1]$$

The resultant R^2 of 0.97 indicates excellent coefficient of correlation. Mesri and Olsen (1970) also used a curved strength envelope to represent the behaviour of remolded montmorillonitic clays.

Figure 5.38 indicates that the end-of-test strength of high density buffer tends to be considerably higher than the above envelope. Thus, the high density data is considered separately in Figure 5.39 which results in the following strength envelope:

$$q = 2.64 p'^{0.79} \quad [5.2]$$

The R^2 value of 0.88 indicates good correlation with some scatter in the data. Figure 5.40 shows the failure strength envelope for the high density buffer based on peak results. This envelope can be expressed mathematically by:

$$q = 2.59 p'^{0.80} \quad [5.3]$$

The R^2 value of 0.94 gives a satisfactory coefficient of correlation. This shear strength envelope is slightly higher (approximately 4%) than the end-of-test strength. Although the difference between Eq. [5.2] and Eq. [5.3] is marginal, it reflects the strain softening nature of the material behaviour in shear.

CHAPTER SIX

DISCUSSION OF TEST RESULTS

6.1 Introduction

The purpose of the research has been to develop a constitutive stress-strain-time model for laboratory compacted buffer as discussed in Section 1.4. Development of a predictive methodology for behaviour of the buffer would provide an essential component for the design of waste repositories. An initial understanding of the behaviour of low density buffer (85% modified ASTM density) was obtained in earlier investigations by Sun (1986) and Wan (1987). To complement and extend the earlier work, thirty five triaxial tests were performed at pressures up to 10 MPa on specimens compacted to different densities from 85% to 100% ASTM modified density. The test results were described in Chapter 5 involving long-duration tests on this complex time- dependent and pressure- dependent material.

This chapter will discuss the results in view of earlier work by Graham et al. (1986). Particular attention will be paid to the influence of initial compaction density, buffer microstructure and stress level. A conceptual model based on Critical State Soil Mechanics is proposed in this chapter which provides a coherent framework for describing the behaviour of the buffer compacted to different densities. This model accounts for the influence of the

buffer fabric structure and offers convincing explanations for the complexity of the buffer behaviour in the consolidation and shearing phases of testing.

6.2 Consolidation

Two aspects of the consolidation behaviour deserve special attention. First, incremental isotropic consolidation tests have provided useful information on the time-dependency of volume compressibility of the buffer in a wide range of stresses up to 10 MPa.

Second, extensive work on the long term swelling pressure (also "equilibrium" or "consolidation" pressure) versus clay specific volume relationship of the buffer compacted at various densities has led to better understanding of the consolidation behaviour, as it relates to the microstructural fabric of the buffer.

6.2.1 Time-Dependent Isotropic Compression

The volume change behaviour of the buffer during successive increments of consolidation pressure p' is greatly influenced by the magnitude of the confining pressure (Table 4.5). The pressure increments were added with a constant ratio $p'_{i+1}/p'_i = 1.32$ and kept constant for 5 to 7 days. Figure 6.1 (Sun 1986) shows a typical volume strain response with arithmetic (time) from a test at low confining pressure ($\sigma_{\text{cons}} = 0.2$ MPa). The data from Figure 6.1 are replotted in Figure 6.2 in terms of $\log(\text{time})$ rather than (time) . After initial compression, the straining slowed considerably and volume strain rates

less than 0.1% - 0.2% per day were generally achieved at the end of each increment in most of the tests. However, it should be remembered that because of the low hydraulic conductivity of the buffer, approximately 10^{-12} m/s (Dixon et al. 1987), consolidation generally takes a few weeks to complete.

In the first stress increment in Figure 6.1 ($p' = 0.07$ MPa) the specimen compressed initially as water that had been absorbed while setting up the test was squeezed from the outer skin of the specimen. However after about 0.1 day, the specimen began to swell. It continued swelling until the end of the increment at 23 days when an expansive volume change approaching -11% was observed. For the second load increment ($p' = .09$ MPa) the specimen began to swell slightly after about .1 days. Then on successive increments it compressed slowly and continuously until the end of that loading increment. There was a small tendency for the compression rate in $\log(\text{time})$ to decrease towards the end of several of the increments.

The long term swelling characteristics of the buffer were found to be inhibited by increased confining pressure. Figures 5.6 and 5.7 showed the volume strain-time behaviour of typical compressive specimens (T915 and T925) during successive increments of consolidation pressure in the medium and high ranges of pressures up to 3 Mpa and 10 MPa respectively. In each of the load increments shown in Figure 5.6, for example, the specimen experienced compressive strains that continued through until the end of the increment. The strains in the first increment were largest. They were smaller in the second increment, and then increased steadily with the confining pressure. As the confining pressures increased in the experimental program, larger

compressive volume strains were observed.

Figure 5.11 shows the consolidation pressure vs. volume strain relationship for the compressive specimen T915 shown previously in Figure 5.6. The strains have been based on accumulated 1-day, 3-day, and 5-day strains (Mesri and Godlewski 1977; Graham et al. 1983). It is clear from Figures 5.11 and 5.12 that the pressure-volume-time behaviour of the buffer is non-linear and complex.

The 3-day consolidation pressure-volume strain relationships for all the incrementally consolidated low density specimens except T905 in this program are summarized in Figure 6.3. (The results of specimen T905 have been discarded because additional water was flushed between the specimen and the membrane in attempting to remove excess air. This procedure has since been superseded). The p',v relationship in Figure 6.3 for the low density specimens reflect the variation of bulk stiffness of the material with consolidation pressure. The results produce a clearly defined relationship between mean pressure and volume straining in the buffer. This is a relationship that will later form one of the prime starting points in forming constitutive models of the buffer behaviour in Chapter Seven.

For purposes of comparison, the results can be described by the slope K of a straight-line approximation to the p',v relationship in the final loading increment in each of the tests. It is accepted that this is an approximation to the observed non-linear behaviour and will be superseded in subsequent constitutive modelling. The resulting values of the bulk modulus K are presented in Table 6.1. It can be seen in Figure 6.4 that when K is normalized with respect to the mean pressure $\bar{\sigma}_{cons}$ in the final increment then it appears to vary only a

small amount with pressure. Table 6.1 gives normalized values of $\bar{K}/\bar{\sigma}_{\text{cons}}$ calculated in this way from measured 3-day strains.

Normalized bulk stiffnesses from 1-days, 3-days and 5-days strains have been plotted against consolidation pressure in Figure 6.5. It is clear that the continued straining during the pressure increments causes the stiffnesses to be time-dependent. However, the normalized stiffness for a given time period does not seem to be affected in a major way by consolidation pressure. The 3-days values in Figure 6.4 appear to be in the range 20 - 40 (with some exceptions in the tests at 0.5 MPa). The time-dependent data in Figure 6.5 can be expressed by a hyperbolic function of time reflecting mobilization of stiffness as;

$$\bar{K}/\bar{\sigma}_{\text{cons}} = T / (0.05 T - 0.04) \quad [6.1]$$

The R^2 value of 0.95 obtained indicates good correlation of the normalized stiffness with time.

It should be remembered that the material behaviour is influenced strongly by the expansive characteristics of the buffer at low pressures. In Series (I) compacted to 1.5 Mg/m^3 dry density, the swelling potential of the mixture is inhibited when the consolidation pressures exceed the "equivalent" compaction swelling pressure about 0.8 - 1.0 MPa. The equivalent swelling pressure is higher about 2.1 - 2.2 MPa for specimens in Series (III) which are compacted to 1.67 Mg/m^3 dry density. To determine the coefficient of volume compressibility of the buffer m_v for settlement calculations the values of bulk stiffness in Table 6.1 should be inversed. Alternatively, the compression index can be evaluated as the slope of the compression line in $V - \lambda \text{LN}(p')$ plot. The compression index and the bulk stiffness are related by the following expression:

$$\lambda = (V p' / K) \quad [6.2]$$

Similar relationship exists between the swelling index k and the elastic bulk stiffness K^e as follows :

$$K = (V p' / K^e) \quad [6.3]$$

The time-dependency of the elastic-plastic and elastic compression indices can be examined using the incremental consolidation data from specimen T929 with an unload-reload cycle as described in Section 5.3.1.2. The relationship between the specific volume V and the effective consolidation pressure p'_{cons} (in natural log-scale) is shown in Figure 6.6, for 1-day, 3-day and 5-day consolidation times. As expected, the compressibility and swelling both increase with time. The results are summarized in Table 6.3 where the ratios between the compressibility index and swelling index λ/K from 1-day, 3-days, 5-days data are found to be 2.0 , 1.9, and 2.0 respectively. On the basis of this data, the ratio between the compression and swelling indices appears to be independent of time. This would then allow the estimation of the long term swelling index from the long term consolidation data in the following section. Further discussion of the time-effect will be presented in Section 6.2.3.

6.2.2 "Long Term" Isotropic Compression

"Long term" end-of-consolidation (EOC) data from all low density and high density specimens tested in this program and earlier programs (Sun 1986; Wan 1987) are plotted in $\ln p'_s, V_c$ -plane in Figure 6.7. The resulting relationship is slightly curved rather than straight as would normally be assumed. The same data have been plotted against

$\ln(V_c)$ in Figure 6.8 (Wroth and Houlsby 1985). Although there is some scatter, a best-fit regression line has been fitted through all the data, including dilative specimens. The resulting relationship is:

$$(\ln V_c) = 1.636 - 0.106 (\ln p'_s) \quad [6.4]$$

The R^2 -value of 0.90 obtained indicates a fairly good correlation. This differs considerably at low stresses from the relationship shown in Figure 2.13 from Dixon et al. (1986). It is postulated that the 1-D specimens consolidated from slurry by Dixon et al. (1986) possessed a more flocculated structure than the compacted specimen in the University of Manitoba studies, therefore the resultant specific volumes are higher for 1-D specimens. In addition, there is some systematic tendency in Figures 6.7 and 6.8 for triaxial specimens that would later be compressive during shearing to lie above the consolidation best-fit line. Conversely, dilative specimens tend to lie below the best-fit line. Similar trends have been reported by Wan (1987). Therefore, it becomes necessary to differentiate between the swelling equilibrium line (SEL) for dilative specimens and the normal consolidation line (NCL) for compressive specimens in Figure 6.8. This means that the microstructure of an expansive specimen compacted to an equivalent pressure higher than the consolidation pressure (say $p'_1 > p'_{cons}$) is considerably different from that of a compressive specimen compacted to an equivalent pressure lower than the consolidation pressure (say $p'_2 < p'_{cons}$). This idea is further discussed (Lambe 1958 and 1965) and illustrated more completely in Section 6.2.3.

Further information about the time-dependency of the material

behaviour was obtained by determination of the contribution of the secondary compression for the buffer. Table 6.2 shows values of the secondary compression coefficient $C_{\alpha\epsilon}$ which has here been determined using the procedure proposed by Sridharan and Rao (1982) and Sridharan and Prakash (1985). Figure 6.9 shows the relationship between $C_{\alpha\epsilon}/C_c$ and consolidation pressure for "long duration" tests in Series (I). The results lie mostly in the range 0.09 - 0.14 which is higher than the range 0.04 - 0.08 proposed by Mesri and Godlewski (1977). Figure 6.9 can also be tentatively interpreted as suggesting some decrease in the secondary compression coefficient $C_{\alpha\epsilon}$ with increasing consolidation pressure. This observation is consistent with the results reported by Sridharan et al. (1986) for monovalent (sodium, lithium and potassium) bentonites as would hopefully be expected from the sodium-rich nature of the Avonseal bentonite used in this program (Quigley 1984). The extensive diffuse double layer formation in sodium and lithium bentonites, in general, decreases the ability to carry effective "contact" stresses and results in relatively high coefficients of secondary compression (Mesri and Olson, 1970; Sridharan et al. 1986).

6.2.3 Influence of Fabric Structure on Equilibrium Condition

The mechanism of consolidation and secondary compression can be described in general as a continuous process of detailed change in the soil structure involving reorientation of clay platelets and domains (Yong and Warkentin 1975). The influence of initial compaction density on the mean stress-strain relationship of the buffer is shown in Figure

6.10. Here, the 3-days consolidation results for specimen T925 compacted at 1.50 Mg/m^3 are compared with specimen T930 compacted at 1.67 Mg/m^3 . Both specimen were consolidated incrementally to 9 MPa consolidation pressure. The high density specimen T930 exhibited higher stiffness (or lower compressibility) under increasing consolidation pressures. While the compressibility of specimen T930 tends to decrease more rapidly with increasing consolidation pressure than specimen T925 as shown in Figure 6.11.

The end of consolidation (EOC) data in Figure 6.8 have therefore been regrouped in term of the initial compaction density into "high density" and "low density" specimens as described in Section 6.1. It can be seen in Figure 6.11 that the EOC specific volumes for high density specimens are consistently lower than the lower density specimens at the same consolidation pressure. Separate best fit regression lines through low density specimens and high density specimens data lead to the following equations for the normal consolidation line. For low density specimens

$$V_c = 4.331 - 0.261 \text{ LN } (p') \quad R^2 = 0.92 \quad [6.5a]$$

$$\text{(or)} \quad \text{LN } (V_c) = 1.609 - 0.099 \text{ LN } (p') \quad R^2 = 0.95 \quad [6.5b]$$

and for high density specimens

$$V_c = 3.541 - 0.171 \text{ LN } (p') \quad R^2 = 0.95 \quad [6.6a]$$

$$\text{(or)} \quad \text{LN } (V_c) = 1.367 - 0.075 \text{ LN } (p') \quad R^2 = 0.89 \quad [6.6b]$$

The resultant R^2 values indicate good correlations between the measured and regressed values. In saturated buffer, the diffused double layer repulsive forces govern the volume change behaviour (Sridharan and Rao

1973). Thus, these repulsive forces should be influenced by the microstructural variations associated with different initial compaction conditions (Seed and Mitchell 1962). That is, high density buffer compacted at lower moisture content (Series III) forms a more orderly (dispersed) microstructure (Sposito and Giraldez 1976) than low density buffer. It therefore should be less compressible under isotropic compression. The lower density specimens (Series I) compacted at higher moisture contents, on the other hand, form a more orderly (dispersed) fabric structure compared to remolded 1-D specimens tested by Dixon et al. (1986). Thus the compressibility of 1-D specimens with more flocculated (disorderly) structure is naturally higher as shown in Figure 6.11.

To examine the above postulation quantitatively a multiple regression analysis of consolidation data was performed. The relationship between the EOC specific volume V_c , consolidation pressure σ'_{cons} , and initial compaction specific volume V_{ci} can be expressed mathematically as follows:

$$V_c = 3.158 (p'_{cons})^{-0.091} (V_{ci})^{0.420} \quad [6.7]$$

$$(or) \quad LN(V_c) = 1.150 - 0.091 LN(p'_{cons}) + 0.420 LN(V_{ci}) \quad [6.8]$$

The resulting R^2 of 0.95 indicates excellent correlation. This represents a clear improvement compared to R^2 values of 0.92 and 0.95 for Eqs. [6.5] and [6.6]. The improvement is more pronounced in comparison with Eq. [6.4] where no distinction was made between high and low density specimens. Eq [6.7] clearly demonstrates that the

compressibility of the compacted buffer depends on the initial compaction specific volume (density). The influence of the initial compaction density on the "equilibrium" clay specific volume can be more closely illustrated in Figure 6.12. It can be seen consistently in this figure that higher initial compaction density leads to lower "equilibrium" clay specific volume under the same consolidation pressure. Eq. [6.8] represents a series of straight regression lines for different initial compaction densities in Figure 6.11. This could be conceptually similar to "time" lines in delayed compression (Bjerrum, 1967). Eqs. [6.5] and [6.6] approximate two of these possible "equilibrium" states in particular at about 85% and 95% modified ASTM standard density respectively. Thus, Eq. [6.8] generalizes the volume change behaviour of the buffer under isotropic compression. This is an important component for the development of a general isotropic constitutive model with the capability to account for variations of the initial fabric structure of the buffer.

Another important parameter which may affect the EOC "equilibrium" specific volume is the consolidation period given in Table 4.5. The data from Series (I) with the same compaction density in Figure 6.13 are labelled with their corresponding consolidation periods in days. The scatter of the EOC data under a given consolidation pressure could then be mainly attributed to the effect of time reflecting the actual degree of consolidation. It can be seen in general that for swelling specimens, the longer the consolidation period the higher the "equilibrium" clay specific volume. Conversely for compressive specimens, the longer the consolidation period the lower the "equilibrium" clay specific volume. To account for the consolidation

time in Eq. [6.8] another multiple regression analysis of data was conducted. The result could be expressed as follows:

$$V_c = 3.193 (p'_c)^{-0.09} (V_{ci})^{0.44} (T)^{-0.014} \quad [6.9]$$

where T represents the consolidation duration in days. The resultant R^2 value of 0.96 indicates an excellent correlation. However, the improvement relative to Eq. [6.8] is marginal. On the basis of this equation the consolidation duration has a minor influence on the "equilibrium" clay specific volume. For example, the clay specific volume could drop by approximately 2% if the consolidation duration is prolonged by a factor of 5, say from 2 weeks to 10 weeks. This provides additional justification for the presumption of the "end of consolidation" when the volume straining rate drops to 0.1% / day as discussed in Section 5.3.1.1. Because of the low hydraulic conductivity of the buffer consolidation and swelling processes involving reorientation of clay platelets and domains requires several weeks to reach 100% completion.

6.3 Shear Stress-Shear Strain Relationships

6.3.1 Undrained Shear

The stress-strain characteristics of the buffer in undrained shear and the associated porewater pressure generation were discussed in Section 5.3.2.1. To examine the repeatability of undrained shear tests, the stress-strain behaviour of two specimens consolidated to the same consolidation pressure are compared in Figure 6.15. Both specimens were compacted to 1.67 Mg/m^3 dry density in Series (III) and exhibited dilative behaviour in shear. The agreement in general

between the two curves is satisfactory. The normalized stress-strain behaviour of four compressive specimens in Series (III) is compared in Figure 6.16. Specimens T914 and T921 were consolidated to 2.2 MPa, while specimens T920 and T929 were consolidated to 3 MPa. It can be seen that when the results are normalized with respect to the consolidation pressure, three of stress-strain curves are virtually identical and the fourth curve actually lies only slightly below the others. It can therefore be concluded that the triaxial test results are reproducible. Moreover, a unique normalized stress-strain curve could represent all the compressive specimens compacted at the same density. However, for dilative specimens at various consolidation pressures, Figure 6.17 shows that the normalization of stress-strain results does not produce a unique relationship (see also Atkinson and Bransby 1978). Here, the normalized stress-strain curves for dilative specimens in Series (III) produce substantially different normalized strength values q / p'_{cons} . As expected, the peak q / p'_{cons} decreases with increasing consolidation pressure along the overconsolidated envelope. Constitutive modelling of the stress-strain behaviour will be discussed in more detail in Chapter Seven.

6.3.2 Drained Shear

Figures 5.34 and 5.35 show typical axial strains ϵ_1 and axial strain rates $\dot{\epsilon}_1$ versus $\log(\text{time})$ data for two drained stress controlled specimens T910, T931 tested with p' constant and q increased incrementally until failure occurred. The axial strains increase slowly (and at decreasing rate) with $\log(\text{time})$. They also increase with deviator stress level until failure is finally initiated at q_{max} .

"Failure" is defined as the onset of the increase in axial strain rates with log (time) as shown in Figure 5.34 (Vaid and Campanella, 1978).

Figures 5.31 and 5.32 show normalized values of $q/q_{\max}$ versus accumulated shear strains one day, three days and five days after loading for the test data in Figures 5.29 and 5.30. Shear strains were calculated from overall measurements of specimen height and volume using the relationships $\epsilon = 2(\epsilon_1 - \epsilon_3)/3 = (\epsilon_1 - v/3)$ (Wood 1984). Results from the 3-day results from all the drained tests performed in this program are combined in Figure 6.18. The stress-strain data lie within a fairly narrow band prior to failure, and as would be expected, they are markedly non-linear in shear. To generalize the shear stress-shear strain relationship, the deviator stress may be normalised with respect to consolidation pressure p'_{cons} (rather than $q_{\max}$) as shown in 6.19. There is some scatter in data partly due to some difficulties in establishing the origin of the shear strain measurements upon the application of the first deviator stress increment (Head 1980). Nevertheless, the stress-strain behaviour of the buffer prior to failure can be represented by a hyperbolic function as follows

$$q / p'_{\text{cons}} = [\epsilon / (2\epsilon + 0.0125)] \quad [6.10]$$

The R^2 value of 0.78 indicates fairly good correlation. The tangent shear stiffness of the buffer derived from the above equation can be expressed as follows:

$$G_t / p'_{\text{cons}} = [1 / (31\epsilon + 0.193)^2] \quad [6.11]$$

Using this equation, the values of normalized tangent shear modulus at zero strain, $G_{ti} / p'_{\text{cons}}$, and the normalized secant shear modulus at $q_{\max}/2$, $G_{t50} / p'_{\text{cons}}$, are found to be about 27 and 13 respectively. These values are average stiffness values in the testing pressure range

up to 3 MPa.

Alternatively, the shear behaviour of the buffer can be approximately characterized by a secant shear modulus G_{s50} between the beginning of shear $q = 0$ and $q = q_{\max}/2$. Table 6.1 presented values of the normalized modulus G_{s50}/p'_{cons} for all the drained specimens tested in Series (I) of this program. (Note that in isotropic consolidation $p'_{\text{cons}} = \sigma_{\text{cons}}$). The relationship between 1-day, 2-day and 3-day values of G_{s50}/p'_{cons} and the consolidation pressure p'_{cons} is shown in Figure 6.20. It is apparent that the normalized shear modulus G_{s50}/p'_{cons} is both time-dependent and pressure-dependent. The pattern of behaviour in Figure 6.20 is rather complicated and it has been simplified in Figure 6.21 to show just the 3-day values of G_{s50}/p'_{cons} versus consolidation pressure. The normalized modulus values tend to decrease slowly with increasing confining pressure, a result also observed by Domaschuk and Valliapan (1975). Once again it is appreciated that this is a simple (perhaps simplistic) way of looking at the complex non-linear behaviour of the material and further work will be presented in Chapter seven towards forming a better constitutive model.

6.4 Synthesis of Triaxial Data

Section 4.3 showed that careful attention to water content determinations and volume change measurements has permitted the "state" of the specimens (in p', q, V_c -space) to be determined at each stage of testing. In this work, the clay specific volumes at any stage have been calculated backwards from the end-of-test water contents, taking account of volume changes measured during shear and consolidation. In

particular, calculation of the clay specific volumes V_c permits plotting of the "end of consolidation", "peak shear failure" and "end of shear" into the $\ln(p')$ vs V_c diagram shown in Figure 6.22. The symbols in the Figure are defined as follows:

- First letter: C = End of consolidation, P = Peak, E = End of shear
- Second letter: C = Compressive, D = Dilative.

Compressive specimens produced porewater pressures which were increasing at failure, while dilative specimens such as T932 (Figures 5.17 and 5.25) produced porewater pressures that were decreasing at failure. That is, they showed a tendency to dilate. The CC and CD data in Figure 6.22 correspond to the End of Consolidation data that were shown previously in Figure 6.7.

The "peak" and "end of shear" test results in q, p' -plane are also shown in Figures 5.35 and 5.33 respectively. The peak strength of the buffer from mostly low density specimens in Figure 5.35 can be described by:

- $c' = 46 \text{ kPa}$, $\phi' = 14^\circ$ from dilative or expansive specimens, and
- $c' = 0 \text{ kPa}$; $\phi' = 14^\circ$ from compressive specimens up to 3 MPa

It is appreciated that this Figure is an oversimplified approach compared with the behaviour postulated in Section 5.5 which differentiates between high density and low density specimens. The Figure again points to the relatively small changes in strength that accompany quite marked changes in the ductility and porewater pressure generation in the buffer.

Specimens which are clearly compressive by nature show little difference between the "peak" and "end of test" failure values in Figures 5.26 and 5.27. It was shown earlier when reviewing the

undrained stress-strain results in Figures 5.16 - 5.21 that the deviator stresses and porewater pressures were not changing significantly at the end of the compressive tests and therefore the "states" of the specimens would not deviate very much from the "critical states" for the specimens. Therefore, the compressive data will be more reliable than the dilative data for the determination of the Critical State Line, CSL.

The "end-of-test" p', q -stresses for all the buffer specimens tested at the University of Manitoba (Sun 1986; Wan 1987) have been summarized in Figure 6.23. On the basis of their clear departure from the general pattern of behaviour shown in Figures 5.26 and 5.27, specimens T913 and T922 have been excluded from the Figure. The data in Figure 6.23 have been notionally divided into two groups. One, shown as open crosses, represents dilative specimens. The second group, shown as open squares, represents compressive specimens. With the reduced amount of data presented in this Figure compared with Figure 6.22 it is easier to see that specimens tested at higher pressures generally tend to be compressive while specimens at lower stresses tend to be dilative.

One other feature of the compressive / dilative nature of the buffer has been observed in Series (II) and Series (III) of this program. Care was taken to measure water contents at the top and bottom of the specimens immediately after shearing was stopped. The results in Table 6.4 indicate that differences in water content between the top and bottom of the specimen correlate well with behaviour which is either "dilative" or "compressive" in the sense described in previous paragraphs. Lower water contents (up to 2%) at the top of the

specimens were associated with dilative behaviour: higher water contents (up to 2%) with compressive behaviour. Similar results were observed by Wan (1987). The magnitude of water content gradient along the specimens at the end-of-test in Table 6.4 is affected by consolidation duration and speed of shearing.

An alternative way of looking at the end of the test data is shown in Figure 6.24 where the high density specimens are separated from the low density specimens. It is apparent that the high density specimens at the end of shear have some tendency to lie lower in V_c -values than the low density specimens. This trend originates from the differences in "state" of low density and high density specimens at the end of consolidation shown in Figure 6.9. The differences in both Figures 6.24 and 6.9 could be attributed to the variations in microstructural fabric of the compacted buffer as discussed in Section 6.2.3. However, large shearing deformations following consolidation appears to have narrowed the gap between the high and low density envelopes substantially by altering the microstructure of the buffer. The best fit line through all the data in Figure 6.24 produces the following equation for CSL :

$$\text{LN}(V_c) = 1.719 - 0.118 \text{ LN}(p') \quad R^2 = 0.951 \quad [6.12]$$

while the high density specimens and low density specimens separately define the following CSLs:

$$\text{LN}(V_c) = 1.763 - 0.126 \text{ LN}(p') \quad R^2 = 0.969 \quad [6.13]$$

$$\text{LN}(V_c) = 1.577 - 0.097 \text{ LN}(p') \quad R^2 = 0.940 \quad [6.14]$$

The discrepancy between the two lines CSLs is smaller than the difference between the corresponding swelling equilibrium lines in Eqs. [6.5] and [6.6].

Critical State Soil Mechanics and the conceptual framework outlined in Section 2.4 suggest that after peak failure, dilative specimens should continue to move plastically towards the CSL with decreasing porewater pressures and increasing p' -values. The use of "whole specimen" instrumentation makes it impossible to follow localized behaviour in shear bands or bifurcation zones through from peak failure to the CSL (Graham et al. 1986b; Vardoulakis 1985).

The data in Figures 6.11 and 6.24 can be correlated as separate $\ln(p')$, $\ln(V_c)$ - functional relationships for the end- of- consolidation (NCL) and end- of- test (CSL) conditions. Two different equations [6.5] and [6.6] are required to describe the NCL for low density and high density specimens separately as discussed in Section 6.2.3. Similarly Eqs. [6.13] and [6.14] describe CSLs for high density and low density buffer respectively. Comparison of Eqs. [6.6] and [6.13] indicates that the NCL from low density specimens is almost parallel to the CSL as shown in Figure 6.25.

The Critical State Lines (CSLs) for low and high density specimens has been plotted into Figure 6.24. As expected from the dilative specimens, their peak failure points tend to lie above the CSL in Figure 6.23 and on the "dense" (or "dry") side of the CSL in Figure 6.24. There is some tendency for them to be different for the two different test durations in the program, with the short durations tests lying below (on the dense side of) the longer duration tests (Wan, 1987).

The conclusion from this section is that functional relationships have now been established for the p', V_c - and p', q -behaviour of the buffer. Consolidation and failure "states" of the

soil can now be examined in p', q, V_c -space as shown in Figures 6.25(a) and 6.25(b) for low and high density specimens separately. Figure 6.25 presents the test data in a three dimensional form similar to that used in critical state soil mechanics and allows comparison with the conceptual model discussed in Section 2.4.

CHAPTER SEVEN

CONSTITUTIVE MODELLING OF THE BUFFER BEHAVIOUR

The mechanical behaviour of the buffer is complex. It has been shown in Chapter 6 that the stress-strain-time behaviour of the material is influenced by many factors such as soil fabric structure, loading rate, loading history, and current stress state. Here, soil fabric structure refers to "microvariables" such as mineralogy, grain characteristics, surface texture, platelets orientation, cementation or bonding (Baladi, 1981). All these microvariables play an interrelated role in the complex mechanical response of the buffer. For practical purposes, however, these microvariables can not generally be individually quantified; rather "engineering" variables such as specific volume and relative density, which are more amenable to measurement, have been used to help characterize, understand, and now to predict the buffer behaviour.

This chapter develops simple, yet realistic, time-independent constitutive relations capable of modelling the mechanical behaviour of the buffer. Three different approaches namely, hyperelastic, hypoelastic and elastic-plastic models are used, involving various degrees of numerical complexity. Special attention has been paid to modelling the anisotropic properties of the buffer material in the latter models. The simple isotropic elastic-plastic model of the Cam Clay family has been implemented for finite element analysis in chapter 8.

7.1 Hyperelastic Formulation

The background to hyperelastic (non-linear elastic) formulation was discussed in Section 2.4.3. The experimental data discussed in Chapter 6 are used here to determine the parameters necessary for calibration of two idealized hyperelastic models. The first one is a rather simplistic isotropic psuedo-elastic model. The second model is an anisotropic elastic model reflecting the inherently anisotropic nature of the buffer material. Both of these models lead to simple constitutive relationships of the buffer behaviour appropriate for the small strain elastic range.

7.1.1 Isotropic Psuedo-Elastic Model

The simplest isotropic constitutive model that can be used to reasonably approximate the behaviour of the buffer in the early stages of loading is a psuedo-elastic model. Here, the long term secant bulk stiffness K_{sec} and shear stiffness G_{sec} are assumed to be constant at a given consolidation pressure. Therefore, K_{sec} and G_{sec} of the buffer at equilibrium are both assumed to be functions of consolidation pressure only as shown in Figure 7.1. Based on the experimental results presented in Sections 6.2 and 6.3, constitutive relationships for the buffer (in terms of stress and strain invariants) can be expressed simply as follows :

$$v = (p' - p'_{cons}) / K_{sec} \quad [7.1a]$$

$$\epsilon = q / 3G_{sec} \quad [7.1b]$$

where K_{sec} (Figure 7.1) and G_{sec} (Figure 6.21) for the low density buffer at pressures upto 3 MPa are

$$K_{sec} = 8.2 p'_{cons} \quad [7.2]$$

$$G_{sec} = [p'_{cons} / (0.041 + 0.034 p'_{cons})] \quad [7.3]$$

Similarly, for the high density buffer

$$K_{sec} = 33.9 p'_{cons} \quad (\text{Figure 6.5}) \quad [7.4]$$

$$G_{sec} = 16.4 p'_{cons} \quad (\text{Figure 5.37}) \quad [7.5]$$

7.1.2 Anisotropic Psuedo-Elastic Model

The results of the experimental program in Section 5.3.2 indicated clearly that the behaviour of the compacted buffer is anisotropic in nature with a horizontal stiffness twice as large as the vertical stiffness. Graham and Houlsby (1983) developed a three-modulus anisotropic elastic model for cross-anisotropic soils as described in Section 2.4.3. The strain and stress invariants can be related through the compliance matrix

$$\begin{bmatrix} \delta v \\ \delta e \end{bmatrix} = \begin{bmatrix} C_1 & C_2 \\ C_2 & C_3 \end{bmatrix} \begin{bmatrix} \delta p' \\ \delta q \end{bmatrix} \quad [7.6]$$

where $C_1 = 3G^* / \text{Det}$, $C_2 = -J / \text{Det}$, $C_3 = K^* / \text{Det}$ and $\text{Det} = (K^* G^* - J^2)$, the determinant of the stiffness matrix. In contrast with the hypoelastic modelling shown by [2.7], the moduli K , G and J are

here assumed to be constant in the elastic range. In any single triaxial test, measurements of $\delta\sigma'_1$, $\delta\sigma'_3$, $\delta\epsilon_1$ and $\delta\epsilon_3$ are generally recorded. These measurements are conveniently converted to stress invariants $\delta p'$ and δq , and strain invariants δv and $\delta\epsilon$. At least two tests with different $\delta q/\delta p'$ ratios are required to solve for the three moduli. A larger data base, such as the one compiled for the buffer, leads to mathematical redundancy. A common approach in solving a large set of equations for the required parameters (K^* , G^* and J) is to use a "least squares" method to minimize random errors.

Treating the stresses as independent parameters (Graham and Houlsby 1983), the sum of the squares of the errors in the strains for all the tests available is given by

$$e = \sum_{\text{Tests}} (C_1\delta p' + C_2\delta q - \delta v)^2 + (C_2\delta p' + C_3\delta q - \delta\epsilon)^2 \quad [7.6]$$

The least square solution for the parameters C_1 , C_2 and C_3 from the set of redundant equations is found by setting the differentials of the error measure e with respect to each of the parameters C_1 , C_2 and C_3 in turn to zero. This results in three equations and three unknowns. Using this procedure the normalized moduli with respect to consolidation pressure for the low density buffer is obtained as follows :

$$K^* / p'_{\text{cons}} = 16.7 \quad [7.7]$$

$$G^* / p'_{\text{cons}} = 13.1 \quad [7.8]$$

$$J / p'_{\text{cons}} = -12 \quad [7.9]$$

These results are based on early stages of both undrained shear and drained constant- p' tests at pressures up to 3 MPa as shown in Figure 7.2. The validity of hyperelastic models calibrated here in the small strain elastic range will be examined in comparison to hypoelastic models and experimental results later.

7.2 Hypoelastic Models

Sophisticated models requiring a large number of stress level dependent parameters have been developed from the general framework of hypoelasticity in [2.6] by a number of researchers. Many of the parameters are difficult, sometimes impossible, to obtain by conventional testing. More practical hypoelastic models involve simplifying assumptions to reduce the number of necessary parameters. Examples of such models are the E, ν model of Duncan and Chang (1970); the K, G model of Domaschuk and Valliappan (1975); the three modulus model of Yin and Yuan (1985a,b); and most recently the new three modulus model by Yin et al. (1988) outlined in Section 2.4.4.2.

Laboratory test data from isotropic consolidation tests and consolidated undrained tests with porewater pressure measurement presented in Chapters 5 and 6 are used here to form two practical constitutive hypoelastic models for the buffer. The first model is a simpler two modulus (K, G) model which as in the hyperelastic case, involves an assumption of isotropy. The second one is a three modulus (K, G, J) model that incorporates the influence of anisotropy of the buffer. This new hypoelastic model has been developed at the University of Manitoba in collaboration with the buffer research program discussed in Section 4.3. The model uses three stress dependent modulus functions referred to as "moduli". These are (1) the bulk modulus K , (2) the shear modulus G , and (3) a coupling modulus J .

(p', v) , (q, ε) , (p', ε) and (q, v) relationships for the buffer. The calibrated models are then used to make predictions for drained tests to be compared with experimentally measured values for validation.

7.2.1 Two Modulus Hypoelastic Model

The two modulus hypoelastic model is well known as a simple yet realistic form of constitutive relationship for soils. This model has been used in finite element analysis to predict foundation settlements (Domaschuk and Valliapan 1975). It decouples the "volumetric" and "shear" components of the deformation and uses two stress dependent modulus functions referred to as "moduli". These are (1) the bulk modulus K , and (2) the shear modulus G . The model assumes isotropic behaviour and no shear dilatancy.. The two modulus (K, G) model is a special case of the more general three modulus (K, G, J) discussed in Section 2.4.4.2. The K, G model will be calibrated using; (1) isotropic consolidation data in Section 7.2.2.1.1; and (2) drained constant- p' data in Section 6.3.2. Comparisons will also be made between the predictions of the two models and the measured buffer behaviour in Section 7.3.2.2.

7.2.2 Three Modulus Hypoelastic Model

A three modulus hypoelastic model is here proposed to represent the anisotropic nature of the compacted buffer. General formulation of the model has been discussed in Section 2.4.3. The experimental data discussed in Chapter 6 will be used to calibrate the model for both the high density and low density buffer. The calibrated model is then used to predict the behaviour of the buffer along other stress paths. Additional test data will then be used for

comparison to examine the validity of the model.

7.2.2.1 Evaluation of Hypoelastic K, G, J Moduli

For a model to be useful, the required tests must be relatively simple to perform and the data should be consistently reliable. For example, Duncan and Chang (1970) used CID data to find E and ν as functions of q and σ'_3 , while Domaschuk and Valliapan (1975) used isotropic consolidation to find K , and constant- p' tests to find G as a function of q and p' .

Conventional CID tests or incremental constant p' tests require long test durations, and in the latter case, are relatively difficult to run. When a load increment is applied, the porewater pressure inside the specimen changes, and the real effective stress path deviates from the target one. Despite requiring additional instrumentation, $CI\bar{U}$ tests are in some ways easier to run and more reliable than CID tests. No water flows into or away from the specimen, and apart from some end effects or localized effects in expansive, strain-softening specimens, effective stresses can often be considered substantially constant throughout the specimens. In this case the specimen can be treated as an infinitesimal element for finding differential relationships for stress-strain behaviour.

The proposed model by Yin et al. (1988) uses a new method for determining K, G and J . The framework of this model was developed by J.H. Yin as part of his doctoral program at the University of Manitoba. Isotropic consolidation is used to find the bulk modulus K , while undrained $CI\bar{U}$ tests are used to find the shear modulus G and the coupling modulus J . Unload-reload cycles are incorporated into these tests to determine the elastic moduli K^e , G^e and J^e . The method will

be described using the results in Figures 6.11 and 7.3 - 7.4 from triaxial testing on the buffer specimens.

It was shown in Chapter 5 that the behaviour of the buffer with different initial densities can be broadly divided into two groups of "low density" and "high density" buffer. The "high density" buffer is compacted at densities more than 90% ASTM Modified Density.

7.2.2.1.1 Mean Pressure-Volume Change Relationship for K and K^e

Isotropic consolidation provides data from low density specimens relating effective mean stresses p' and volumetric strains v (Figures 6.3 and 6.11). Fitting appropriate mathematical functions to these data produces equations $v = f_1(p')$ for loading and $v = f_1^e(p')$ for unloading-reloading. The bulk modulus K is then found from the first equation in [2.7] with $dq = 0$:

$$K = \frac{dp'}{dv} \quad [7.11]$$

and K^e is obtained in a similar way from unload/reload data. The K and K^e moduli are functions only of p' .

The behaviour of sand-clay mixtures has been shown earlier to be related to clay specific volume V_c , the volume of clay and water occupied by unit volume of clay solids. The swelling/compression equilibrium line (SEL or NCL) in Figure 6.3 can be represented (see Figure 6.11), by:

$$V_c = -\lambda_c \ln(p'_{\text{cons}}) + V_{c0} \quad [7.12]$$

The relation between V_c and V for the buffer material is $V = 0.491V_c + 0.509$ (Graham et al. 1988a) and this produces from [7.12]:

$$V = -\lambda \ln(p'_{\text{cons}}) + V_o \quad [7.13]$$

and the relationship between ϵ_v and p' :

$$v = \lambda/V_i \ln(p'_{\text{cons}}) + v_o \quad [7.14]$$

where $V_i = 1 + e_o$, the initial specific volume before straining.

Figure 6.3 was drawn using strains accumulated in the first three days of each increment (Mesri and Godlewski 1977; Graham et al. 1983). The resulting parameters in [7.12], [7.13] and [7.14] are shown in Table 7.1. The equations show that low density specimens will swell after three days when p' is less than 0.68 MPa, and compress under higher confining pressures. The long-term swelling characteristic of high density buffer is inhibited under confining pressures exceeding 2.13 MPa.

Table 7.1 - Parameters in Eqns. [7],[8],[9]

Characteristics & Behaviour	Test Duration	λ_c	V_{co}	λ	$\frac{\lambda}{V_i}$	V_o	v	R^2
Low Density Compressive	3 days	0.139	2.487	0.068	0.0385	1.730	0.0148	0.97
Low Density Compressive	EOC	0.261	2.530	0.128	0.0729	1.751	0.0028	0.92
High Density Compressive	EOC	0.171	2.360	0.084	0.052	1.668	-0.039	0.95

The behaviour of buffer is strongly time-dependent (Graham et al. 1988a and Section 6.2.1) - test durations up to 60 days were required to reduce the volume strain rates to 0.1% /day. Figure 6.11 shows "end-of-consolidation" (EOC) relationships rather than the "3-day" results shown in Figure 6.3 (The dashed line [7.12] in Figure 6.11 was drawn through data from low density specimens that were compressive (+ve porewater pressure changes) during the subsequent undrained shear phase of the test). Table 7.1 also shows the EOC parameters resulting from this data in [7.12], [7.13] and [7.14]. The resulting compression index $\lambda = 0.128$ is larger than the corresponding value from "3-day"

data, and suggests the specimens were not fully equilibrated at the earlier stage (Graham et al. 1988b). The long-term swelling pressure of the buffer at 85% ASTM modified density is estimated at 0.95 MPa. This value increases to 2.13 MPa for the buffer compacted at 95% ASTM modified density.

The elastic bulk compressibility of the buffer is determined through an isotropic swell-compression cycle represented mathematically by:

$$v = \kappa/V_i \ln(p'_{\text{cons}}) + v_o^e \quad [7.15]$$

The data show $\kappa/V_i = 0.0105$. ($R^2 = 0.95$) and $\kappa/V_i = 0.0125$ for 3-day and 5-day strains respectively.

Applying [2.7] and differentiating [7.14] and [7.15], the bulk moduli K and K^e are:

$$\begin{cases} K = p' / (\lambda/V) \\ K^e = p' / (\kappa/V) \end{cases} \quad [7.16]$$

The introduction of p' in [7.16] instead of p'_{cons} in [7.12] and [7.15] involves an important assumption that the volumetric strain induced by p' is independent of q . Using the "end-of-consolidation" data in Table 7.1 produces $K/p' = 13.7$ for low density specimens and $K/p' = 19.2$ for high density specimens. The 5-day unload/reload data suggest in [10] that $K^e/p' = 80.0$ for low density specimens and $K^e/p' = 73$ for high density specimens, ($K/K^e = 5.8$ and 3.8 respectively).

7.2.2.1.2 Effective Mean Stress-Deviator Stress Relationship for J and J^e

In CIU tests the total confining stress σ_3 is constant and the

total vertical stress σ_1 increases ($dq > 0$). Mean effective stresses increase in specimens that are expansive in shear ($dp' > 0$), and decrease when the specimens are compressive ($dp' < 0$). Two independent relationships, q vs. p' (Figure 7.3) and q vs. ϵ (Figure 7.4) can be measured directly. The first is used to determine the coupling modulus J , and the second to evaluate the shear modulus G .

In undrained shear, the volumetric strain is constant, $dv = 0$. So from the first equation in [2.7]:

$$\frac{J}{K} = \frac{dq}{d(-p')} \quad [7.17]$$

where $-p' = u - p$; K has already been determined from isotropic consolidation tests; and u , p and q are measured during the test. Eq. [7.17] means that J/K can be obtained by differentiating a curve fitted to q vs. $(-p')$ test data such as those shown in Figure 7.3. In this form, [7.17] is valid for expansive specimens in which p' is increasing. In compressive specimens p' is decreasing ($A \geq 1/3$), so K in [7.17] should be replaced by K^e . An unload-reload cycle in the CI $\bar{U}$ test allows J^e to be determined using similar procedures.

Stress paths for compressive CI $\bar{U}$ specimens in Figure 7.3 have been normalized by p'_{cons} . The stress paths incline leftwards at the beginning of shearing indicating anisotropy. A power law [7.18] has been used to describe the best-fit normalized stress path:

$$q/p'_{\text{cons}} = A(1 - \frac{p'}{p'_{\text{cons}}})^n \quad [7.18]$$

A and n parameters are given in Table 7.2 for the low density buffer and the high density buffer (both compressive and dilative specimens separately). The R^2 coefficients given in the same Table reflect some variability of the normalized stress paths due to small differences in

test variables such as initial density, fabric structure and consolidation duration.

Using [7.17] and differentiating [7.18] with respect to p' , the coupling modulus can be written:

$$\begin{aligned} J/K^e &= nA^{1/n} (q/p'_{\text{cons}})^{(n-1)/n} \\ &= c (p'_{\text{cons}}/q)^d \end{aligned} \quad [7.19]$$

where $c = nA^{1/n}$, $d = (1-n)/n$. These parameters are also given in Table 7.2.

Characteristics & Behaviour	A	n	c	d	R^2
Low Density Compressive	1.32	0.71	1.05	0.41	0.81
High Density Compressive	1.25	0.65	0.91	0.54	0.64
High Density Dilative	3.37	1.00	3.37	0	0.61

Using the values of K^e/p'_{cons} obtained in the previous section from [7.16], the coupling modulus of the buffer for various cases can be expressed as follows:

$$\begin{aligned} J &= 1.05 \times 80p' (0.962 \exp(13.72v)/q)^{0.41} && \text{(L.D. Comp.)} \\ J &= 0.91 \times 73p' (2.13 \exp(19.23v)/q)^{0.54} && \text{(H.D. Comp.)} \\ J &= 3.37 \times 73p' && \text{(H.D. Dil.)} \end{aligned} \quad [7.20]$$

It can be seen that J and J^e are functions of p' and q , and vary with the volumetric strain v from the beginning of the test due to consolidation under p'_{cons} . When $q = 0$, the coupling modulus $J = \infty$, implying that the initial behaviour in shear is isotropic. This

deviates from the observed behaviour in Figure 7.3 which is initially anisotropic but is considered a second order approximation in the modelling.

The porewater pressure generated during undrained shear for compressive specimens can be expressed in terms of deviator stress using [7.18] as follows:

$$\delta u = q / 3 + (1/A)^{1/n} q^{1/n} p'_{\text{cons}}^{(1-1/n)} \quad [7.21]$$

This expression can then be used to define the porewater pressure parameter (m) in term of deviator stress as follows:

$$m = 1 + 3 (1/A)^{1/n} (q / p'_{\text{cons}})^{(1/n - 1)} \quad [7.22]$$

7.2.2.1.3 Undrained Shear Stress-Strain Relationship for G and G^e

Measured $q - \epsilon$ relationships such as those in Figure 7.4 can then be used to determine G and G^e . Using [7.17] with the second equation in [2.7], we have:

$$3D = \frac{dq}{d\epsilon} = \frac{3GJ^2}{J^2 - 3GK} \quad [7.23]$$

where $3D$ is introduced for convenience to denote the local slope of q vs. ϵ , called the equivalent shear modulus. From [16]:

$$G = \frac{DJ^2}{J^2 + 3DK} \quad [7.24]$$

If the soil is neither expanding nor compressing in shear, then $J = \infty$, and the shear modulus G equals the apparent shear modulus D . Again, G is a function of p' and q , and varies with the total ϵ_v produced by p'_{cons} .

In a general case, K , K^e , J and J^e have already been determined at this stage. (Figure 7.3 showed only first-loading compressive data, so only J and not J^e has been evaluated in this program). If q, ϵ data such as those in Figure 7.4 are fitted with an appropriate mathematical function, this can be differentiated to provide D . The

shear modulus G is then obtained from [7.24]. As in previous cases, unloading-reloading CI $\bar{U}$ data allow determination of the elastic shear moduli D^e and G^e .

Figure 7.4 shows reasonably consistent q - ϵ results from six compressive specimens with effective confining pressures up to 3 MPa. Peak shear stresses are generally reached at about 4% shear strain and the shearing resistance remains essentially unchanged with further straining. The shear stress-strain relationship has been modelled using a best-fit hyperbolic function (Duncan and Chang 1970, Domaschuk and Valliappan 1975):

$$q/p'_{\text{cons}} = \frac{\epsilon}{a + b\epsilon} \quad [7.25]$$

with a and b parameters given in Table 7.3.

Characteristics & Behaviour	a	b	R^2	$\frac{D^e}{p'_{\text{cons}}}$
Low Density Compressive	0.0086	2.15	0.96	37.5
High Density Compressive	0.01076	1.832	0.99	25.0
High Density Dilative	0.0106	1.104	0.84	25.0

Using [7.23] and differentiating [7.25] results

$$3D = \frac{p'_{\text{cons}}}{a} \left(1 - b \frac{q}{p'_{\text{cons}}}\right)^2 \quad [7.26]$$

Then replace p'_{cons} with ϵ from [7.14]:

$$3D = 111.86 \exp(13.72v) [1 - 2.235q \exp(-13.72v)]^2 \quad [7.27]$$

which shows D to be a function of q and the strains v produced by p'_{cons} . The parameter D represents an equivalent shear modulus G_{eq} (Graham and Houlsby 1983) for an idealized isotropic elastic material or for a two modulus (K , G) hypoelastic model in Section 7.3.1. Using

$K^e = 80p'$ from [7.16], J obtained from [7.20], and D obtained from [7.27], the shear modulus G can be found from [7.24] as a function of p' , q and v .

The elastic parameter D^e has been measured as $D^e/p'_{\text{cons}} = 25$ from the results of unload-reload cycles in an undrained shear test. It is assumed here that the unload-reload behaviour is isotropic elastic, so, from [7.24], $G^e = D^e$.

7.2.2.2 Prediction and Comparison

The three-modulus model was calibrated in the previous section using isotropic consolidation and undrained CI $\bar{U}$ test data. Now it will be used to predict the results of drained tests with constant p' . The predicted results will be checked against experimentally measured values. The required condition is $dp' = 0$. From [2.7], [7.19] and [7.16]:

$$\frac{p'}{\kappa/V_i} c \left[\exp(-v_o \frac{\lambda}{V_i}) \exp(v \frac{\lambda}{V_i}) / q \right]^d dv = dq \quad [7.28]$$

which on integration and combination with [9] gives:

$$v = \frac{\lambda}{V_i} \ln(p') + v_o + \frac{\lambda}{V_i d} \ln \left[1 + \frac{\kappa d}{(1+d)\lambda c} \left(\frac{q}{p'} \right)^{1+d} \right] \quad [7.29]$$

The volumetric strain produced by shearing is then:

$$\Delta v = \frac{\lambda}{V_i d} \ln \left[1 + \frac{\kappa d}{(1+d)\lambda c} \left(\frac{q}{p'} \right)^{1+d} \right] \quad [7.30]$$

which is a function of $\eta = (q/p')$. Substituting appropriate values for buffer parameters in [7.30]:

$$\Delta v = 0.178 \ln \left[1 + 0.0475(q/p')^{1.41} \right] \quad [7.31]$$

Figure 7.5 shows the calculated relationship resulting from [7.31] between q/p' and Δv , and also experimentally measured relationships from four constant- p' tests at different stress levels.

Although there is some scatter in the data, the prediction looks promising considering the small strains that are typically encountered in these tests and the relative insensitivity of volume change measurements.

It is also possible to predict q vs. ϵ relationships in constant- p' tests. From the second equation in [2.7] with $dp' = 0$:

$$dq = 3G(p', q, v) d\epsilon \quad [7.32]$$

where G is expressed in terms of D and J from [7.27] and [7.20] with $K^e = 80p'$. An analytical solution can not be obtained for Eqn. [7.32] and it has been solved with the fourth-order Runge-Kutta method in conjunction with [7.29].

For special isotropic case (K, G model) a closed form solution may be obtained because the constitutive equations are not coupled. Using equation [6.10] for low density buffer the solution can be found as follows:

$$dq = 3G(q, v) d\epsilon \quad [7.33a]$$

$$q = \int [0.0125/(2\epsilon + 0.0125)^2] d\epsilon \quad [7.33b]$$

Figure 7.6 compares predicted behaviour by the two models and measured results for q/p' vs. ϵ . The predicted normalized curves for different constant- p' values are unique and agree well with the test data, especially when it is remembered that the tests were performed by incremental loading and are known to be difficult to interpret. As expected, the agreement between measured and predicted response is better for the three modulus model which accounts for the anisotropy of the buffer.

7.2.2.3 Discussion and Conclusions

Hypoelastic modelling permits examination of the deformability of the buffer along different stress paths. This is done using the q/p'_{cons} vs. ε relationship in [7.25] and the q/p'_{cons} vs. p'/p'_{cons} relationship in [7.18]. Three different stress paths have been examined here (1) $\text{CI}\bar{\text{U}}$ with $dp'/dq < 0$. (2) constant- p' with $dp'/dq = 0$, and (3) CID with $dp'/dq = 1/3$. The stress ratios $\eta = q/p'$ for these three stress paths are shown in Table 7.4 for $\varepsilon = 4\%$ and 5% .

Table 7.4 - Values of $\eta = q/p'$ at $\varepsilon = 4\%$ and 5% along Different Stress Paths

Test Types	:	$\text{CI}\bar{\text{U}}$	Const. p'	CID
dp'/dq	:	< 0	0	0.333
$\eta = q/p'$ at $\varepsilon = 4\%$	:	0.440	0.435	0.429
$\eta = q/p'$ at $\varepsilon = 5\%$	:	0.448	0.445	0.440

These results show that the shear behaviour of the buffer becomes slightly more deformable as the slope of the stress path dp'/dq increases. The differences are more marked if expressed in terms of axial strains ε_1 rather than the shear strains ε in Table 7.4.

Work on this K,G,J model began when the experimental program was already well under way. Thus although the model can be calibrated from Figures 6.11, 7.3 and 7.4 with some certainty, its validation has had to rely only on stress paths with constant p' (Figures 7.5 and 7.6). Other graduate students are now undertaking further testing to provide more complete validation.

In terms of constructing the model, the principal uncertainties at this stage involve the assumption in [7.16] that K and K^e are

independent of $\eta = q/p'$, and the relationship between J and J^e .

The stress states used for calibrating and verifying the model have only been triaxial (axisymmetric). A more general model needs to be developed and verified for general stress states that incorporates the influence of time (Truesdell 196). Work on time-dependent constitutive modelling of behaviour of clays is currently in progress by another graduate student at the University of Manitoba. Recently, Yin and Graham (1989) developed a general 1-D viscous elastic plastic model to describe the time-dependent behaviour of clays. This model is now being extended to a general 3-D stress conditions (Yin 1989).

For practical purposes, a simple hypoelastic model was presented in this section which accounts for irreversible, nonlinear, shear-compressive or shear-expansive characteristics of soil behaviour, and the shear strains which can accompany changes in mean effective stress. The physical meaning of the moduli is clear, the mathematical structure of the model is not complicated, and the method for determining the parameters is relatively straightforward.

7.3 Elastic-plastic Modelling

The stress-strain behaviour of the buffer may (in principle) be separated into recoverable (elastic) and irrecoverable (plastic) components. The recoverable behaviour is treated within the framework of elasticity theory; while the irrecoverable part is based on plasticity theory. Under monotonically increasing load elasticity-based models provide a much simpler approach. Two examples of application of hyperelasticity models to buffer will be presented in the subsequent sections. However, elastic-plastic modelling of the material behaviour is more useful when cyclic loading-unloading is encountered.

Elastic-plastic modelling is based on the principal assumption that deformations of the material are fully reversible within a particular yield locus. In general, the model accounts for three important characteristics of soil behaviour namely, non-reversibility, non-linearity and dilatancy. The development of a particular elastic-plastic model, namely the family of Cam clay models (Roscoe and Schofield, 1963; Roscoe and Burland, 1968) is based on a number of additional simplifying assumptions, as discussed in Section 2.4.2. These include isotropic behaviour in the elastic range, a constant elastic shear modulus, elliptical or bullet-shaped yield loci dependent on stress history, an associated flow rule, exponential elastic unload-reload and hardening laws.

Evidence of "elasticity" and hardening law for the buffer was presented in Chapters 5 and 6, particularly in Figures 5.18, 5.22-

5.25, 5.35- 5.37, 6.3 and 6.10. The assumption of isotropic behaviour in Cam clay models is not strictly valid for the buffer. Furthermore, little information is available on the shape of the yield loci and flow rule in the buffer. Nevertheless, experimental data collected on compressibility (loading and unloading in isotropic consolidation) and shearing behaviour (loading and unloading in triaxial shear) of the buffer can be used to calibrate a family of Cam clay models.

In fact the experimental data has already been expressed in a form suitable for elastic-plastic modelling and does not need to be presented again in detail. The volume change and shearing parameters for the low density and high density buffer can be summarized as follows :

	<u>Low Density</u>	<u>High Density</u>
λ =	0.128 (Figure 6.11)	0.084 (Figure 6.11)
k =	0.022 (Figure 6.6 (b))	0.022 (Figure 6.6 (b))
e_o =	0.775 (Figure 6.24)	0.668 (Figure 6.24)
M =	0.50 (Figure 5.38)	0.526 (Figure 5.39)
G^e / p'_{cons} =	37.5 (Figure 5.18)	36.0 (Figure 5.37)

The validity of these models will be examined in Section 8.2, where other triaxial test results are compared with Cam clay predictions. A major shortcoming of this model is the fact that it does not reflect the variation of shear modulus of the buffer with confining pressure reported in Section 6.3. Consequently, either an average shear modulus or a constant Poisson ratio should be used to calibrate the model. The

validity of the assumptions made in Cam clay models will be further examined in Section 8.2.

CHAPTER EIGHT

FINITE ELEMENT ANALYSIS OF ROCK-BUFFER-CONTAINER-BACKFILL PROBLEM

The containers for disposal of nuclear fuel waste should be designed to resist the groundwater pressure, the pressure from swelling of the buffer, any ambient stresses transferred through the buffer from the surrounding rock, and pressures arising from differential thermal expansion between the container, rock mass and buffer. The data presented by Dixon et al.(1986) suggest that once the buffer is saturated the swelling pressure of the buffer will be superimposed fully on the pressure in the groundwater. In addition, due to the compressibility of the buffer relative to the rock mass, the stresses within the rock should not be transferred as pressures to the waste container. Moreover, the pressures on the container arising from differential thermal expansion of the container, buffer and rock will be insignificant.

The buffer will be confined by the surrounding plutonic rock, the container and the vault backfill. Experimental evidence (Pusch and Borgesson 1985) has shown that there will be little clay lost from the buffer by extrusion into the rock fractures that intersect the emplacement boreholes. In addition, the backfill material can be expected to act as an effective filter medium, preventing loss of clay fraction from the buffer (Chamberlain and Yoder 1958). Therefore, apart from the influence of ground water chemistry, the magnitude of swelling pressure will be governed by the emplacement density of the

buffer and subsequent volume changes. The volume change of the buffer will depend on the temperature changes that occur in the vault and the compliance of its surrounding environment. The latter is a function of relative stiffnesses of the container, buffer, rock and backfill. The effect of temperature on hard natural clays from Italy and Belgium has been studied by Baldi et al. (1985 & 1986) and Hueckel et al. (1987). The results suggest that the swelling pressure of the clay decreases with increasing temperature. The effect of temperature on the mechanical properties and swelling pressure of the buffer is the subject of a follow up investigation at the University of Manitoba by another graduate student, and is therefore beyond the scope of this thesis. High temperature and high pressure triaxial testing facilities for this work have just been completed.

This chapter is a preliminary examination of the interactions between buffer, rock, container and backfill in the vault environment, and the resulting volume changes that are likely to occur in the buffer. The conceptual Critical State Model described in Chapter 6 and the elastic-plastic constitutive relationships developed in Chapter 7 will be used here to conduct a finite element analysis of the vault problem.

The chapter starts with the introduction of a CRITICAL State soil Program (CRISP) developed by Britto and Gunn (1987). Then, the question of representative modelling of the buffer behaviour will be examined critically. This is done by (1) comparing consolidation test results and Biot's coupled model predictions, and (2) comparing the triaxial shear test results with the predictions made by Cam clay family models. Finally, the analysis of the buffer- rock- container-

backfill problem under various conditions will be discussed in detail.

8.1 Introduction of CRITICAL State Program (CRISP)

The critical state model has been implemented in a finite element program known as CRISP (Britto and Gunn 1987) by researchers of the soil mechanics group at the Cambridge University starting in 1975. The program can be used in various geotechnical engineering applications, for example to predict ground movements associated with foundations or excavations. The program utilizes a non-linear incremental (tangent stiffness) approach and allows different types of analysis, soil models, element types and boundary conditions. Important facilities and features of the program can be summarized as follows :

(a) Types of analysis

Undrained, drained or fully-coupled (Biot) consolidation analysis of two-dimensional plane strain or axisymmetric (with axisymmetric loading) solid bodies.

(b) Soil models

Anisotropic elasticity, inhomogeneous elasticity (properties varying with depth), critical state soil models (Cam clay, modified Cam clay)

(c) Element types

Linear strain triangle and cubic strain triangle (with extra pore pressure degrees of freedom for a consolidation analysis)

(d) Boundary conditions

Element sides can be given prescribed incremental values of displacements or excess pore pressures. Loading applied as nodal loads

or pressure loading on element sides. Automatic calculation of loads simulating excavation or construction when elements are removed or added.

CRISP has been recently mounted on the University of Manitoba Mainframe Computer System (Mantes) with minor modifications. This was done to facilitate the analytical study of the buffer behaviour in the vault environment using the elastic-plastic model in the form of the Cam clay family developed in Chapter 7. The implemented program has been tested by comparing the results of analysis of several example problems to the published values by Britto and Gunn (1987). The accuracy of the results obtained has been found to be satisfactory.

8.2 Reproduction of the Laboratory Results by CRISP

CRISP is used in this section to examine the validity of the Cam clay family models for the buffer. This is done by analysing the triaxial test specimens under different stress conditions using CRISP, then comparing the numerical and experimental results obtained. By taking advantage of the bi-axial symmetry of the triaxial specimen, only a quarter of the specimen needs to be considered for analysis as shown in Figure 8.1. Third order cubic strain triaxial elements are used to obtain more accurate results. To improve the accuracy of the predictions a finer mesh is recommended. The specimen is subjected to different stress conditions; (1) isotropic consolidation; (2) undrained shear ; and (3) drained shear. The analytical results for each case are presented separately in the following subsections.

8.2.1 Comparison of Consolidation Test Results and Biot's Coupled Model

A true three-dimensional theory of consolidation couples the equilibrium of total stresses and the continuity of the soil mass (Lambe and Whitman 1969). A pseudo three-dimensional theory uncouples these two conditions, under the assumption that the total stresses are constant, so that the rate of change of excess pore pressure is equal to the rate of change of volume at all points in the soil. This condition is only strictly true in special cases. One-dimensional consolidation is one such case, where the increment of total stress is uniform and equal to the applied stress.

The general theory of three-dimensional consolidation of an elastic porous medium was introduced by Biot (1941). It is based on a number of simplifying assumptions; (1) isotropy of the material, (2) reversibility of stress-strain relations under final equilibrium conditions, (3) linearity of stress-strain relations, (4) small strains, (5) the water contained in the pores is incompressible, (6) the water may contain air bubbles, (7) the water flows through the porous skeleton according to Darcy's law. Biot proposed a solution for the fully coupled consolidation problem using the method of operational calculus. This solution has been implemented in CRISP for the cases of two-dimensional plane strain and axisymmetric (with axisymmetric loading). A more complete treatment is given by Rice and Cleary (1976).

Figures 8.2 and 8.3 compares the consolidation behaviour of two specimens T919 and T916 to the predicted volume strain-time relationships by Biot's theory using CRISP. The finite element mesh used is shown in Figure 8.1 which represents one quarter of the

specimen with a drainage boundary at the top. Thus, the analysis represents the double drainage condition for the whole specimen. The consolidation times for the double drainage case can be quadrupled to obtain the predicted consolidation curves for specimens with single drain only. However, side drains were used in tested specimens to decrease the consolidation times. The consolidation period for specimens with bottom and side drains is expected to be only 10% longer than specimens with double drains (Bishop and Henkel 1962). Applying 10% correction to the consolidation times for double drainage case the predictions for the tested specimens are obtained as shown in Figures 8.2 and 8.3. The agreement between the predicted and measured volumetric strains is promising considering the assumptions involved in the consolidation theory. The predicted long term volume strains tend to be lower than the measured values. The discrepancies could be attributed partly to creep straining which is not taken into account in Biot's theory.

8.2.2 Prediction of Shear Behaviour of the Buffer by Cam Clay Models

The buffer behaviour is examined both in undrained and drained shear. First, the stress-strain and pore pressure behaviour of the buffer in shear can be predicted using the Cam clay models which were calibrated in Section 7.1. Figure 8.4 compares the predicted response of the buffer in a strain controlled undrained shear test to the measured results for compressive specimen T915 at 3 MPa consolidated pressure. The agreement between predicted and measured values is better in the early stages of loading and towards the end of the test. The data indicate that Modified Cam clay model can generally represent

the compressive behaviour of the buffer in undrained shear reasonably well .

For the drained condition, constant- p' test results will be compared with the predictions of Cam clay family. The data for two specimens T906 and T918 consolidated isotropically to 1 MPa and 3 MPa pressure respectively are used for the comparison. Figure 8.5 shows the relationship between the normalized shear stress q/q_{max} and shear strain ϵ_s obtained from Modified Cam clay model (MCCM) compared to the experimental data from specimen T918. Here, the finite element results are shown from three different runs with the shear stress q_{max} applied in 4, 20 and 50 increments respectively. The ratio between the pre-consolidation pressure at the end of an increment to the pre-consolidation pressure at the beginning of the same increment, referred to as "yield ratio" YR, is a measure of convergence of the results. Values of about 0.98-1.02 are generally regarded as leading to sufficiently accurate calculations (Britto and Gunn 1987). Application of smaller load increments (that is larger number of increments) leads to a more accurate result as evident in Figure 8.5. The agreement between the test results and MCCM predictions using 50 increments is excellent.

The relationship between axial strain and time under different deviator stress increments in a constant- p' test is examined in Figure 8.6. Here, the test results for specimen T906 with effective mean pressure of 1 MPa is compared to the analytical results obtained from CRISP. Although the analytical results overestimated the initial elastic axial deformation of the buffer, they were in general agreement with the test results during the second and third increments. The

test results in the fourth increment (failure increment) is not reliable due to the generation of large excess porewater pressures which led to the premature failure of the specimen.

8.3 Finite Element Analysis of Vault Behaviour

Recent design concepts for the waste repository regard the eventual flooding of the disposal vault as inevitable (Cameron 1982; Pusch and Borgesson 1985). Experimental evidence (Pusch et al. 1985) shows that the time required to achieve saturation of the vault depends on the groundwater flow characteristics in fissures of the rock mass and the fracture distribution at the buffer-rock interface. The data also suggest that the buffer will be fully saturated during a large portion of the design lifetime of the waste container (500 years). Therefore, this analytical study focuses on the interactions between saturated buffer, container, rock and backfill caused by the influx of water into the waste disposal vault and the development of the swelling pressure in the buffer. This analysis is limited to the thermally inactive condition, where there is no interaction between the generated heat and the incoming fluid. Such a situation can occur at the end of construction of the facility when insufficient time has elapsed for the occurrence of either steady-state or transient heat conduction (Selvadurai et al. 1985). The analysis of the vault problem under two different conditions is discussed here in separate subsections.

8.3.1 Fluid-Induced Rock-Buffer Separation

The presence or development of fissures in the rock mass will

subject the buffer-container system to high porewater pressures that are naturally present in the rock. These fluid pressures can act at arbitrary locations of the buffer-rock interface and the question should be asked if they result in separation between the relatively impermeable buffer and the rock mass (Figure 8.7, after Selvadurai et al. 1985). Alternatively, the water could enter the buffer region via cracks or fissures which may develop by processes similar to hydraulic fracturing or thermally-induced desiccation effects (see for example, Radhakrishna 1984).

The state of deformation in the system is three-dimensional in general. However, it is here assumed for simplicity that the extent and location of the separation zone is such that the deformation is approximately plane strain. The plane strain analysis essentially yields a possible upper bound loading configuration for the estimation of the displacements of the container and the pressures that are generated at the buffer container interface. Selvadurai (1984) developed an analytical solution for the separation problem in a special case where the buffer material is idealized as an ideal elastic-plastic material characterized by a Tresca-type yield condition (i.e. $c \neq 0$; $\phi = 0$). However, extensive triaxial testing conducted for this thesis indicated that such material idealization is not realistic for the buffer. The real strength envelope for the buffer presented in Section 5.5 will be used here in a finite element analysis of the separation problem via a critical state model.

To complete the modelling of the problem it is necessary to make additional simplifying assumptions related to the mechanical characteristics of the individual components of the disposal vault as

follows :

- (1) The rock mass is treated essentially as an intact rigid medium.
- (2) The interface between non-separated zone of the buffer and rock mass is assumed to be completely bounded.
- (3) The deformations in the waste container are small. Thus, it can be modelled as an elastic solid.
- (4) The boundary between the container and the buffer is assumed to be completely bounded.
- (5) The buffer behaviour can be represented by simple elastic-plastic models such as the Modified Cam clay model calibrated in Section 7.3.

Figure 8.8 shows the finite element mesh used for the analysis representing only one-half of the cross section. It is assumed that the buffer-container geometry and loading is symmetrical (with respect to a horizontal axis) to reduce the computational needs. The extent of initial fluid-induced separation at the buffer-rock interface depends on the hydrogeological conditions of the surrounding rock. Thus, it is at this stage an unknown parameter in the calculations. In a complete analysis of the problem an iterative technique should be used to determine the extent of the separation zone (Selvadurai et al. 1985). In the analysis presented here, however, the extent of the separation zone is assumed to be prescribed.

Figures 8.9 shows the results of the finite element analysis for an arbitrary case, where the initial angle of separation zone is 60° as shown in Figure 8.8. Uniform fluid pressure of 10 MPa that could be suddenly applied on the buffer should be considered as an external loading in the early stages of borehole response. Under such a high

external pressure in undrained condition the water phase in the buffer can no longer be assumed incompressible. The compression of the interstitial water results in displacements in the order of 1 mm as shown in Figure 8.9. In addition, large porewater pressures will develop in the buffer. The deformation of the buffer leads to some circumferential extension of the buffer-rock separation zone, lateral translation of the waste container, and more importantly shearing deformation in the buffer as shown in Figure 8.9. Since the initial magnitude of effective stresses are low (say about 250 kPa) in undrained conditions, the buffer will initially deform elastically. The preliminary results presented in Figure 8.9 indicate that the shearing of the buffer is concentrated in a localized zone of high mobilized shear resistance at about 60° anti-clockwise from the axis of symmetry. The analysis reflects the tendency of the buffer to exhibit strain softening behaviour in the zone of the highest shear stress. The results also suggest that the magnitude of effective normal stresses at the rock-buffer interface becomes smaller near the prescribed separation zone. This reflects the tendency of the separation zone to extend tangentially.

8.3.2 Long Term Deformation of Buffer

The mechanism of time-dependent swelling of the buffer is complex, involving changes in soil fabric structure and redistribution of water within the soil. Yong et al. (1986) studied the development of swelling pressure for different aggregate-clay mixtures. Figure 8.10 shows a schematic of the swelling pressure build-up and relief mechanism in confined swell tests. The high swelling pressure initially

developed (due to the high suction capacity of the compacted clay in the mixture) subsides following continued water intake. In the disposal vault, the buffer will be surrounded by the rock, the container and the backfill. It starts to develop swelling pressure upon access to groundwater. The resulting pressure interacts with the surrounding environment. If deformation can take place the swelling pressure will be reduced. The deformation mainly depends on the density of the emplaced buffer, relative stiffnesses of the backfill, the container and the rock. Although the state of deformation in the system is generally three-dimensional, it is reasonable to assume that the deformation is axi-symmetrical with respect to central axis of the container. To facilitate modelling of this complex interaction problem, further simplifying assumptions (some similar to those in fluid-induced separation case) are made as follows :

- (1) The rock mass is treated essentially as an intact rigid medium.
- (2) The interface between the buffer and rock mass is assumed to be completely unbounded with no frictional forces between them.
- (3) The waste container is modelled as an elastic solid.
- (4) The boundary between the container and the buffer is assumed to be completely bounded, with full mobilization of frictional stresses.
- (5) The buffer behaviour can be represented by simple elastic-plastic models such as modified Cam clay model (see Section 7.3).

Figure 8.11 shows the finite element discretization representing only one-half of the vertical cross section. The axi-symmetrical condition of the buffer-container geometry and loading drastically reduces the computational memory requirements. The finite element analysis is used to examine the buffer behaviour under various loading

conditions. Two different cases of loading are considered in this section. In the first case the buffer is assumed to be in equilibrium under its own weight in a dry borehole before the container weight and backfill loading are applied. Secondly, the buffer is assumed to have reached equilibrium in a wet borehole under a "no- volume- change" condition. The interaction between the swelling pressures developed in the buffer and the backfill will then be examined in detail. The results are presented in terms of stresses and displacements trajectories.

8.3.2.1 Container Weight and Backfill Loading

The buffer compacted at 95% ASTM modified density will be initially in equilibrium with the in-situ overburden stresses from overlying buffer. Then, it will be subjected to the weight of the container and the surcharge pressure from the vault backfill. The weight of the container is here applied as uniformly distributed pressure of 277 kPa at the bottom of the container. The backfill surcharge pressure at the buffer surface is assumed to be uniformly distributed and approximately equal to 120 kPa in magnitude (Yong et al. 1986). Figure 8.12 depicts the resultant vertical displacement contours in both undrained and drained conditions. The displacement patterns in the buffer are mainly influenced by the presence of the relatively rigid container. The results are comparable with the measured response of the physical model tests reported by Yong et al. 1986). The vertical displacement patterns obtained in the finite element analysis generally agree qualitatively with measured values in the model tests. It should be remembered that scaling

between model and prototype in terms of extrapolation of results can not be readily applied using simple geometric or mass scaling laws, mainly because the material properties are not scaled. Nevertheless, the analysis appears to overestimate the magnitude of long term settlement of the buffer in the drained condition and underestimates the undrained deformation of the buffer. The finite element results were checked by simple hand calculations. For instance, using approximate vertical stress distribution in the buffer beneath the container, the container settlement in the drained condition was estimated at approximately 11.9 mm. This compares favorably with calculated settlement of 11.75 mm from the finite element analysis.

Figure 8.13 shows the resultant vertical stress σ'_y and horizontal stress σ'_x trajectories within the buffer mass from the finite element analysis. The stress distributions are obviously influenced by the presence of the rigid container leading to non-uniform stress distributions, particularly around the corners of the container. It is evident that the mean stresses developed in the buffer at its interface with the backfill are in good agreement with the overburden pressure imposed. Relatively high vertical stresses develop in the buffer at the bottom of the container. The buffer material confined between the container and the host rock walls is subjected to compression by the upward movement of the buffer around the container bottom corner and the downward movement of the buffer at the top of the container due to the overburden pressure. This results in high stress concentrations near the container bottom corner, and lower stresses in the buffer along the height of the container. The horizontal stresses exhibit similar behaviour. The stress distribution in the buffer above the

container is essentially uniform, while the distribution around the container is non-uniform. The development of high horizontal stresses within the buffer significantly influences the interaction between the container-buffer-rock system, especially where the interfaces are not sufficiently smooth to reduce friction.

8.3.2.2 Long Term Swelling of the Buffer

To examine the long-term implication of the development of swelling pressures in the buffer, an idealized drained case of swelling and pressure relief in the borehole is examined here. Under a "no-volume-change" condition, the buffer (at the emplacement density) could develop swelling pressures up to 2.13 MPa as discussed in Section 6.2. However, interaction between the swelling pressures in the buffer and the backfill leads to deformation of the backfill. The relief of the swelling pressures upon deformation of the backfill results in redistribution of stresses in the borehole. An analysis of the problem was performed under displacement controlled condition, allowing uniform swelling displacements of up to 50 mm at the top of the borehole in small increments.

The resultant normalized stress ratio σ/M and vertical displacement trajectories in the buffer mass are summarized in Figures 8.14. The contours of vertical displacement of the buffer are shown in Figure 8.14 (b). The results indicate that the waste container will lift up approximately 17.4 mm as a result of swelling of the buffer if the backfill displaces upward uniformly by 50 mm. The swelling of the buffer above the container will be essentially uniform in this case. Figure 8.15 shows that the vertical and horizontal stress distribution

in the buffer are non-uniform due to the interaction with the container. It is apparent that swelling of the buffer results in considerable release of vertical stresses in the buffer, while the horizontal stresses are reduced marginally. As a result large shear stresses and strains are produced in the buffer which may potentially lead to localized yielding as shown in Figure 8.14 (a). The shear stress ratio τ/M is highest (about 1.02) around the corners of the container in the buffer. It should however be remembered that the buffer will be strain hardening upon further straining. The buffer can potentially approach the critical state conditions especially at the bottom of the container. That means confined zones of potentially large plastic deformations might then be expected in the buffer in the long-term. However, the buffer material confined between the container and the host rock walls is not very much affected by shear deformations.

It is important to note that a more refined finite element analysis of the problem could improve the accuracy of the stress trajectories in the buffer. It should also be appreciated that the presence of different material interfaces, sharp corners, rigid walls in the region high stress concentrations could have reduced the accuracy of the stress solutions.

CHAPTER NINE

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

9.1 Summary

Extensive high pressure triaxial laboratory testing has been performed on the buffer at mean effective pressures up to 9 MPa, and porewater pressures up to 7 MPa at ambient temperatures. Preceding chapters have outlined this work in detail and may be summarized as follows:

1. The material properties exhibit the strong tendency of smectite clays to expand under low stresses less than their swelling pressure. The long term swelling characteristic of the buffer is inhibited under higher confining pressures.
 2. The compaction moisture content strongly influences the microstructure of the clay-water in the buffer. The higher density buffer forms a more orderly (dispersed) microstructure than the lower density buffer. This affects the compressibility and strength properties of the buffer.
 3. The compressibility of reconstituted buffer from slurry (Dixon et al. 1986) was found to be higher than the low density buffer in this program. The higher the compaction density of the buffer,
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the lower the compressibility under isotropic compression and the "equilibrium" specific volume.

4. The strength of the buffer is dominated by the bentonite. Electron micrography on the reference buffer material has shown that the sand grains are dispersed through the soil matrix and do not significantly contribute to the strength of the buffer.
 5. In the early stages of shearing, compressive specimens behave anisotropically with an average porewater pressure parameter $m \approx 2$, while dilative specimens behave isotropically with $m \approx 1$.
 6. The strength envelope defined by compressive specimens is $c' = 0$, $\phi' = 14^\circ$ at pressures up to 3.5 MPa. Dilative specimens have slightly higher strength.
 7. At pressures higher than 3.5 MPa the evidence supports lower "local" ϕ -values with increasing consolidation pressure. Curved strength envelopes in the form of power laws are proposed in [5.1] and [5.2] for the whole range of stresses up to 10 MPa for low density and high density buffer respectively.
 8. The results indicate that the compacted buffer is anisotropic with horizontal stiffness twice its vertical stiffness.
 9. Three different approaches to constitutive modelling namely, hyperelastic, hypoelastic and elastic-plastic models were
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developed in this thesis and compared successfully with the experimental data.

10. The material behaviour was idealized by hyperelastic and hypoelastic models using both coupled (anisotropic) and uncoupled (isotropic) formulations.
 11. In uncoupled hyperelastic formulation, the pseudo-elastic normalized bulk stiffness of the buffer does not seem to be significantly affected by stress level, whereas normalized shear modulus decreases with increasing confining pressures.
 12. A three-modulus anisotropic hyperelastic was proposed for the pseudo-elastic range. The model permits coupling of mean pressures with shear strains, or deviator stresses with volume strains.
 13. A three-function hypoelastic model was introduced to describe the constitutive relationships for straining-to-failure. This model allows prediction of stress-strain behaviour of the buffer along generalized stress paths.
 14. An elastic-plastic model belonging to the Cam clay family was defined for the buffer which accounts for non-reversibility, non-linearity and dilatancy in the plastic range.
 15. The interaction between buffer, container, rock and backfill was
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examined under different vault conditions using a non-linear finite element program (CRISP) with the proposed elastic-plastic model for the buffer.

9.2 Principal Conclusions

The principal conclusions from the work are as follows:

1. The swelling pressure of the buffer depends on the initial compaction moisture content or compaction density. The equivalent (isotropic) swelling pressures for buffer compacted to 85% and 95% ASTM modified density were found to be about 0.9 MPa and 2.1 MPa respectively.
 2. A generalized specific volume - pressure relationship at equilibrium has been proposed in [6.9] which accounts for the influence of compaction moisture content and time on fabric structure of the buffer.
 3. The material behaviour is ductile, time-dependent, and slightly strain-softening in shear.
 4. A coherent conceptual model based on critical state soil mechanics was developed to describe the behaviour of the buffer compacted to different densities.
 5. The preliminary results of the analysis suggest that swelling of
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the buffer against compressive backfill could potentially produce large shear strains in the buffer. However, the regions where high proportions of the strength were mobilized were confined and small.

9.3 Suggestions for Future Research

1. The buffer should maintain its integrity in the vault environment, where the temperature could approach 100°C and the porewater pressures will be as high as 10 MPa. Laboratory triaxial tests need to be performed at high temperatures and porewater pressures to provide the necessary understanding of the behaviour of the saturated buffer under these conditions. The testing program should be especially designed to study the applicability of the "effective" stress principle at elevated temperature, and the time- dependency of the behaviour.
 2. The initial degree of saturation of the compacted buffer after placement in the borehole is about 75% . The buffer is also expected to be unsaturated in the desiccated zone close to the fuel waste container. It is therefore necessary to gain an appreciation of the behaviour of unsaturated buffer in the borehole before vault flooding.
 3. The buffer will be influenced by time- dependent transient moisture and heat flow processes in the vault. The coupled heat-moisture transport mechanisms should be studied further in order.
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provide a meaningful analytical model for investigation of the buffer behaviour in the actual vault environment.

4. More laboratory testing is needed to examine the nature of the state boundary surface and the plastic flow rule for both saturated and unsaturated buffer.
 5. In-situ large-scale prototype tests are necessary for validating numerical and analytical models of buffer-container-rock-backfill problem.
 6. The influence of compaction moisture content on the microstructure of the clay-water phase in the buffer should be studied further using the scanning electron microscopy.
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APPENDIX (A)

DETAILED SCHEDULE OF TESTING PROGRAM

- T901: Consolidated directly to 0.6 MPa; Sheared undrained.
- T902: Consolidated directly to 1.0 MPa; Sheared undrained.
- T903: Consolidated directly to 1.5 MPa; Sheared undrained.
- T904: Consolidated directly to 1.5 MPa; Sheared in 5 increments of deviator stress in drained load-controlled shearing to examine the shear modulus behaviour G.
- T905: Consolidated in five increments of cell pressure from 0.33 MPa to 1.0 MPa with Load Ratio 1.32; Sheared undrained.
- T906: Consolidated directly to 1.0 MPa; then sheared in 5 increments of deviator stress as in T904.
- T907: Consolidated for 15 days to 3.00 MPa. Due to equipment failure the cell pressure had to be lowered again to 1.5 MPa. After stabilizing for a further 13 days at this pressure the specimen was sheared undrained.
- T908: Consolidated directly to 1.2 MPa; Sheared undrained.
- T909: Consolidated directly to 0.8 MPa; Sheared undrained.
- T910: Consolidated directly to 1.2 MPa; Sheared in five increments of deviator stress in drained load-controlled shearing to examine the shear modulus behaviour G.
- T911: Consolidated directly to 0.8 MPa; Sheared in six increments of deviator stress (with the final increment being half as large as the others) in drained load-controlled shearing to examine the shear modulus behaviour G.
- T912: Consolidated in five increments of cell pressure from 0.5 MPa to 1.5 MPa with Load Ratio 1.32; Sheared undrained.
- T913: Consolidated directly to 3 MPa; Sheared undrained.
- T914: Consolidated directly to 2.2 MPa; Sheared undrained.

- T915: Consolidated in five increments of cell pressure from 1.0 MPa to 3.0 MPa with Load Ratio 1.32; Sheared undrained.
- T916: Consolidated directly to 2.2 MPa; Sheared in 5 increments of deviator stress in drained load-controlled shearing to examine the shear modulus behaviour.
- T917: Consolidated in five increments of cell pressure from 0.73 MPa to 2.2 MPa with Load Ratio 1.32; Sheared undrained.
- T918: Consolidated directly to 3.0 MPa; Sheared undrained.
- T919: Consolidated directly to 1.5 MPa; Sheared undrained.
- T920: Compacted to 1.64 Mg/m^3 at 23.0%; Consolidated directly to 3.0 MPa; Sheared undrained.
- T921: Compacted to 1.62 Mg/m^3 at 23.4%; Consolidated directly to 2.2 MPa; Sheared undrained.
- T922: Consolidated directly to 6.0 MPa; Sheared undrained.
- T923: Consolidated directly to 1.5 MPa with incremental increase of back pressure to 1.8 MPa; Sheared undrained.
- T924: Compacted to 1.75 Mg/m^3 at 19.0%; Consolidated directly to 3.0 MPa; Sheared undrained .
- T925: Consolidated directly to 9.0 MPa; Sheared undrained.
- T926: Compacted to 1.70 Mg/m^3 at 21.1%; Consolidated directly to 0.2 MPa; Sheared undrained.
- T927: Compacted to 1.66 Mg/m^3 at 22.4%; Consolidated directly to 1.5 MPa; increased back pressure from 0.2 MPa to 1.0 MPa and then 1.8 MPa in two stages keeping the effective stress constant; Sheared undrained.
- T928: Compacted to 1.67 Mg/m^3 at 22.4%; Consolidated directly to 6.0 MPa; Sheared undrained.

- T929: Compacted to 1.67 Mg/m^3 at 22.5%; Consolidated in five increments of cell pressure from 1.0 MPa to 3.0 MPa with load ratio 1.32; Sheared undrained.
- T930: Compacted to 1.67 Mg/m^3 at 22.3%; Consolidated in five increments of cell pressure from 3.0 MPa to 9.0 MPa with load ratio 1.32; Sheared undrained.
- T931: Compacted to 1.66 Mg/m^3 at 22.3%; Consolidated directly to 3.0 MPa; sheared in 8 increments of deviator stress in drained load-controlled shearing with a unload-reload cycle to examine the shear modulus.
- T932: Compacted to 1.66 Mg/m^3 at 22.8%; Consolidated directly to 1.5 MPa; increased back pressure from 1.0 MPa to 4.0 MPa keeping the effective stress constant; Sheared undrained.
- T933: Compacted to 1.66 Mg/m^3 at 22.8%; Consolidated directly to 3.0 MPa; increased the back pressure from 1 MPa to 7 MPa keeping the effective stress constant; Sheared undrained.
- T934: Compacted to 1.67 Mg/m^3 at 22.4%; Consolidated directly to 9 MPa; Sheared conventional drained.
- T935: Compacted to 1.50 Mg/m^3 at 28%; Consolidated directly to 9 MPa; Sheared undrained.

TABLE 3.1 - Input, Output and DMM Voltage Ranges Used for Various Devices

Device	Input Voltage	Output Range	DMM Range
LVDT	6	± 4 V	± 20 V
Load Cell	5/10/20	0 to 100 mV	± 200 mV
Volume Change (LVDT)	6	± 4 V	± 20 V
Pressure Transducer	5	0 to 200 mV	± 200 mV

TABLE 4.1 - Quality Control Test Results - T900 Series

Specimen No. ¹	<u>TARGETTED</u>		<u>MEASURED</u>		
	Water Content percent	Dry Density (Mg/m ³)	Water Content percent	Standard Deviation	Dry Density Mg/m ³)
(A)					
QC901	28.0	1.50	24.6	± 0.7 ²	1.57
QC902	29.0	1.58	27.3	± 0.4	1.52
QC903	30.4	1.58	28.0	± 0.2	1.51
QC904	30.4	1.58	27.9	± 0.4	1.51
QC905	29.5	1.55	27.5	-	1.52
QC906	29.5	1.55	27.8	-	1.51
QC907	30.0	1.54	28.2	± 0.2	1.51
QC908	30.0	1.54	28.5	± 0.1	1.51
(B)					
QC909	20.0	1.67	19.1	± 0.4	1.68
QC910	23.0	1.67	22.3	± 0.4	1.66

1 All samples were formed after curing sand-clay mix for 3 days and deairing the mix for 5 minutes.

2 Water content and standard deviation are taken from ten sub-samples taken from the complete specimen to check its uniformity.

TABLE 4.2 - Initial Condition of Triaxial Samples at Beginning of Consolidation

Specimen No.	Moisture Content (%)	Dry Density (Mg/m ³)	Specific Volume V	Eff. Clay Dry Density (Mg/m ³)	Clay Specific Volume V _c	Saturation (%)
T901	27.5	1.52	1.74	1.07	2.57	99.0
T902	27.8	1.52	1.75	1.07	2.57	98.0
T903	29.0	1.49	1.78	1.04	2.64	97.5
T904	27.6	1.51	1.75	1.06	2.59	98.0
T905	27.5	1.52	1.74	1.07	2.57	98.0
T906	28.0	1.51	1.76	1.06	2.59	98.0
T907	27.3	1.51	1.74	1.06	2.59	93.2
T908	29.9	1.49	1.81	1.04	2.64	100.0
T909	29.6	1.47	1.80	1.05	2.63	95.0
T910	29.7	1.48	1.80	1.04	2.63	97.0
T911	30.2	1.47	1.82	1.03	2.66	97.0
T912	27.9	1.52	1.75	1.09	2.53	96.6
T913	27.9	1.52	1.75	1.09	2.53	95.6
T914	25.6	1.58	1.69	1.14	2.41	97.5
T915	27.6	1.51	1.75	1.09	2.52	95.3
T916	28.9	1.50	1.78	1.06	2.59	98.5
T917	29.0	1.48	1.78	1.06	2.60	95.3
T918	28.2	1.50	1.76	1.08	2.55	95.2
T919	29.2	1.48	1.79	1.06	2.61	95.3
T920	23.0	1.64	1.62	1.21	2.27	95.9
T921	23.4	1.62	1.63	1.20	2.28	94.8
T922	28.0	1.51	1.76	1.08	2.54	95.4
T923	28.4	1.51	1.77	1.07	2.56	97.2
T924	19.0	1.75	1.51	1.34	2.05	94.8
T925	27.9	1.52	1.75	1.09	2.53	96.7
T926	21.1	1.70	1.57	1.27	2.16	96.8
T927	22.4	1.66	1.61	1.23	2.23	97.0
T928	22.4	1.67	1.61	1.23	2.23	97.5
T929	22.5	1.67	1.61	1.23	2.24	98.1
T930	22.3	1.67	1.60	1.24	2.23	97.7
T931	22.3	1.66	1.60	1.24	2.23	96.2
T932	22.4	1.66	1.61	1.23	2.23	95.7
T933	22.8	1.66	1.62	1.22	2.25	96.1
T934	22.6	1.65	1.61	1.23	2.24	96.1
T935	28.3	1.53	1.76	1.08	2.56	99.7

Mean ± St. Dev.

Series

(I)	28.4 ±0.9	1.50 ±0.02	1.77 ±0.02	1.07 ±0.02	2.58 ±0.04	98 ±1.5
(III)	22.5 ±0.2	1.66 ±0.01	1.61 ±0.01	1.23 ±0.01	2.24 ±0.01	97 ±0.9

TABLE 4.3 SCHEDULE OF TRIAXIAL TESTING PROGRAM

Series (I) - Compacted at 1.5 Mg/m^3 , 28% water content

Specimen No	Test Type	Consolidation Pressure - MPa	Parameters Examined
T901	A	0.6	G_u, J, s_u
T902	A	1.0	G_u, J, s_u
T903	A	1.5	G_u, J, s_u
T904	C	1.5	G, J
T905	B	1.0	K, G_u, J, s_u
T906	C	1.0	G, J
T907 ¹	A	1.5	G_u, J, s_u
T908	A	1.2	G_u, J, s_u
T909	A	0.8	G_u, J, s_u
T910	C	1.2	G, J
T911 ²	C	0.8	G, J
T912	B	1.5	K, G_u, J, s_u
T913	A	3.0	G_u, J, s_u
T914	A	2.2	G_u, J, s_u
T915	B,E	3.0	$K, G_u, J, s_u, \rho_{0.1}$
T916	C	2.2	G, J
T917	B	2.2	K, G_u, J, s_u
T918 ³	C	3.0	G, J
T919	A	1.5	G_u, J, s_u
T922	A	6.0	G_u, J, s_u
T923 ⁴	A,I	1.5	G_u, J, s_u
T925	A	9.0	G_u, J, s_u
T935	A	9.0	G_u, J, s_u

Series (II) - Compacted at equilibrium water content (for a given confining pressure)

Specimen No	Test Type	Consolidation Pressure - MPa	Parameters Examined
T920	A,E	3.0	$G_u, J, s_u, \rho_{0.1}$
T921	A,E	2.2	$G_u, J, s_u, \rho_{0.1}$
T924	A	3.0	G_u, J, s_u
T927 ⁴	A,G,I	1.5	G_u, G_u^e, J, s_u

(Continued)

TABLE 4.3 SCHEDULE OF TRIAXIAL TESTING PROGRAM (Continued)

Series (III) - Compacted at 1.67 Mg/m^3 , 22.5% water content

Specimen No	Test Type	Consolidation Pressure - MPa	Parameters Examined
T926	A	0.2	G_u, J, s_u
T928	A	6.0	G_u, J, s_u
T929	B,F	3.0	K, G_u, J, s_u
T930	B	9.0	K, G_u, J, s_u
T931 ⁵	A,H	3.0	G_u, G_e, J, s_u
T934	A	9.0	K, G, J

Series (IV) - Compacted at various densities

Specimen No	Test Type	Consolidation Pressure - MPa	Parameters Examined
T932 ⁶	A,I	1.5	G_u, J, s_u
T933 ⁷	A,I	3.0	G_u, J, s_u

- 1 Initially consolidation to 3.0 MPa for 15 days. Leakage and cell sleeve failure required pressure to be reduced to 1.5 MPa after 13 days.
- 2 Sheared in 6 increments of deviator stress (with the final increment being half as large as the others)
- 3 Failed in the fourth increment of deviator stress
- 4 Back pressure was increased incrementally to 1.8 MPa keeping effective stress constant
- 5 Sheared in 8 increments (with the last five increments being half as large as the others)
- 6 Back pressure was increased to 4 MPa keeping the effective stress constant
- 7 Back pressure was increased to 7 MPa keeping the effective stress constant

(Continued)

TABLE 4.3 SCHEDULE OF TRIAXIAL TESTING PROGRAM (Continued)

DESCRIPTION OF DIFFERENT TYPES OF TESTSTest Type

- (A) Consolidated directly to P'_{cons} ; sheared undrained (CIU)
- (B) Consolidated in 5 increments of cell pressure from $P'_{cons}/3$ to P'_{cons} with load increment ratio 1.32; sheared undrained (CIU)
- (C) Consolidated directly to P'_{cons} ; sheared in 5 increments of deviator stress in drained load-controlled shearing (CID, constant- p')
- (D) Consolidated directly to P'_{cons} ; sheared drained in strain controlled test (CID)
- (E) Post-peak relaxation and/or strain rate step-changing undrained shear
- (F) Incremental unload-reload test during isotropic compression
- (G) Post-peak unload-reload undrained shear
- (H) Pre-failure incremental unload-reload constant- p' drained
- (I) Multi-stage consolidation under elevated back pressure

TABLE 4.4 - Skempton Porewater Pressure Parameters (A and B) for Undrained Tests

Specimen No.	Initial Degree of Saturation S_r (%)	Back Pressure (A_f) (kPa)	(B) within 20 Minutes	(B) within 1 Hour	Observations
<u>Compressive</u>					
T901	99.0	200	0.71	1.00	leakage
T902	98.0	200	0.52	1.00	leakage
T903	97.5	200	0.80	1.00	leakage
T905	98.0	500	0.75	1.00	-
T907	93.2	400	0.12	0.67	1.00
T908	100.0	300	0.80	1.00	-
T909	95.0	200	0.72	0.90	1.00
T912	96.6	0	0.22	0.17	-
T913	95.6	200	0.19	0.28	1.00
T914	97.5	200	0.54	1.00	-
T915	95.3	200	0.91	1.00	-
T917	95.3	200	1.02	-	0.96
T919	95.3	350	0.47	0.7	-
T920	95.9	400	0.77	-	1.00
T921	94.8	500	0.69	0.12	1.00
T922	95.4	1000	0.94	0.55	0.55
T923	97.2	1800	0.69	0.81	1.00
T924	94.8	400	0.16	-	0.50
T925	96.7	1000	1.16	0.71	1.00
T928	97.5	1000	0.74	0.71	1.00
T929	98.1	300	0.93	1.00	-
T930	97.7	1000	1.81	0.56	1.00
T933	97.8	7000	0.66	0.56	0.99
T935	99.7	1000	2.65	0.69	1.00
<u>Dilative</u>					
T926	96.8	200	0.39	0.97	-
T927	97.0	200-1800	0.28	0.78	1.00
T932	95.7	4000	0.07	0.66	-

TABLE 4.5 - Measured Volume Strains During Consolidation

Specimen No.	Consolidation Pressure (MPa)	Consolidation Duration (days)	Volume Strain (%)	Clay Specific Volume (Vc) _{EOC}
T901	0.6	31	8.3	2.62
T902	1.0	34	8.3	2.54
T903	1.5	28	5.3	2.43
T904 ¹	1.5	31	-	-
T905 ¹	1.0	57	7.4	2.52
T906 ²	1.0	12	5.5	2.52
T907 ²	1.5	15	-	2.40
T908	1.2	16	7.4	2.52
T909	0.8	15	4.7	2.61
T910	1.2	18	6.9	2.54
T911 ¹	0.8	18	4.9	2.60
T912 ¹	1.5	25	4.6	2.40
T913	3.0	16	8.2	2.27
T914 ¹	2.2	9	3.8	2.23
T915 ¹	2.9	60	8.7	2.23
T916 ¹	2.2	51	7.8	2.31
T917 ¹	2.2	46	7.8	2.30
T918	3.0	16 ⁵	6.3	2.27
T919	1.5	37 ⁵	4.9	2.43
T920	3.0	10	2.7	2.16
T921	2.2	13	2.1	2.23
T922 ⁴	6.0	27	10.9 ⁽⁶⁾	2.14
T923 ⁴	1.5	41	5.1	2.38
T924 ¹	2.9	17	-0.2	2.05
T925 ¹	8.8	32	13.2	2.06
T926	0.2	24	-15.9	2.65
T927	1.5	12	-0.2	2.21
T928 ^{1,3}	5.8	19	4.0	2.02
T929 ¹	2.7	51	1.8	2.17
T930 ¹	8.7	35	4.8	2.04
T931 ⁴	3.0	12	4.8	2.14
T932 ⁴	1.5	50	1.5	2.18
T933	3.0	15	1.2	2.12
T934	9.0	23	4.8	2.05
T935	9.0	35	13.1	2.04

(Continued)

TABLE 4.5 - Measured Volume Strains During Consolidation (Continued)

- 1 Incremental consolidation in 5 increments, with load ratio 1.32, up to the specified pressure.
- 2 Initially consolidation to 3.0 MPa for 15 days. Leakage and cell sleeve failure required pressure to be reduced to 1.5 MPa after 13 days.
- 3 Unload -reload cycle included.
- 4 Elevated porewater pressure.
- 5 Sample was later unloaded and loaded again.
- 6 Volume strain was back calculated using end-of-test moisture content.

TABLE 4.6 - Summary of Undrained Shear Test Results at Failure

Specimen No.	σ'_{cons} (MPa)	γ_c	ϵ_{1f} (%)	q_f (MPa)	p'_f (MPa)	$(q/p')_f$	u_f (MPa)	A_f	$\dot{\epsilon}_1$ % /hour
T901	0.6	1.05	4.63	0.36	0.58	0.62	0.14	0.71	1.20
T902	1.0	1.08	4.64	0.42	0.94	0.45	0.20	0.52	1.20
T903	1.5	1.13	3.42	0.64	1.20	0.53	0.52	0.80	1.20
T905	1.0	1.09	2.97	0.65	0.94	0.69	0.28	0.75	1.20
T907 ⁽¹⁾	1.5	1.15	3.56	0.63	1.63	0.38	0.08	0.12	1.20
T908	1.2	1.09	4.51	0.58	0.93	0.62	0.46	0.80	1.20
T909	0.8	1.05	5.08	0.44	0.60	0.73	0.32	0.72	1.20
T912	1.5	1.15	5.04	0.65	1.57	0.41	0.14	0.22	1.20
T913	3.0	1.21	4.86	1.02	3.16	0.32	0.19	0.19	1.20
T914	2.2	1.23	5.26	1.08	1.98	0.55	0.59	0.54	1.20
T915 ⁽²⁾	3.0	1.21	5.00	1.24	2.28	0.54	1.13	0.91	0.24
T917 ⁽²⁾	2.2	1.20	5.99	0.92	1.57	0.59	1.13	1.02	0.24
T919	1.5	1.13	3.37	0.71	1.44	0.49	0.30	0.47	0.24
T920 ⁽²⁾	3.0	1.27	5.46	1.44	2.38	0.61	1.10	0.77	0.24
T921	2.2	1.23	5.79	1.08	1.83	0.59	0.73	0.69	0.24
T922	6.0	1.29	4.02	2.27	4.62	0.49	3.14	0.94	0.10
T923	1.5	1.16	5.22	0.68	1.26	0.54	0.47	0.69	0.24
T924	3.0	1.34	7.32	1.76	3.3	0.53	0.28	0.16	0.24
T925	8.8	1.33	15.89	1.76	7.31	0.24	2.05	1.16	0.10
T926	0.2	1.04	15.65	0.23	0.37	0.63	0.09	0.39	0.24
T927 ⁽³⁾	1.5	1.24	10.55	1.18	1.56	0.76	0.33	0.28	0.24
T928	6.0	1.36	3.51	2.10	5.02	0.42	1.68	0.74	0.10
T929	2.65	1.27	3.33	1.30	2.21	0.59	1.23	0.93	0.24
T930	9.0	1.35	16.68	2.07	5.96	0.35	3.74	1.81	0.10
T932	1.5	1.26	5.04	1.13	1.83	0.62	0.08	0.07	0.24
T933	3.0	1.30	5.08	1.50	2.52	0.6	0.96	0.66	0.10
T935 ⁽²⁾	9.0	1.35	3.62	2.05	4.26	0.48	5.42	2.65	0.10

(1) Confining pressure 3 MPa for 15 days, subsequently reduced to 1.5 MPa for 13 days.

(2) Relaxation tests / step changing speed.

(3) Unload -reload cycle in shear.

TABLE 4.7 - Summary of Drained Shear Test Results

Specimen No.	σ'_{cons} (MPa)	ϵ_{1f} (%)	q_f (MPa)	$(q/p)_f$	G_{50} (MPa)	(3-Days) $G_{50}/\sigma_{\text{cons}}$	
						<u>Tangent</u>	<u>Secant</u>
T904	1.5	9.01	1.0	0.67	7.2	4.8	13.7
T906	1.0	2.99	0.52	0.52	7.1	7.1	10.3
T910	1.2	3.85	0.73	0.61	11.4	9.5	10.2
T911	0.8	4.57	0.53	0.67	7.7	9.6	18.4
T916	2.2	4.72	1.09	0.50	12.5	5.7	10.5
T918	3.0	4.11	1.20	0.40	8.1	2.7	6.4
T931	3.0	6.13	1.93	0.64	27.9	9.3	18.2
T934 ⁽¹⁾	9.0	8.98	2.94	0.29	-	-	-

Mean ± St. Dev.

7.0 ± 2.7 12.5 ± 4.5

(1) Conventional Drained Test

TABLE 4.8 - Summary of Initial and Final Water Contents of Triaxial Specimens

<u>Water Content (%)</u>						
Specimen No.	<u>Initial</u>		<u>Final</u>		Difference	Source of Discrepancy
	Measured	Calculated	Measured	Calculated		
T901	27.5	-	29.5	22.0	7.5	flushing & leakage flushing & leakage flushing & leakage flushing & leakage flushing & air flushing leakage flushing flushing flushing
T902	27.8	-	28.0	22.8	5.2	
T903	29.0	-	25.5	19.1	6.4	
T904	27.6	-	24.9	-	-	
T905	27.5	-	27.6	23.2	4.4	
T906	28.0	-	26.8	23.5	3.3	
T907	27.3	-	25.5	-	-	
T908	29.9	-	27.6	24.9	2.7	
T909	29.6	-	29.2	27.3	1.9	
T910	29.7	-	26.0	24.5	1.6	
T911	30.2	30.5	28.2	27.9	0.3	air connection leaked leakage saturation saturation saturation
T912	27.9	28.5	26.0	25.4	0.6	
T913	27.9	28.4	23.0	22.4	0.6	
T914	25.6	25.6	23.2	23.2	0	
T915	27.6	28.0	22.3	21.9	0.4	
T916	28.9	29.0	23.4	23.3	0.1	
T917	29.0	28.9	23.7	23.8	0.1	
T918	28.2	28.2	23.1	23.1	0	
T919	29.2	-	25.9	-	-	
T920	23.0	22.6	21.0	21.4	0.4	
T921	23.4	23.8	22.4	22.0	0.4	saturation saturation saturation saturation saturation saturation saturation saturation saturation saturation
T922	28.0	-	20.8	25.0	4.2	
T923	28.4	-	25.0	25.6	0.6	
T924	19.0	19.0	19.2	19.2	0	
T925	27.9	-	19.2	-	-	
T926	21.1	20.7	30.0	30.4	0.4	
T927	22.4	22.1	22.0	22.3	0.3	
T928	22.4	-	18.5	20.0	1.5	
T929	22.5	22.3	21.2	21.4	0.2	
T930	22.3	21.8	18.9	19.4	0.5	
T931	22.3	21.8	20.2	20.7	0.5	saturation saturation saturation saturation saturation
T932	22.4	-	21.5	22.8	1.3	
T933	22.8	-	20.4	22.0	1.6	
T934	22.6	21.9	17.3	18.0	0.7	
T935	28.3	27.4	18.9	19.8	0.9	

TABLE 5.1 - Summary of Normalized Undrained Shear Test Results at Failure

Specimen	σ'_{cons}	$(V_c)_{\text{EOC}}$	ϵ_{1f}	$q_f/\sigma'_{\text{cons}}$	$(q/p')_f$	$\Delta u_f/\sigma'_{\text{cons}}$	m
No.		(MPa)	(%)				
T901	0.6	4.63	2.62	0.60	0.62	0.22	1.43
T902	1.0	4.64	2.54	0.42	0.45	0.20	-
T903	1.5	3.42	2.43	0.43	0.53	0.35	3.01
T905	1.0	2.97	2.52	0.36	0.42	0.69	2.14
T907 ⁽¹⁾	1.5	3.56	2.40	0.42	0.38	0.05	-
T908	1.2	4.51	2.52	0.48	0.62	0.38	2.39
T909	0.8	5.08	2.61	0.55	0.73	0.4	2.15
T912	1.2	5.04	2.40	0.43	0.41	0.13	-
T913	3.0	4.86	2.27	0.34	0.32	0.63	-
T914	2.2	5.26	2.23	0.49	0.55	0.27	1.13
T915	3.0	5.00	2.27	0.41	0.54	0.38	1.87
T917	2.2	5.99	2.31	0.42	0.59	0.43	2.65
T919	1.5	3.37	2.43	0.47	0.49	0.20	2.11
T920	3.0	5.46	2.16	0.48	0.61	0.37	2.58
T921	2.2	5.79	2.23	0.49	0.59	0.33	2.45
T922	6.0	4.02	2.14	0.38	0.49	0.36	-
T923	1.5	5.22	2.38	0.45	0.54	0.31	1.34
T924	2.9	7.32	2.05	0.59	0.53	0.09	0.16
T925	8.8	15.89	2.06	0.20	0.24	0.23	6.0
T926	0.2	15.65	2.65	1.15	0.63	0.44	0.47
T927	1.5	10.55	2.21	0.79	0.76	0.22	1.27
T928	6.0	3.51	2.02	0.35	0.42	0.26	2.17
T929	2.7	3.33	2.17	0.43	0.59	0.40	2.27
T930	9.0	16.68	2.07	0.23	0.35	0.42	2.68
T932	1.5	5.04	2.18	0.75	0.62	0.05	1.73
T933	3.0	5.08	2.12	0.50	0.60	0.33	3.02
T935	9.0	3.62	2.04	0.23	0.48	0.60	5.51

(1) Confining pressure 3 MPa for 15 days, subsequently reduced to 1.5 MPa.

TABLE 5.2 - Porewater Pressure Drop Between Peak and 12% Axial Strain for Dilative Specimens

Specimen No.	σ_{cons} (kPa)	Δu (kPa)	$\Delta u / \sigma_{\text{cons}}$	End-of-Consolidation Moisture Content (%)

900-Series

T924	3000	34	0.01	19.2
T926	200	67	0.34	30.0
T927	1500	112	0.08	22.0

800-Series (Sun 1986)

T803	200	44	0.22	34.9
T807	100	29	0.29	38.6
T809	200	33	0.17	38.4
T811	600	57	0.09	29.0
T812	300	73	0.24	33.0

TABLE 5.3 - Values of Undrained Modulus E_{50}

Specimen No.	σ'_{cons} (MPa)	s_u (MPa)	E_{50} (MPa)	E_{50}/s_u	$E_{50}/\sigma'_{\text{cons}}$	$G_{50}/\sigma'_{\text{cons}}$ ⁽¹⁾
T901	0.6	0.18	-	-	-	-
T902	1.0	0.21	44	210	44	15
T903	1.5	0.32	110	343	73	24
T905	1.0	0.32	55	172	55	18
T907 ⁽²⁾	1.5	0.32	71	223	48	16
T908	1.2	0.29	59	202	49	16
T909	0.8	0.22	38	173	48	16
T912	1.5	0.32	67	208	45	15
T913	3.0	0.56	40	79	14	5
T914	2.2	0.54	108	99	49	16
T919	1.5	0.36	147	408	98	33
T920	3.0	0.72	130	181	43	14
T921	2.2	0.54	94	174	43	14
T922	6.0	1.15	235	204	39	13
T923	1.5	0.34	145	426	97	32
T924	3.0	0.88	123	140	41	14
T925	8.8	0.88	123	140	14	5
T926	0.2	0.11	16	143	79	26
T927	1.5	0.52	59	114	39	13
T928	6.0	1.01	265	263	44	15
T929	2.65	0.61	175	287	58	19
T930	9.0	1.03	227	220	25	8
T932	1.5	0.57	75	132	50	17
T933	3.0	0.75	101	135	34	11
T935	9.0	1.03	194	189	22	7

Compaction Density

	Mean $\pm$ St. Dev.
Low Density	54 $\pm$ 24 18 $\pm$ 8
High Density	46 $\pm$ 14 15 $\pm$ 5
All Density	49 $\pm$ 21 16 $\pm$ 7

(1) $G_{50} = E_{50}/2(1 + \nu)$. In undrained tests $\nu = 0.5$, so $G_{50} = E_{50}/3$.

(2) Confining pressure 3 MPa for 15 days, subsequently reduced to 1.5 MPa for 13 days.

TABLE 6.1 - Pseudo-Elastic Parameters (3 days results)

(a) Low Density Buffer

σ'_{cons} (MPa)	K (MPa)	G_{50} (MPa)	$K/\bar{\sigma}_{\text{cons}}$	$G_{50}/\sigma_{\text{cons}}$
0.2	5	2	29	9
0.4	-	5	-	12
0.6	32	6	74	10
0.8	-	15	-	18
1.0	23	10	27	10
1.2	-	12	-	10
1.5	53	20	40	14
2.2	60	23	31	10
3.0	66	19	25	6
9.0	251	-	32	-
Avg. $\pm$ St. Dev.		12 $\pm$ 7	37 $\pm$ 17	11 $\pm$ 3

(b) High Density Buffer

σ'_{cons} (MPa)	K (MPa)	G_{50} (MPa)	$K/\bar{\sigma}_{\text{cons}}$	$G_{50}/\sigma_{\text{cons}}$
3.0	141	55	54	18
9.0	333	-	42	-
Avg. $\pm$ Std. Dev.		55	48 $\pm$ 2	18

TABLE 6.2 - Secondary Consolidation Coefficient for Long Duration Tests

Specimen No.	σ_{cons} (MPa)	U (%)	$C_{\alpha e}$	$C_{\alpha \epsilon}$	Consolidation Duration (Days)	ϵ_v (%)	$C_{ce}^{(1)}$	$C_{\alpha e}/C_{ce}$
T906	0.6	>90	0.07	0.04	12	6.1	0.61	0.12
T910	1.2	<<90	0.10	0.07	18	6.9	0.57	0.19
T911	0.8	>90	0.05	0.03	18	4.9	0.59	0.09
T913	3.0	>98	0.06	0.04	16	8.2	0.52	0.13
T915 ⁽²⁾	3.0	90	0.07	0.04	60	8.7	0.52	0.14
T916	2.2	>98	0.06	0.03	51	7.8	0.53	0.11
T917 ⁽²⁾	2.2	<90	0.05	0.03	46	7.8	0.53	0.09
T918	3.0	>90	0.07	0.04	16	6.3	0.52	0.13
T934	9.0	<90	0.06	0.04	23	4.8	0.34	0.19
T935	9.0	>98	0.09	0.05	35	13.1	0.46	0.21
Average								0.13
Std. Dev.								± 0.05

(1) Isotropic Incremental Consolidation

(2) Using the Measured Swelling Equilibrium Lines:

For Low Density Buffer $\text{LN}(V_c) = -0.099 \text{ LN}(P') + 1.609$

For High Density Buffer $\text{LN}(V_c) = -0.075 \text{ LN}(P') + 1.367$

TABLE 6.3 - Compressibility and Swelling Indices from Specimen T929

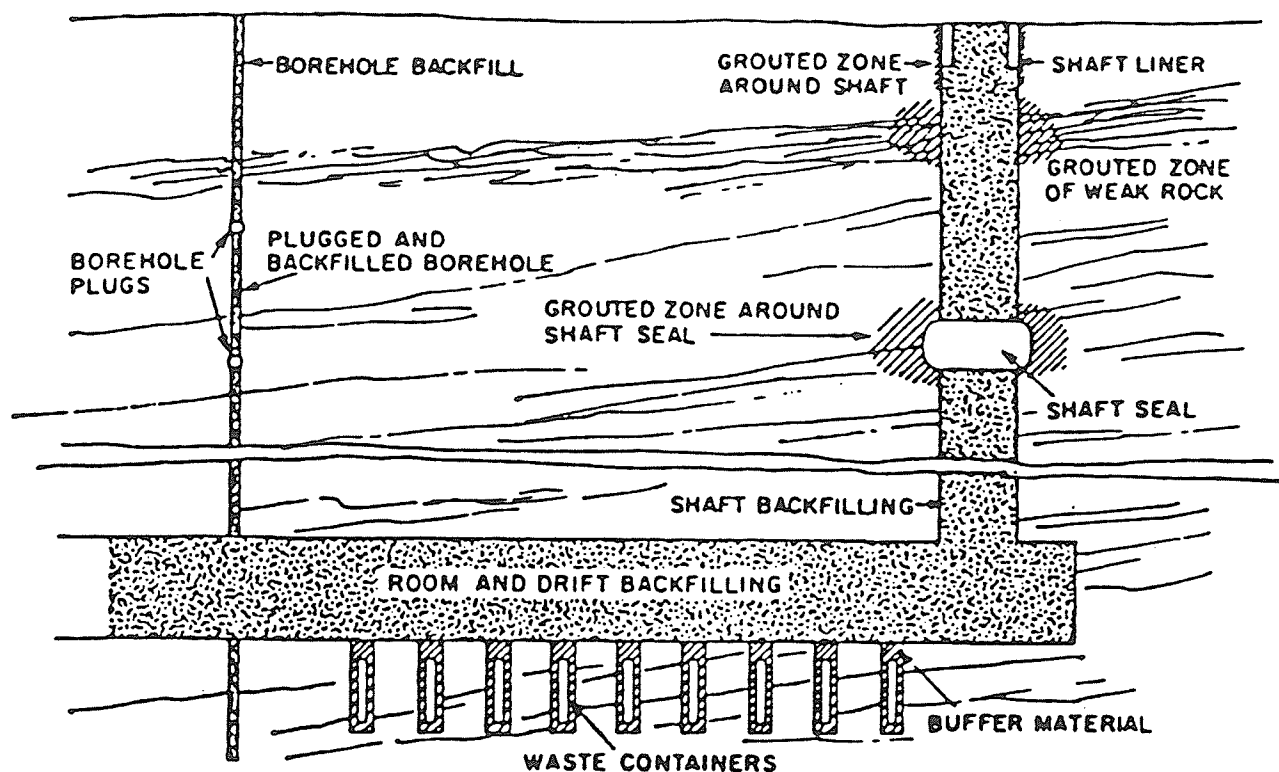
Duration	V_o	λ	R^2	V_o	K	R^2	λ/K
1-Days	1.615	0.017	0.966	1.608	-0.0086	0.996	2.0
3-Days	1.624	0.032	0.952	1.612	-0.0171	0.999	1.9
5-Days	1.631	0.043	0.927	1.612	-0.0215	0.927	2.0

TABLE 6.4 - End-of-Test Water Content Gradients along Compressive and Dilative Specimens

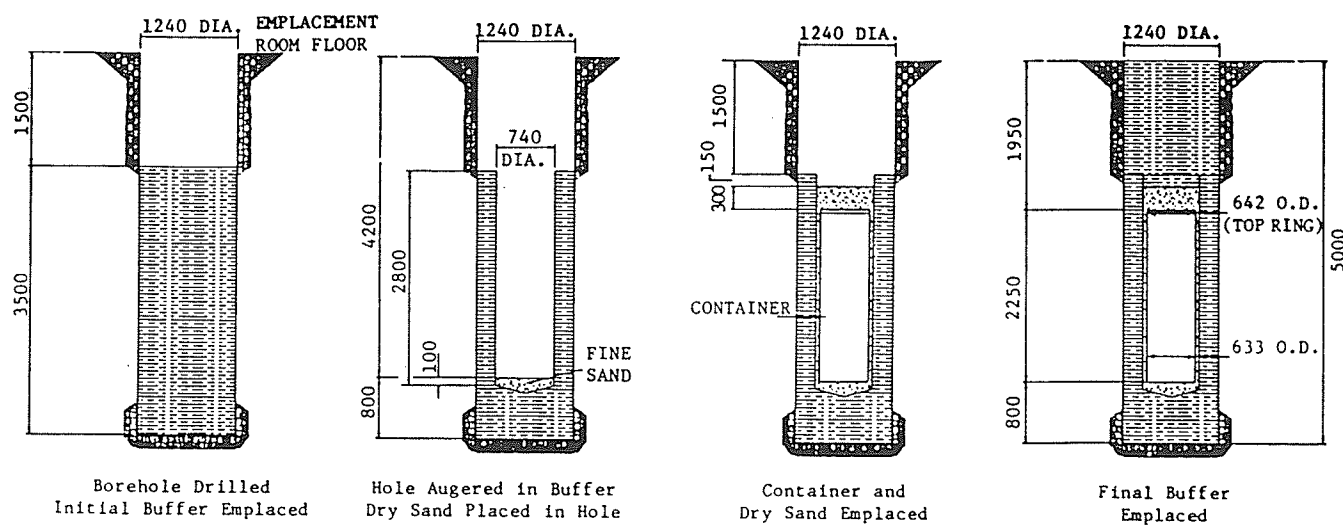
Specimen	Water content (%)		Consolidation Behaviour		Undrained Shearing
Behaviour	Top	Bottom	Exp./Comp.	Duration (Days)	Duration (Days)
<u>Compressive</u>					
T901	29.6	29.6	C	31	0.5
T902	28.0	28.0	C	12	0.5
T903	25.5	25.5	C	28	0.5
T904	24.9	25.0	C	31	-
T905	27.8	27.4	C	57	0.5
T906	26.9	26.9	C	12	-
T907	26.0	24.3	C	15	0.5
T908	27.6	27.6	C	16	0.5
T909	29.2	29.1	C	15	0.5
T910	26.1	26.0	C	18	-
T911	28.3	28.2	C	18	-
T912	26.0	25.9	C	25	0.5
T913	23.5	22.4	C	16	0.5
T914	23.2	23.1	C	9	0.5
T915	22.2	22.3	C	60	2.5
T916	23.6	23.1	C	51	-
T917	23.7	23.8	C	46	-
T918	23.6	22.4	C	16	2.5
T919	26.0	25.5	C	37	2.5
T920	21.1	20.9	C	10	2.5
T921	22.1	22.5	C	13	2.5
T922	21.1	20.5	C	27	6
T923	25.0	24.8	C	41	2.5
T924	19.2	19.2	E	17	2.5
T925	20.1	18.0	C	32	6
T928	19.3	17.9	C	19	6
T929	21.4	21.1	C	51	2.5
T930	19.3	18.5	C	35	2.5
T931	20.5	20.1	C	12	-
T932	21.5	20.9	C	50	-
T933	20.3	20.5	C	15	6
T934	17.9	16.5	C	23	24
T935	19.3	18.5	C	35	20
<u>Dilative</u>					
T926	28.5	30.5	E	24	2.5
T927	21.5	22.4	E	12	2.5
T932	21.5	21.4	C	50	2.5

TABLE 6.5 - Comparison of Buffer Material Constants with other Specimens

Soil (Source)	λ	K	λ/K	$\wedge$
London Clay (Loudon)	0.161	0.062	2.6	0.61
Weald Clay (Parry)	0.093	0.035	2.7	0.63
Kaolin Clay (Parry)	0.26	0.05	5.2	0.81
Winnipeg Clay (Graham)	0.305	0.09	3.4	0.71
Buffer - Low Density	0.128	-	-	-
Buffer - High Density	0.084	0.022	3.8	0.74



(A) SCHEMATIC DIAGRAM OF THE UNDERGROUND VAULT FOR NUCLEAR WASTE DISPOSAL



(B) BOREHOLE EMPLACEMENT SEQUENCE (all dimensions in mm)

FIGURE 1.1 - THE COMPONENTS OF THE CANADIAN CONCEPT FOR NUCLEAR FUEL WASTE MANAGEMENT (AFTER ROSINGER AND DIXON, 1982; KJARTANSON AND GRAY 1987)

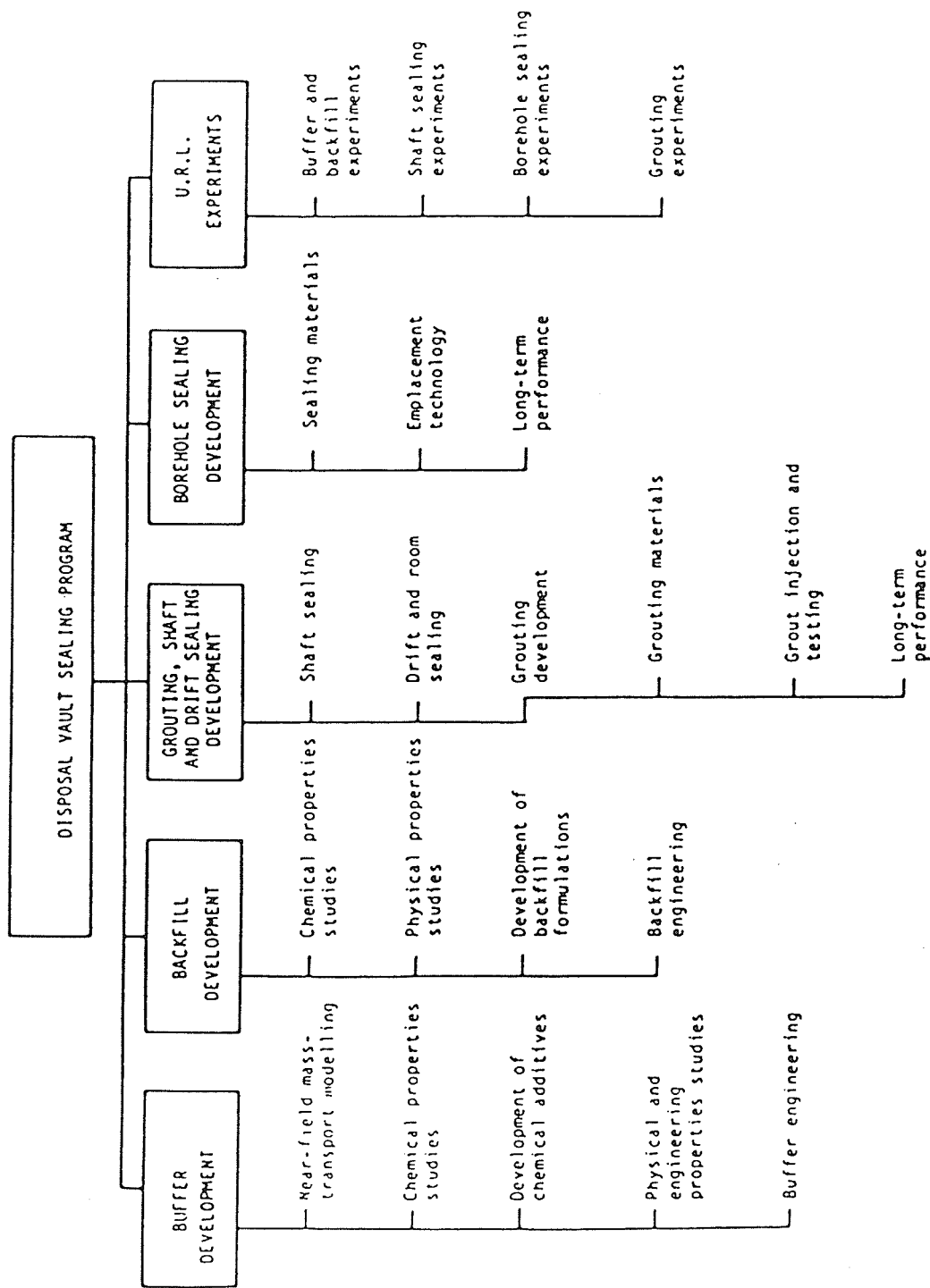
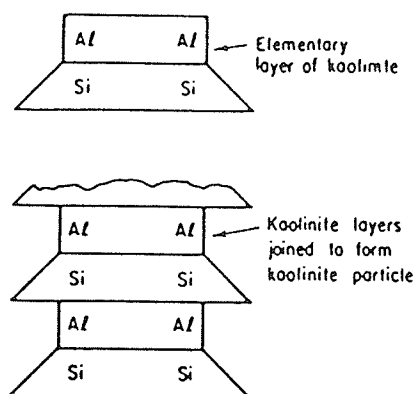
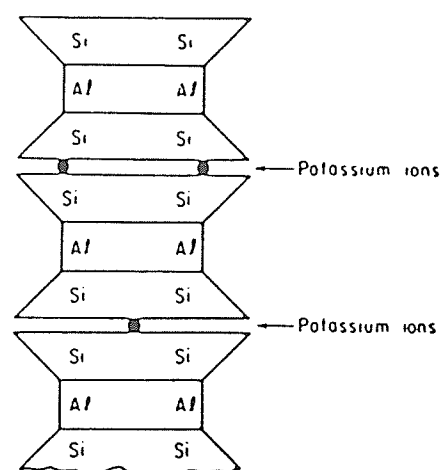


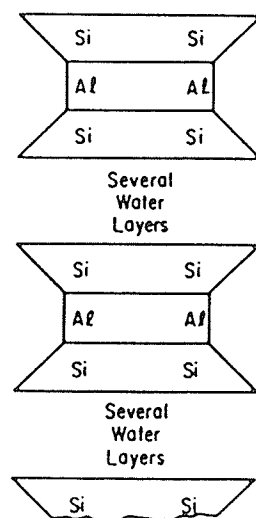
FIGURE 1.2 - ORGANIZATION CHART OF THE VAULT SEALING PROGRAM (AFTER BIRD AND CAMERON, 1982)



(A) KAOLINITE



(B) ILLITE



(C) MONTMORILLONITE

FIGURE 2.1 - STRUCTURE OF CLAY MINERALS (AFTER YONG AND WARKENTIN, 1975)

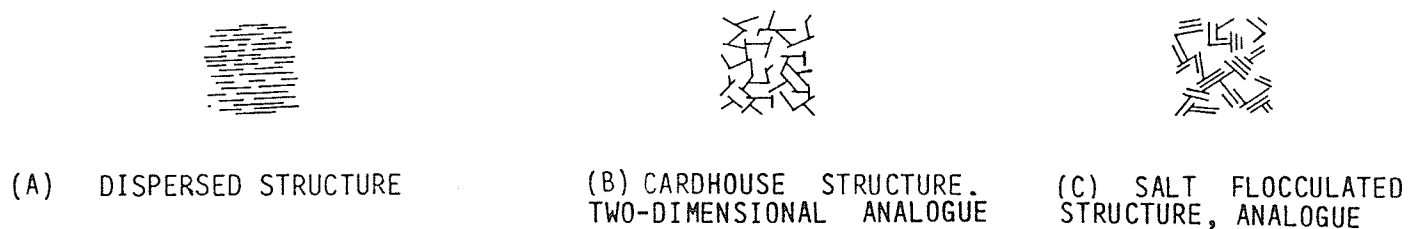


FIGURE 2.2 - STRUCTURAL ARRANGEMENTS OF PARTICLES (AFTER SMART, 1971)

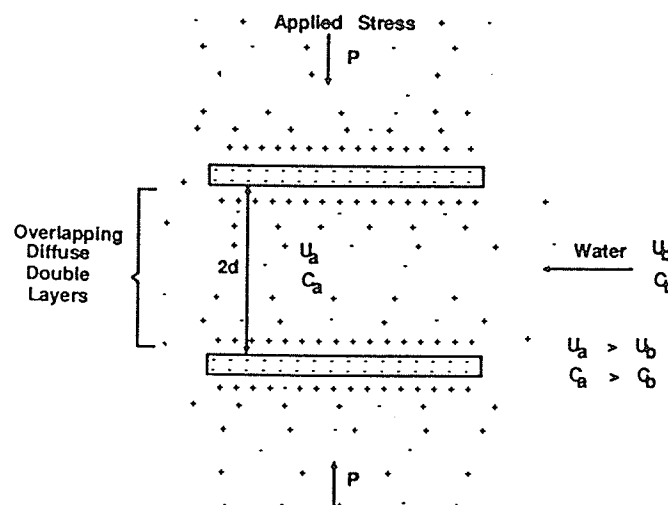


FIGURE 2.3 - ILLUSTRATION OF THE OSMOTIC PRESSURE CONCEPT (AFTER MITCHELL 1976)

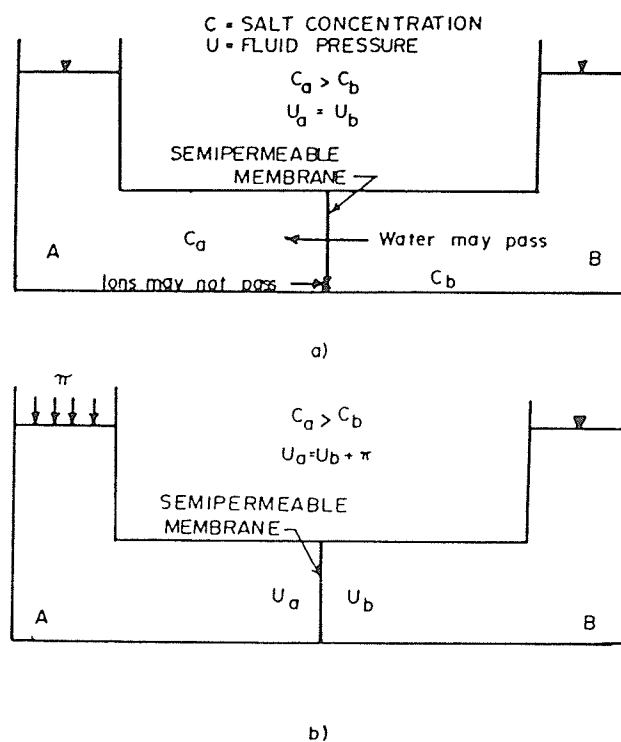


FIGURE 2.4 - SCHEMATIC ILLUSTRATION OF OSMOSIS - (A) OSMOTIC FLOW; (B) OSMOTIC PRESSURE (AFTER MITCHELL 1976)

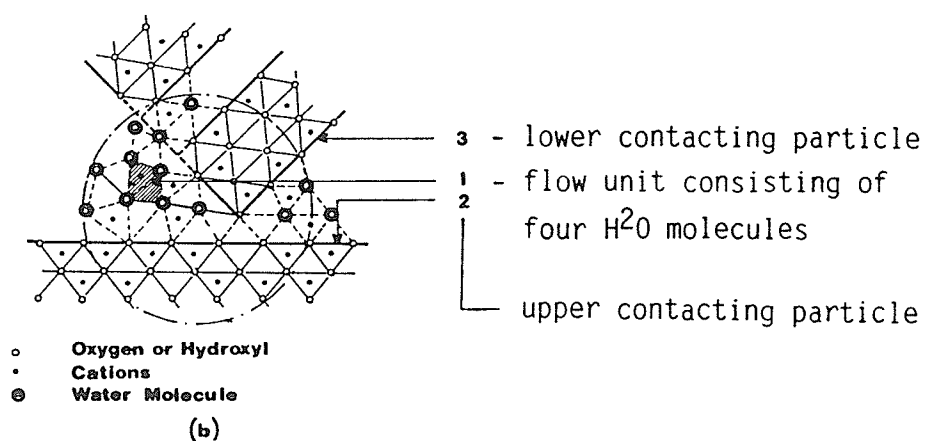
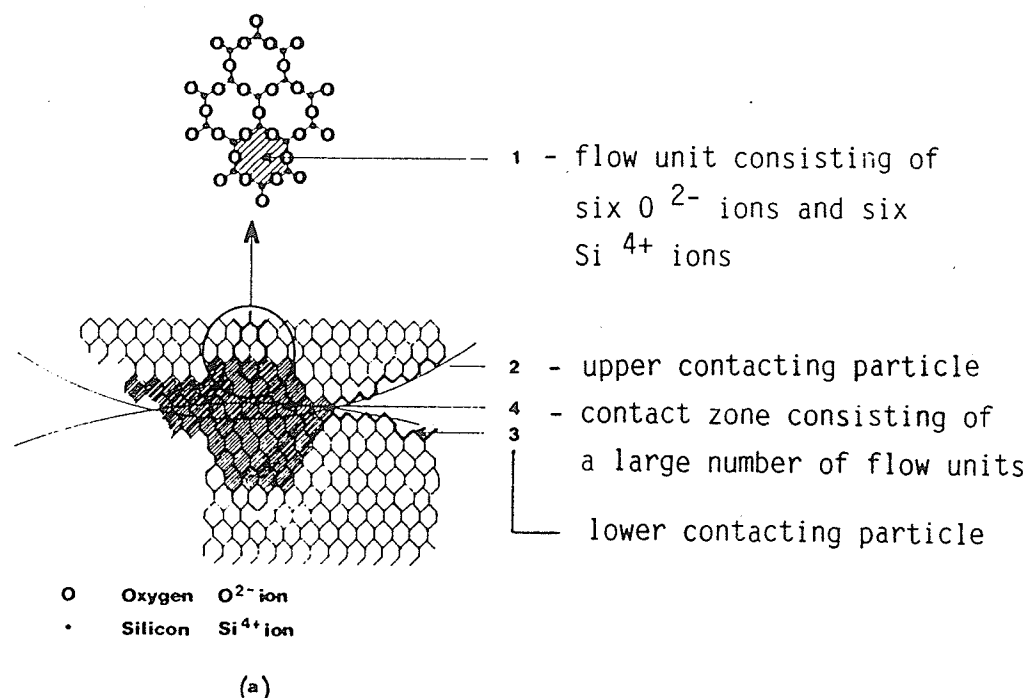


FIGURE 2.5 - TWO-DIMENSIONAL ANALOGUE OF CONTACT ZONES - (A) CONTACT ZONE FORMED BETWEEN TWO QUARTZ SAND PARTICLES; (B) CONTACT ZONE FORMED BETWEEN TWO MONTMORILLONITE CLAY PARTICLES (AFTER ARNOLD, 1967)

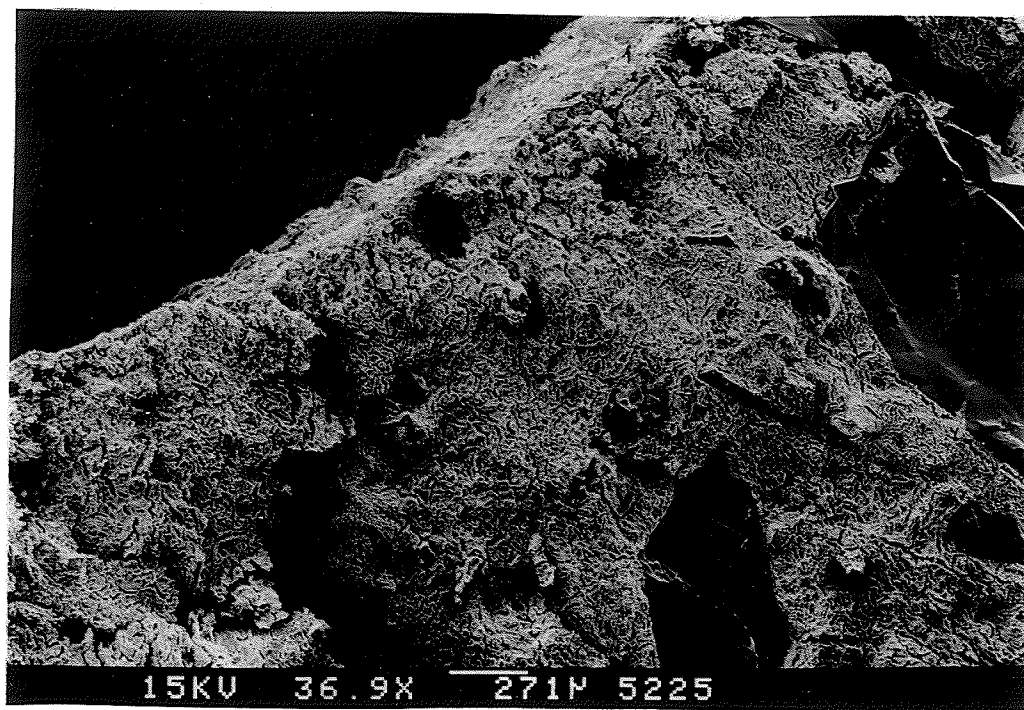
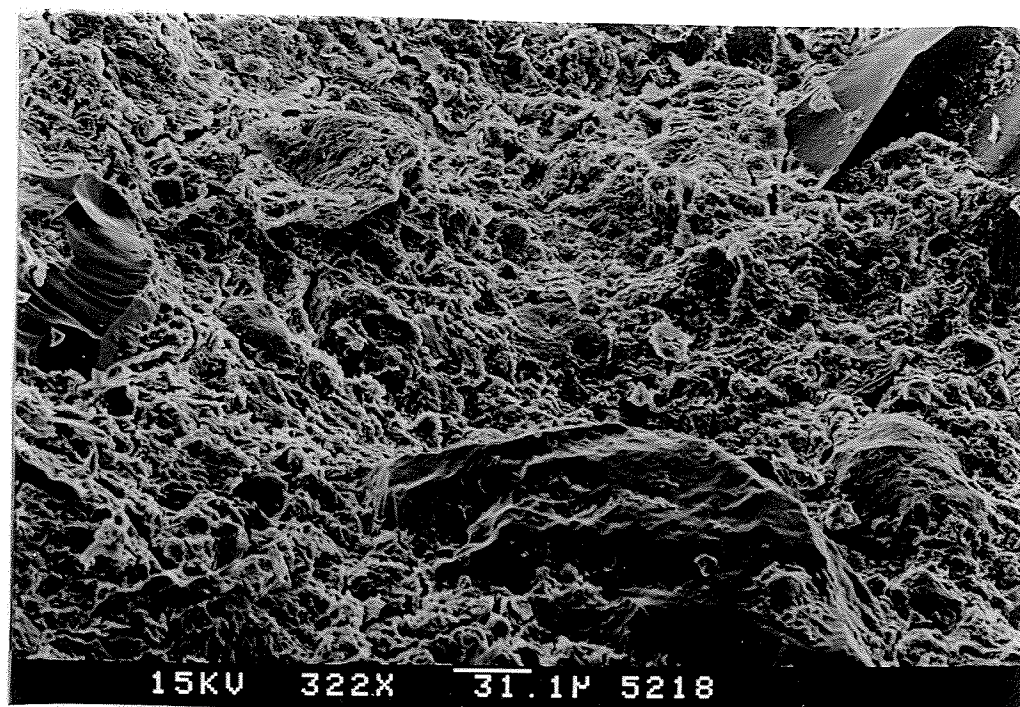
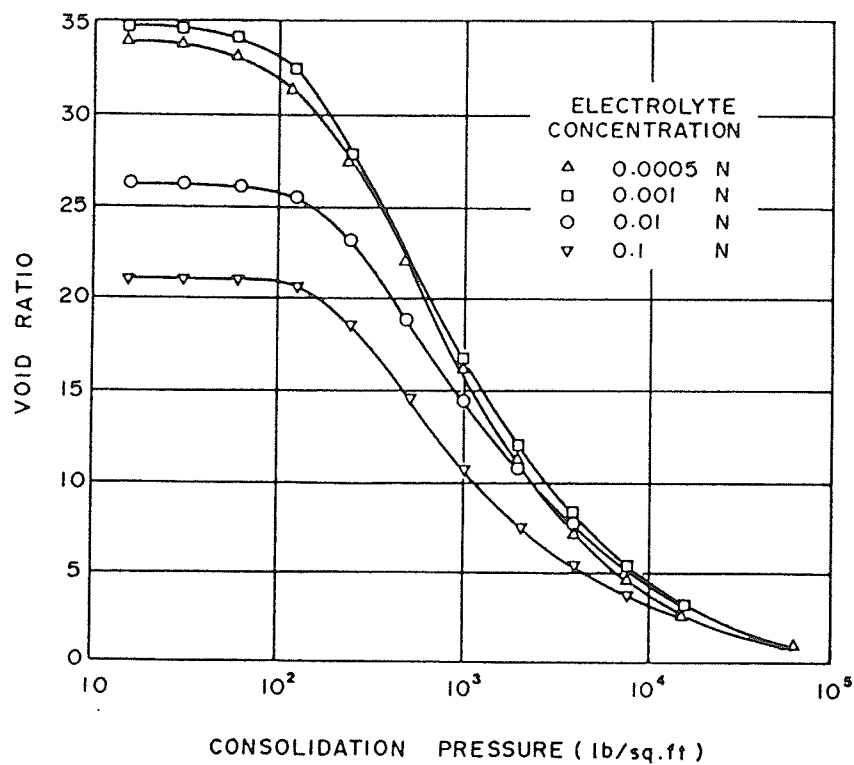
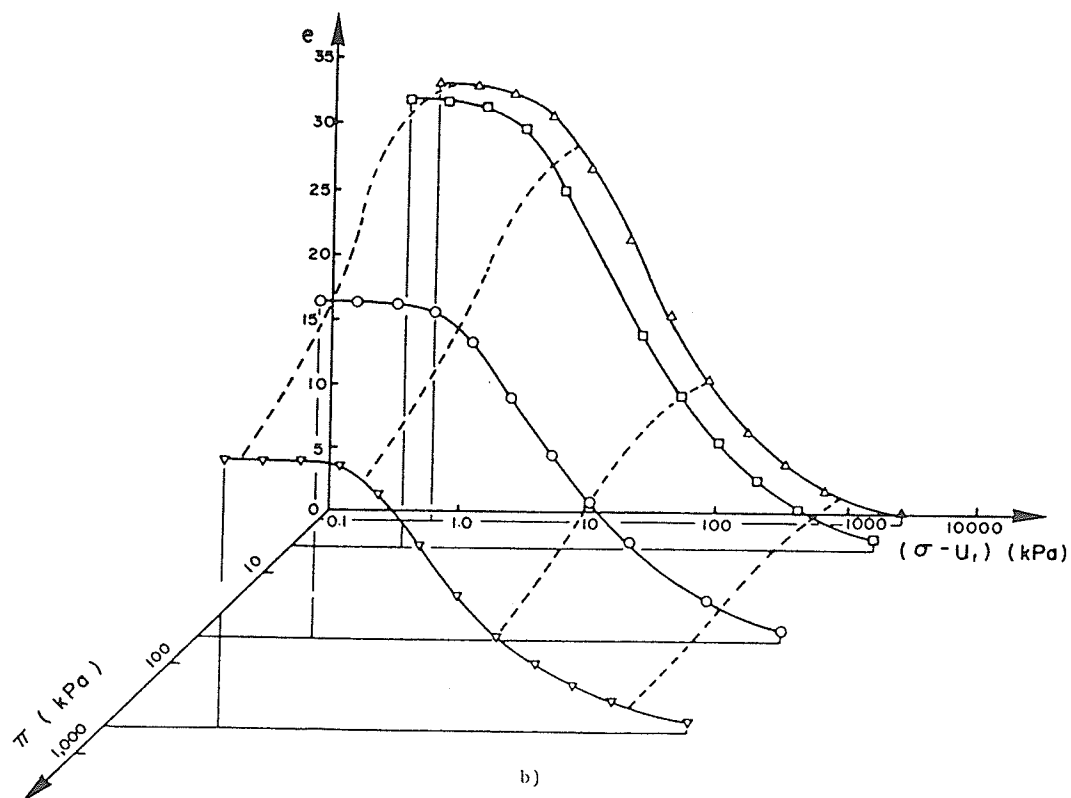


FIGURE 2.6 - SCANNING ELECTRON MICROGRAPHS OF MICROSTRUCTURE OF SAND-BENTONITE BUFFER



a)



b)

FIGURE 2.7 - (A) ONE-DIMENSIONAL CONSOLIDATION CURVES (MESRI AND OLSON 1971)
(B) CONSTITUTIVE SURFACE FOR NA⁺ MONTMORILLONITE (AFTER BARBOUR 1987)

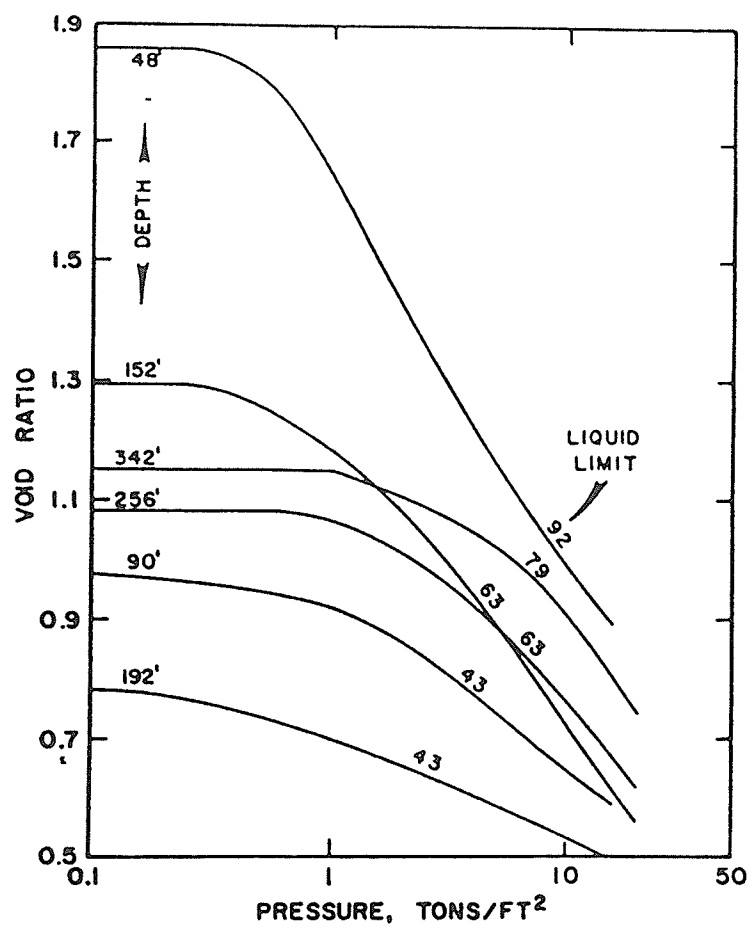


FIGURE 2.8 - PRESSURE - VOID RATIO CURVES (AFTER MCCLELLAND 1967)

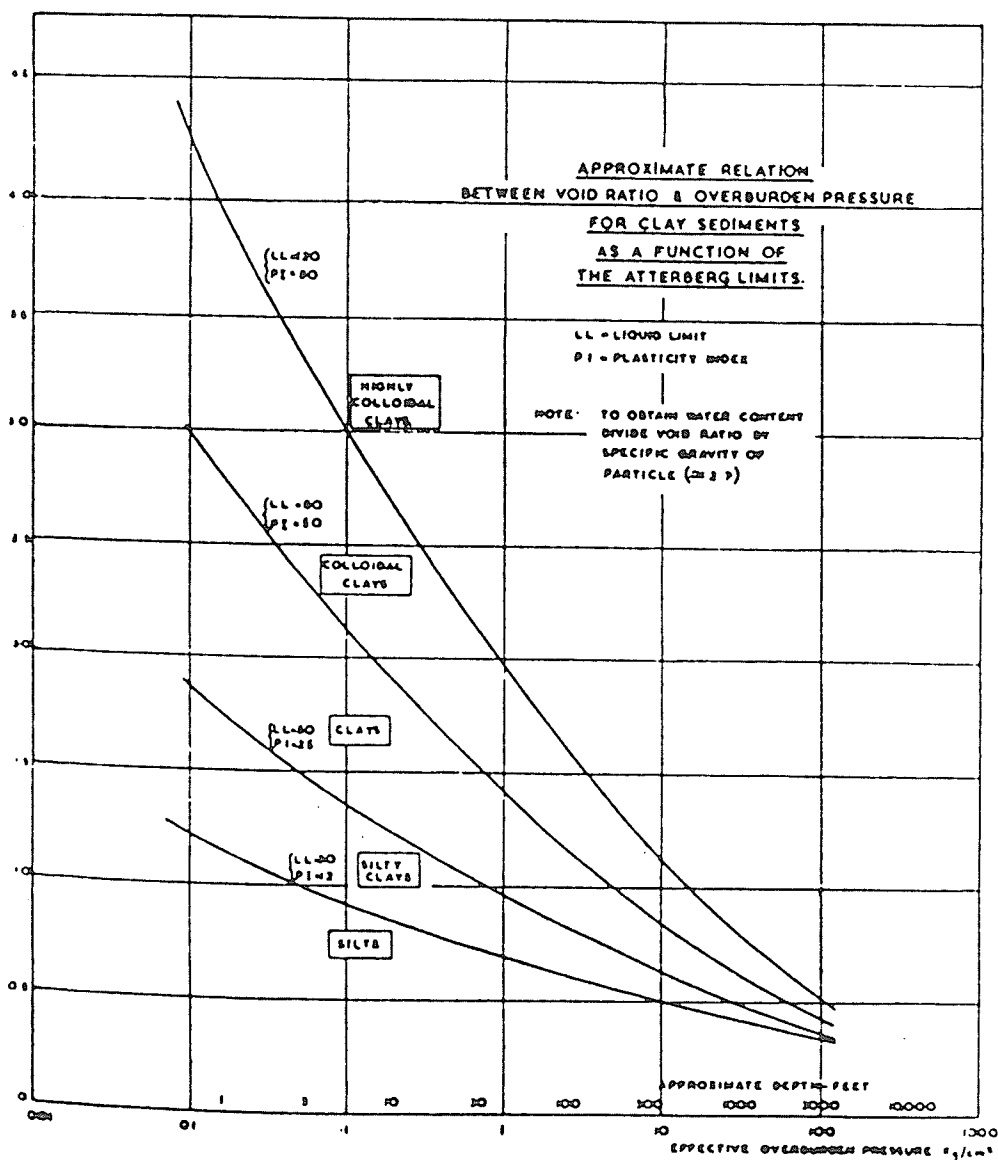


FIGURE 2.9 - GRAPHS OF THE APPROXIMATE RELATION BETWEEN VOID-RATIO AND OVER-BURDEN PRESSURE FOR CLAY SEDIMENTS, AS A FUNCTION OF THE ATTERBERG LIMITS (AFTER SKEMPTON 1953)

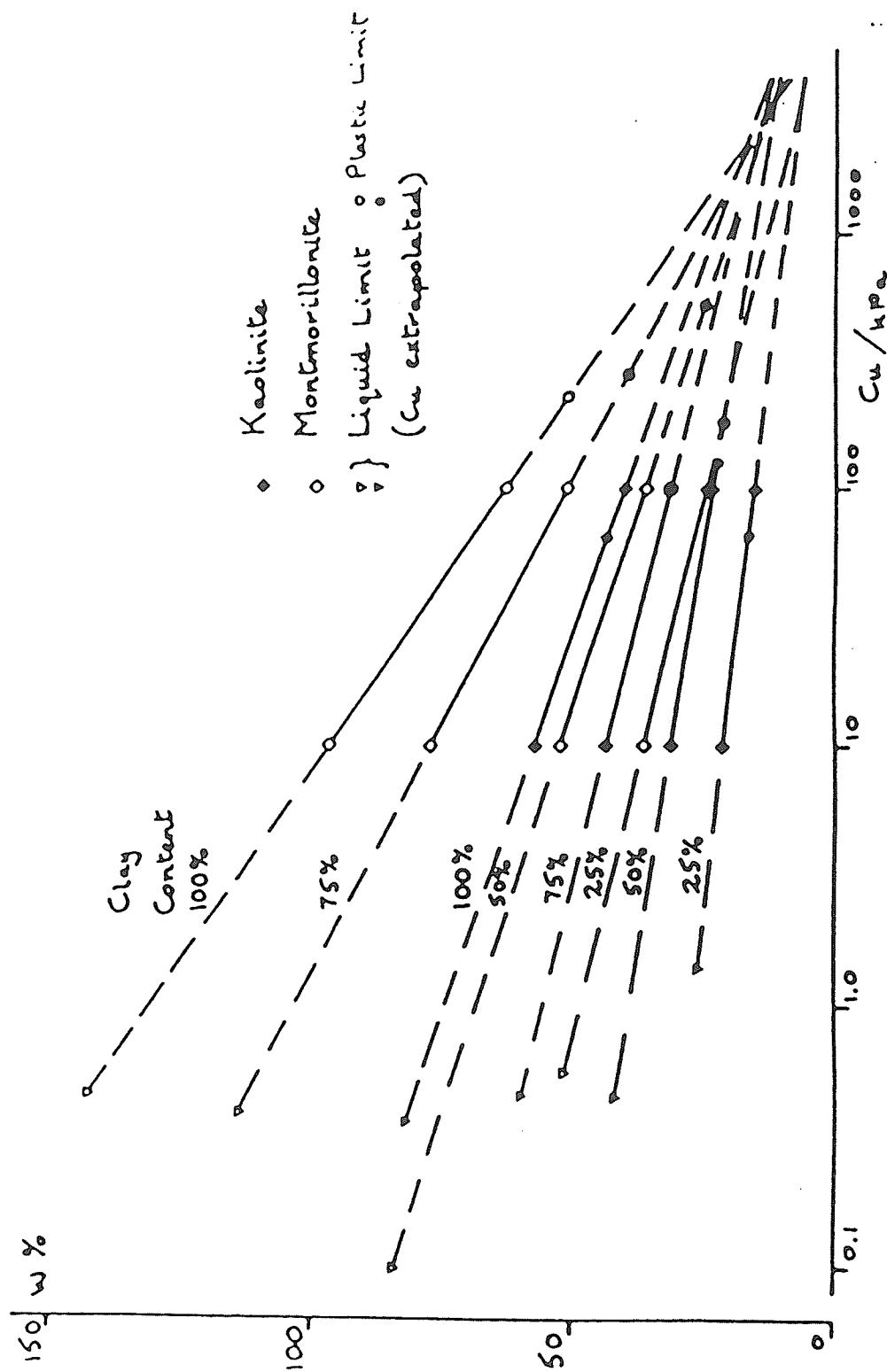


FIGURE 2.10 - INFLUENCE OF CLAY CONTENT ON EQUILIBRIUM WATER CONTENT AND UNDRAINED SHEAR STRENGTH OF SAND-CLAY MIXTURES (DUMBLETON AND WEST, 1970)

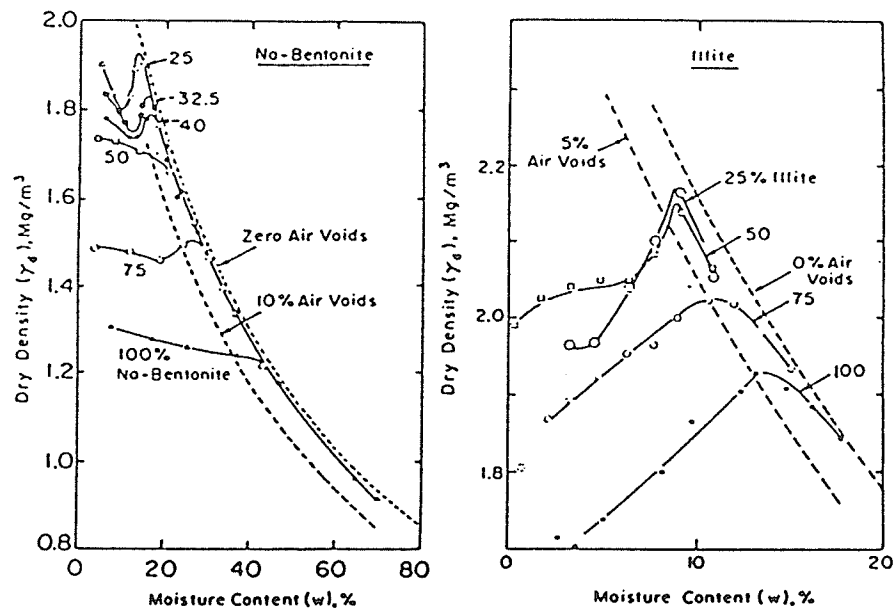


FIGURE 2.11 - DRY DENSITY - WATER CONTENT RELATIONSHIPS (AFTER DIXON ET AL. 1985)

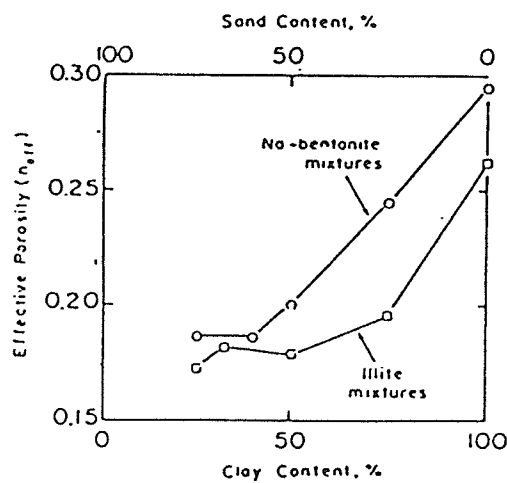


FIGURE 2.12 - INFLUENCE OF SAND CONTENT ON THE EFFECTIVE POROSITY OF CLAY-SAND MIXTURES (AFTER DIXON ET AL. 1985)

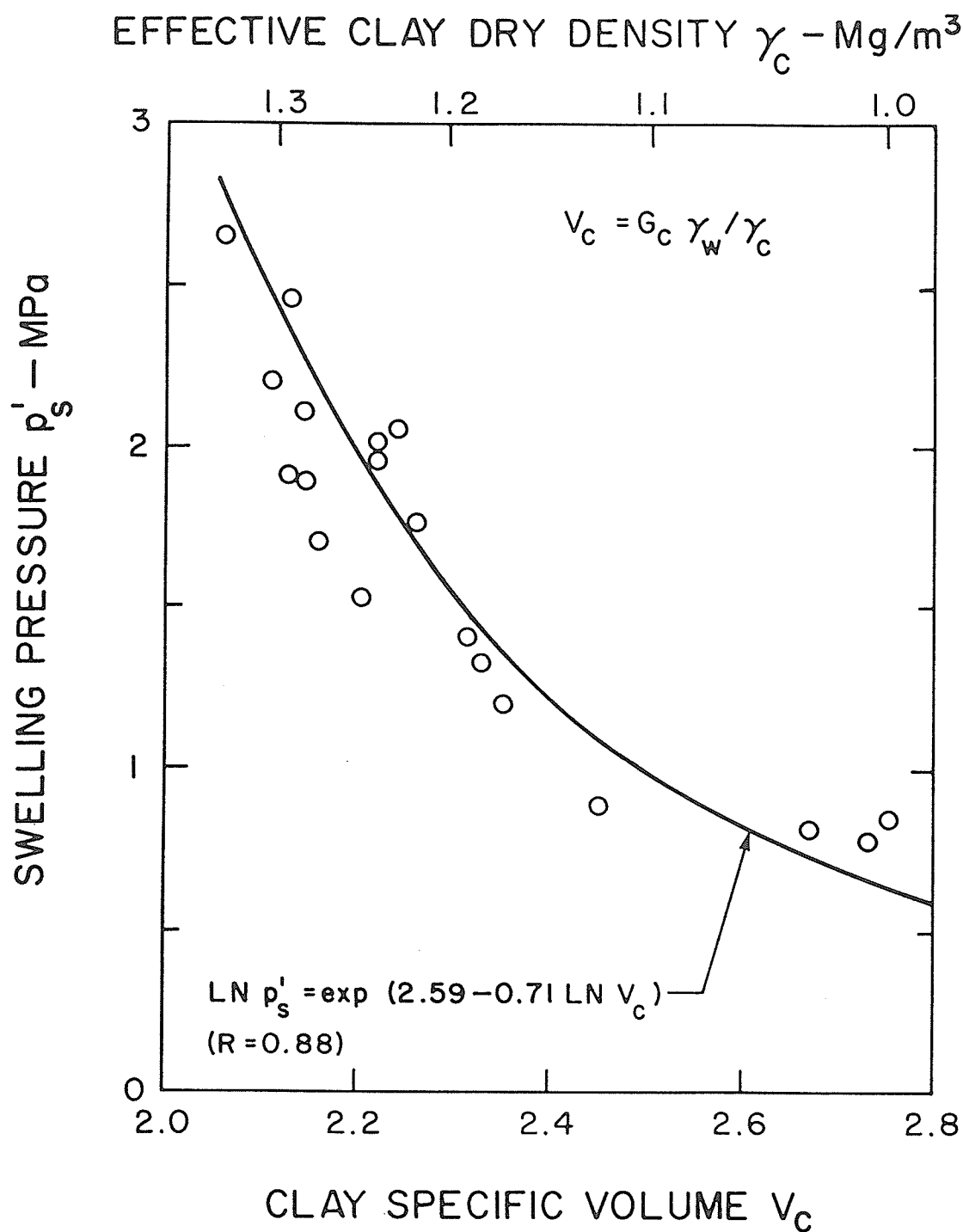


FIGURE 2.13 - SWELLING PRESSURE VERSUS EFFECTIVE CLAY DENSITY (CLAY SPECIFIC VOLUME IN CONFINED 1-D TESTS (GRAHAM ET AL., 1986))

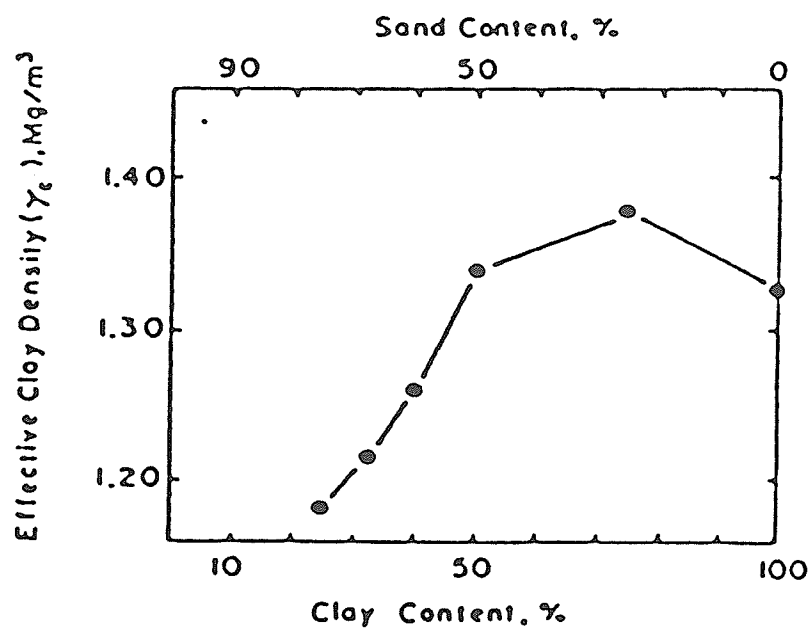


FIGURE 2.14 - INFLUNENCE OF SAND CONTENT ON THE EFFECTIVE CLAY DENSITY OF BENTONITE - SAND MIXTURES (AFTER DIXON ET AL. 1985)

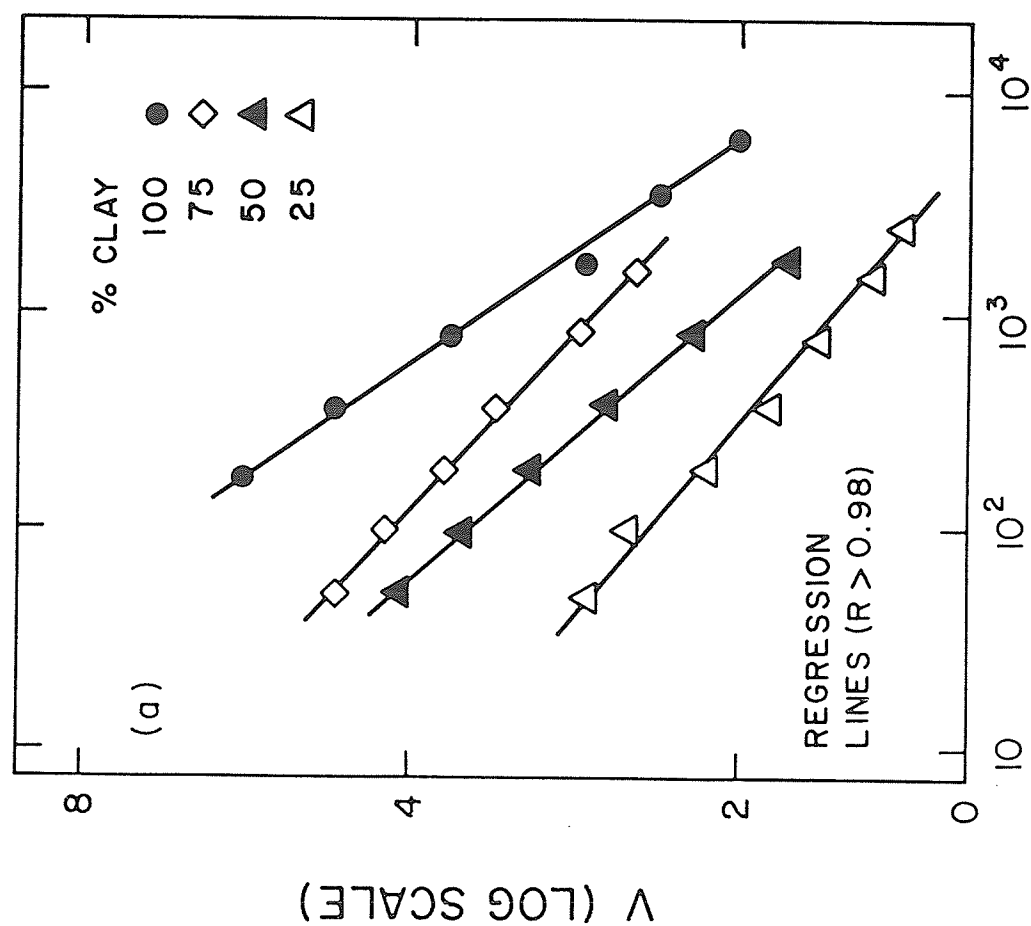
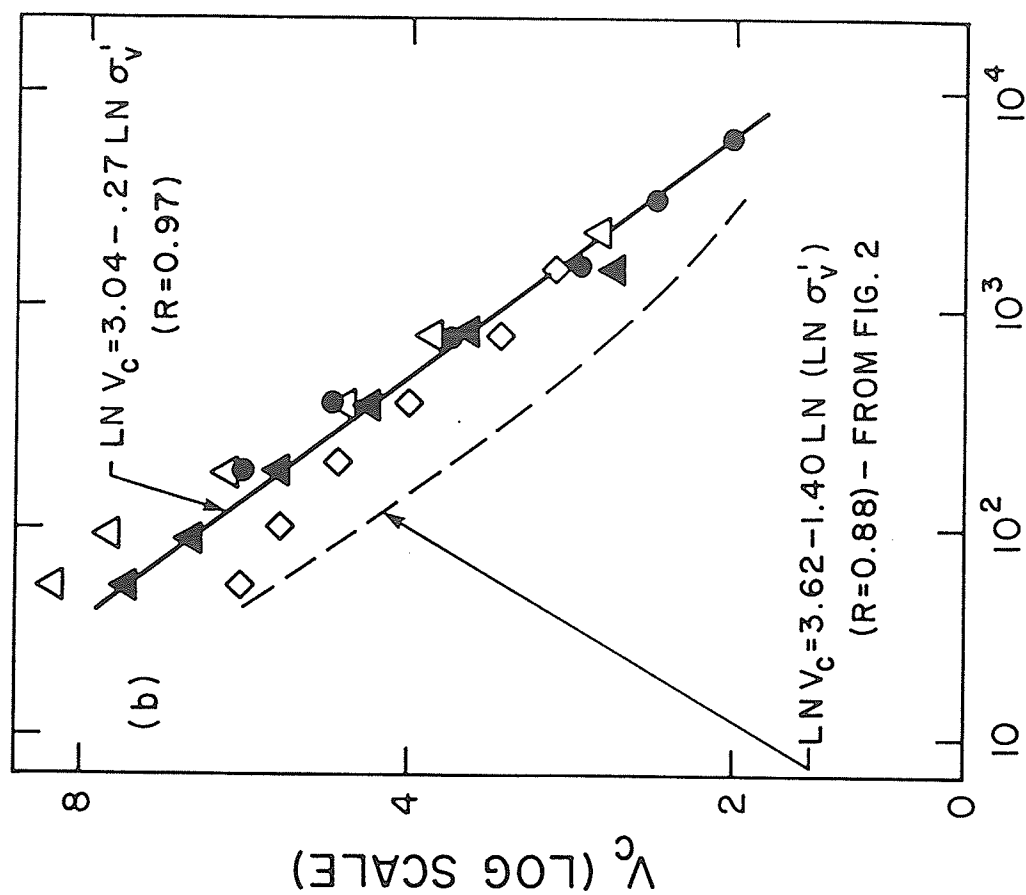

 $\sigma'_v - \text{kPa (LOG SCALE)}$

 $\sigma'_v - \text{kPa (LOG SCALE)}$

FIGURE 2.15 - (A) SPECIFIC VOLUME, AND (B) CLAY SPECIFIC VOLUME VERSUS VERTICAL PRESSURE IN 1-D COMPRESSION TESTS (GRAHAM ET AL. 1986)

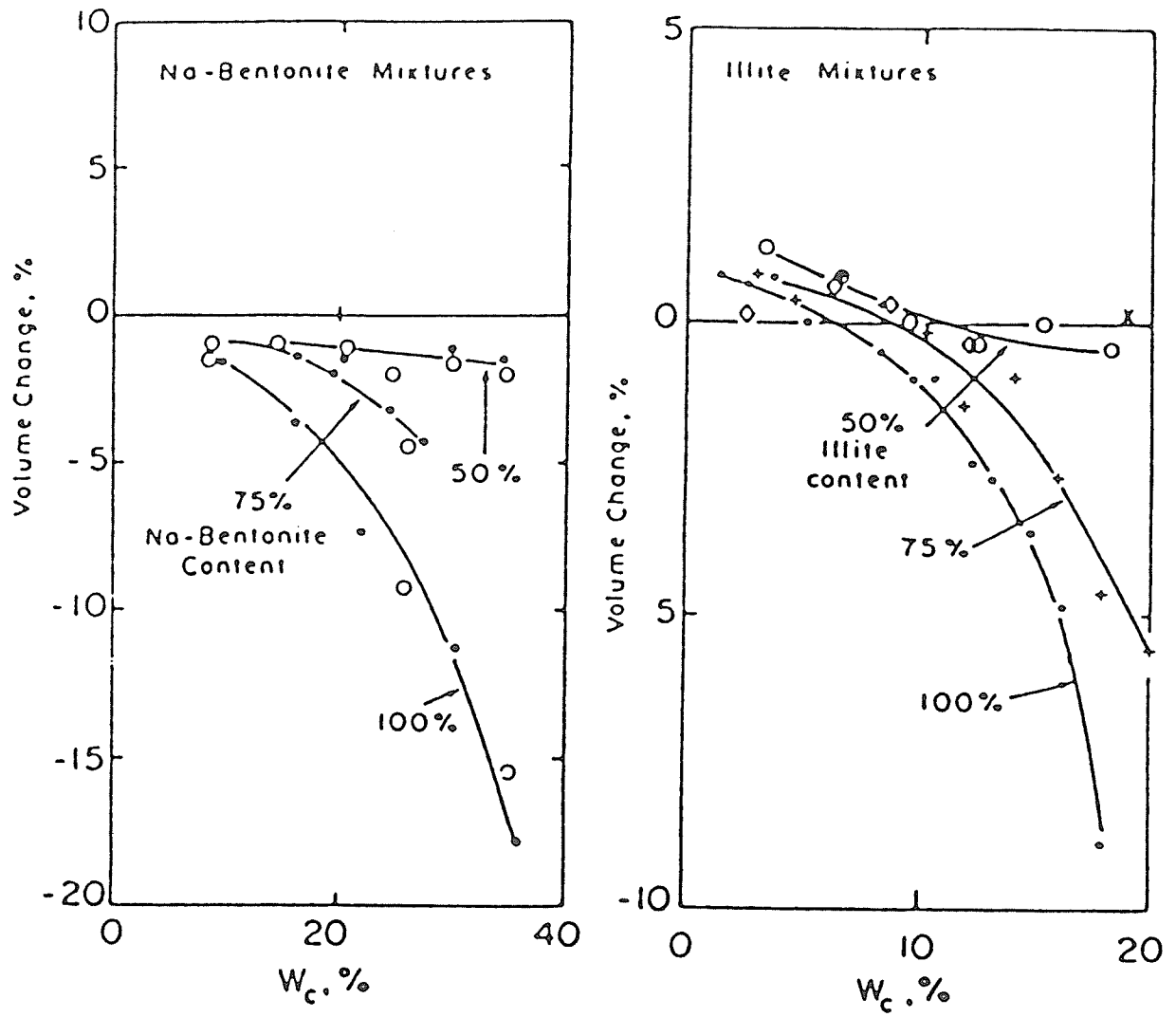


FIGURE 2.16 - VOLUME CHANGE OF COMPACTED CLAY - SAND MIXTURES (AFTER DIXON ET AL. 1985)

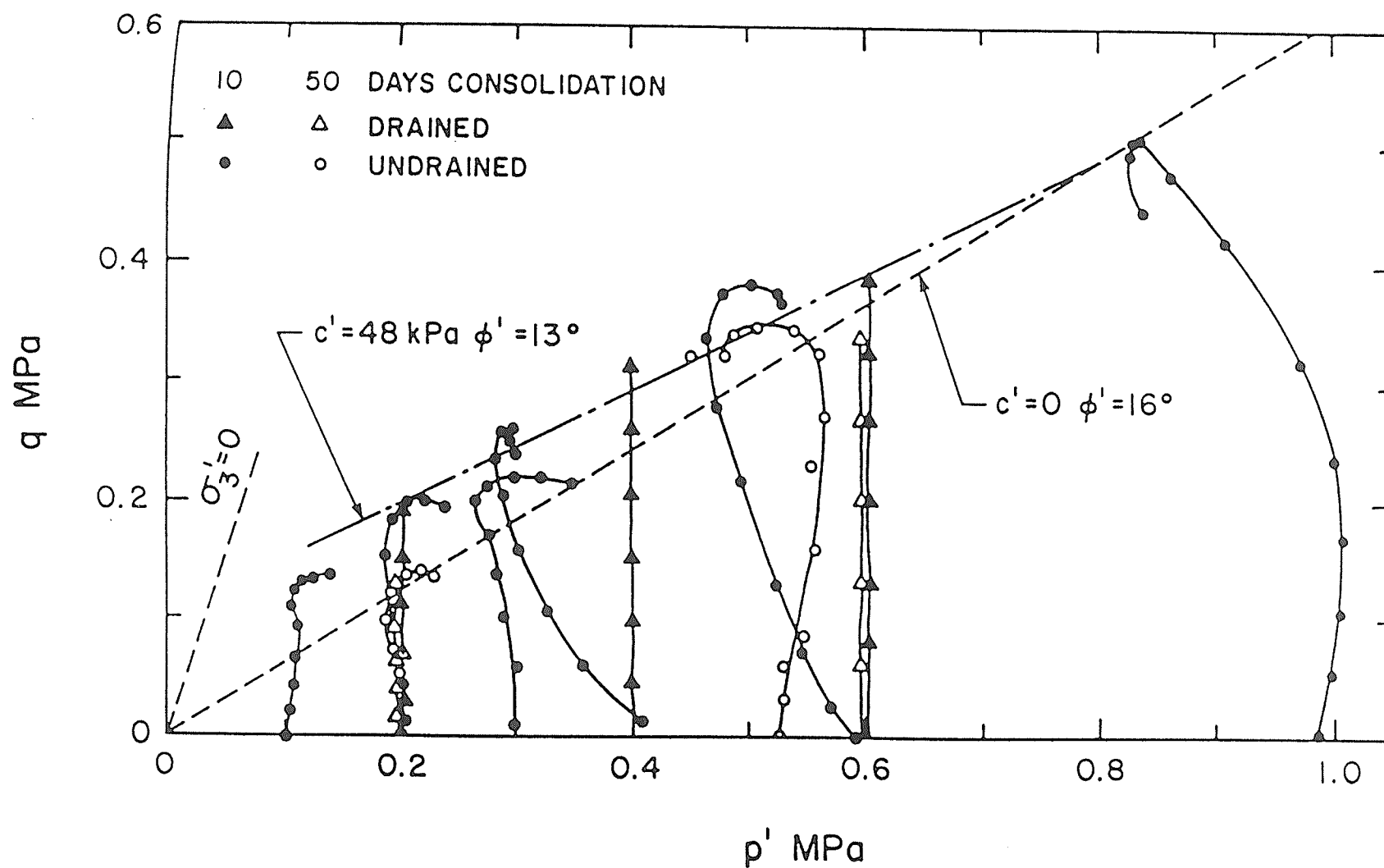


FIGURE 2.17 -SHEAR STRENGTH ENVELOPE FROM 800-SERIES (AFTER SUN, 1986)

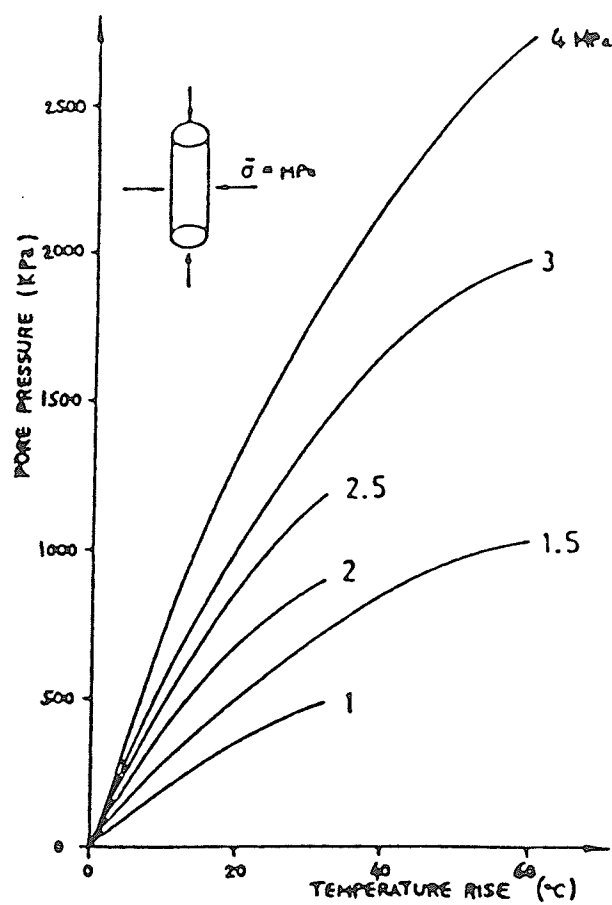


FIGURE 2.18 - PORE WATER PRESSURE RISE UNDER HEATING IN UNDRAINED ORTE CLAY SAMPLE UNDER VARIOUS CONSTANT TOTAL STRESS (AFTER BALDI ET AL. 1985)

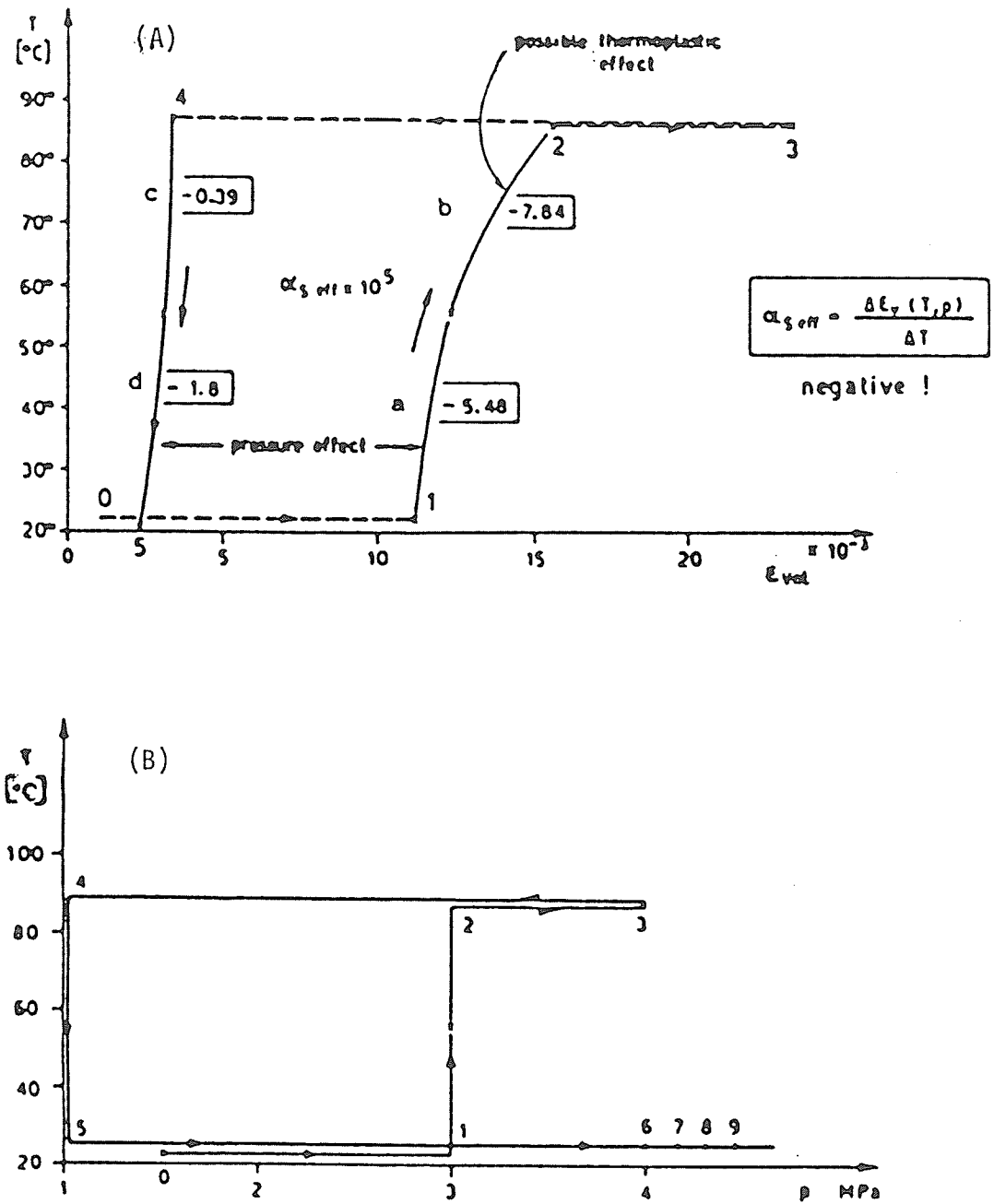


FIGURE 2.19 - (A) TEMPERATURE - PRESSURE VARIATIONS

(B) THERMAL STRAINS IN BOOM CLAY (AFTER BALDI ET AL. 1985)

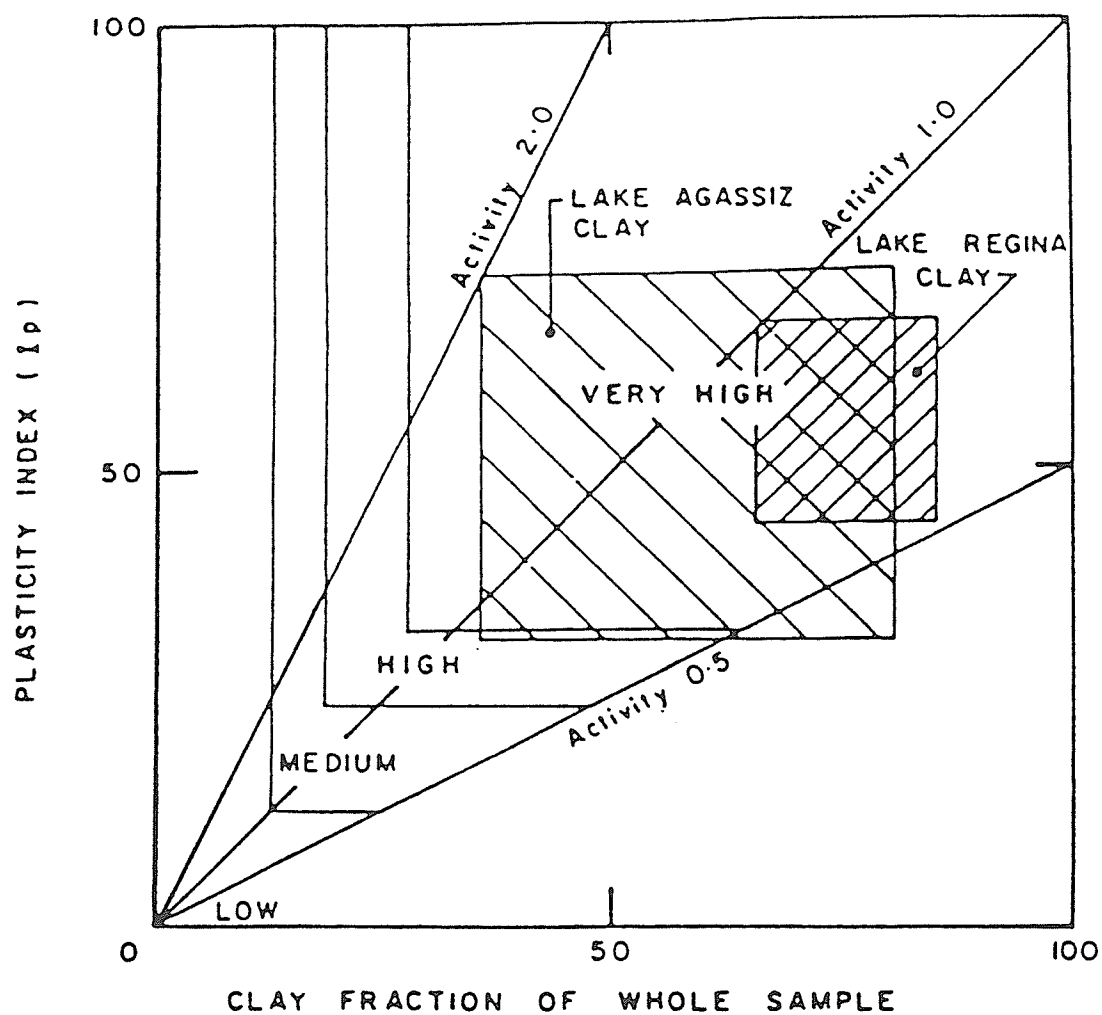


FIGURE 2.20 - VOLUME CHANGE POTENTIAL FOR WINNIPEG AND REGINA CLAYS
(AFTER HAMILTON, 1980)

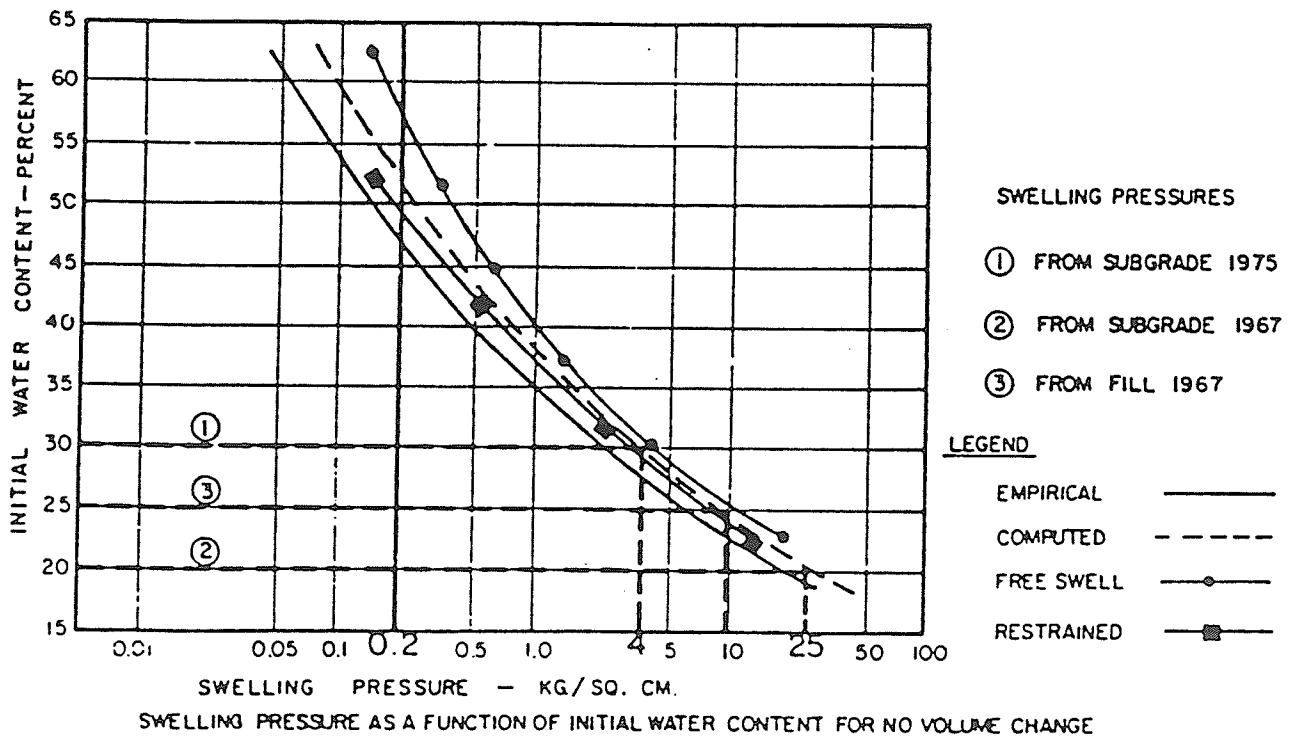
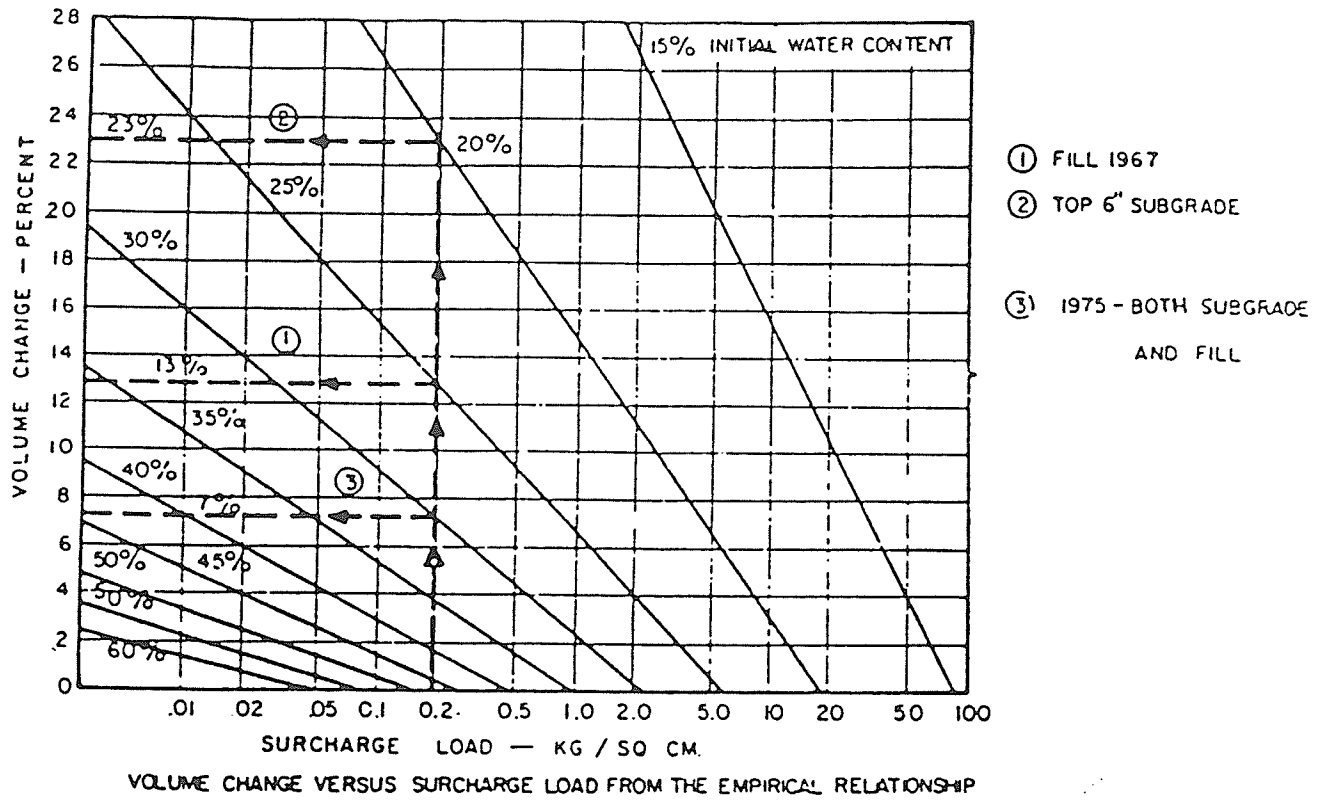


FIGURE 2.21 - SWELLING PRESSURE AS A FUNCTION OF INITIAL WATER CONTENT

(AFTER NOBLE 1976)

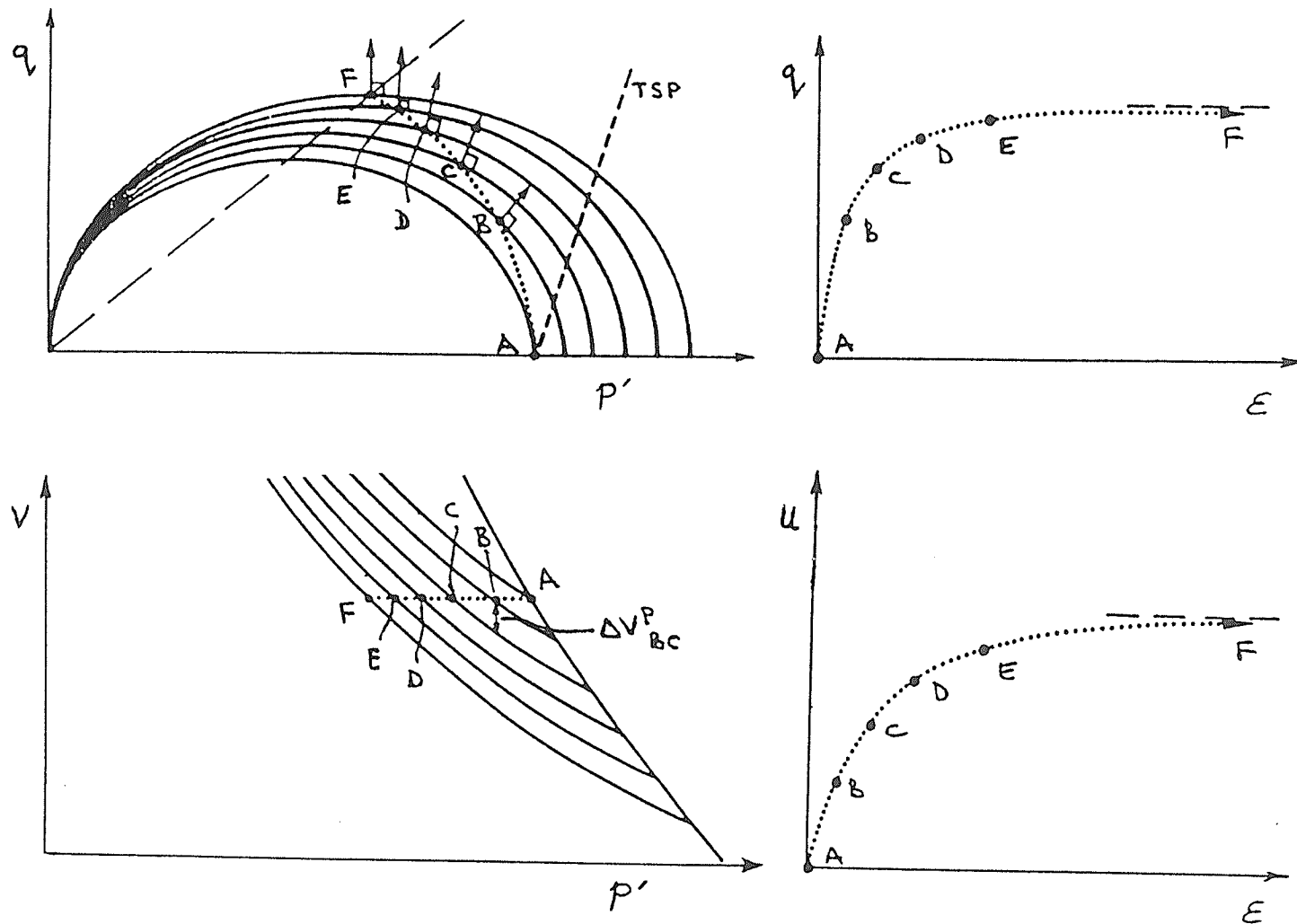


FIGURE 2.23 - YIELDING OF NORMALLY CONSOLIDATED (LIGHTLY OVERCONSOLIDATED) CLAY IN UNDRAINED SHEAR (AFTER WOOD 1986)

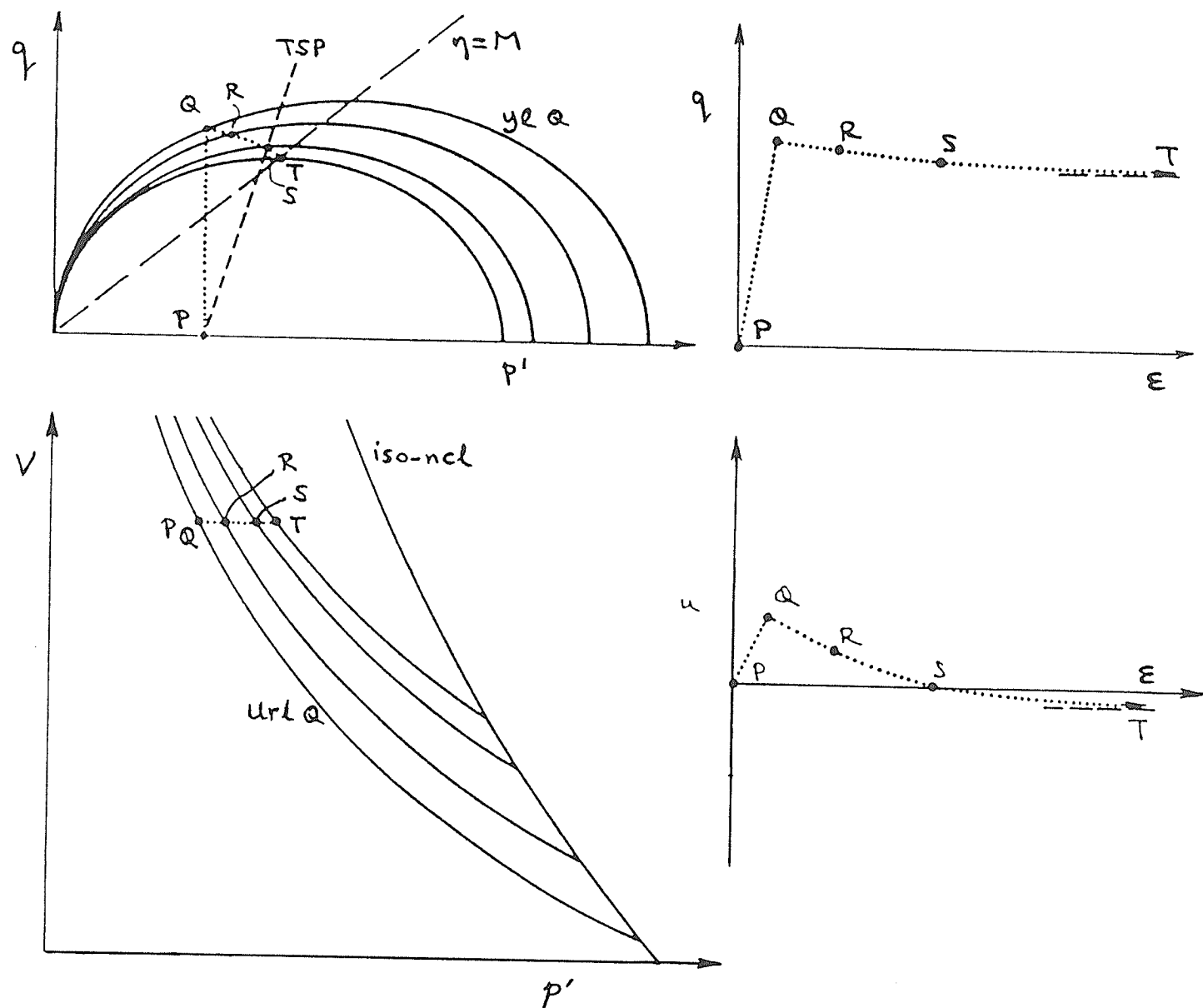


FIGURE 2.24 - YIELDING OF HEAVILY OVERCONSOLIDATED CLAY IN UNDRAINED SHEAR (AFTER WOOD 1986)

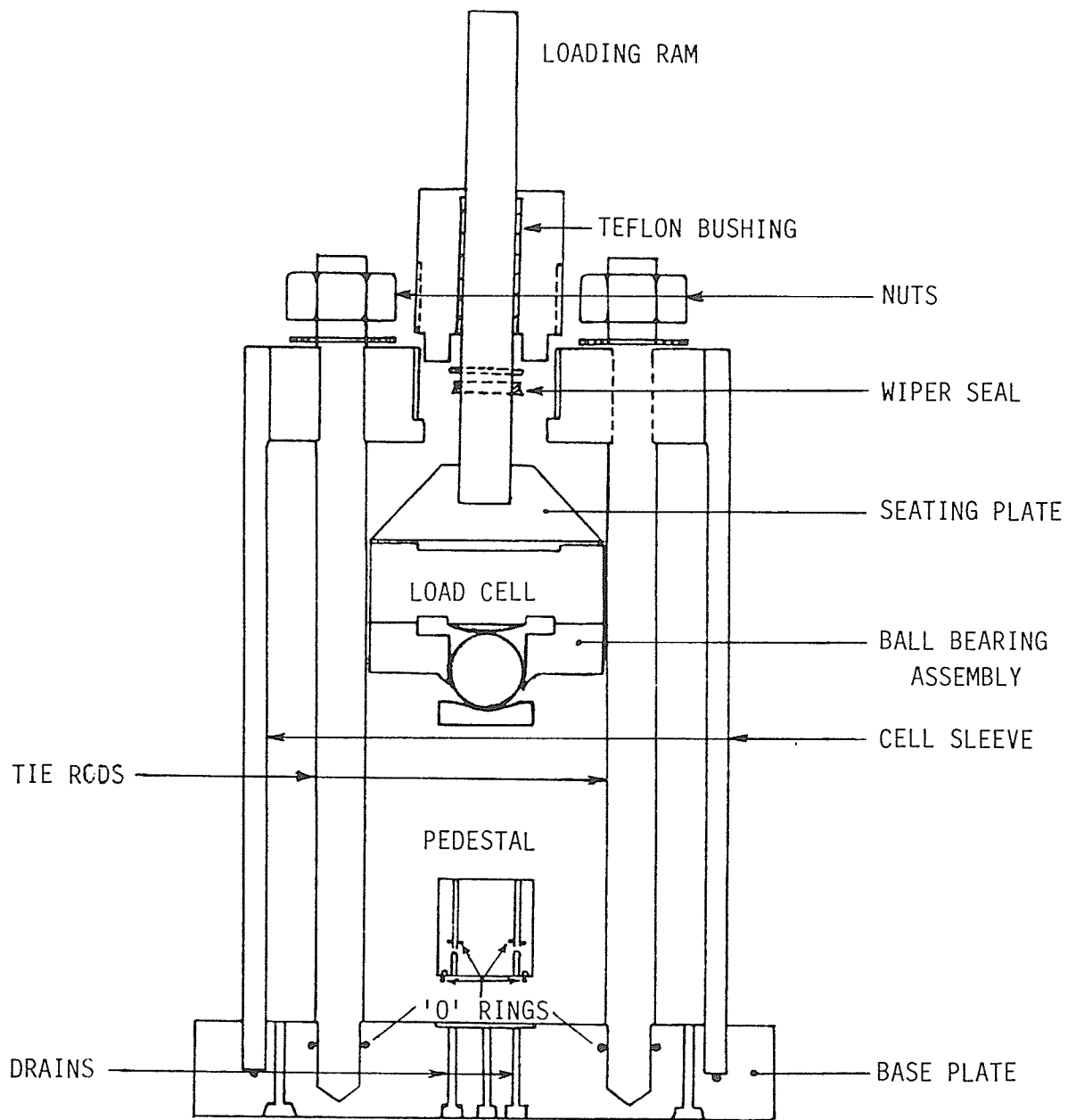


FIGURE 3.1 - DETAILED DESIGN OF HIGH PRESSURE TRIAXIAL CELL WITH INTERNAL LOAD CELL

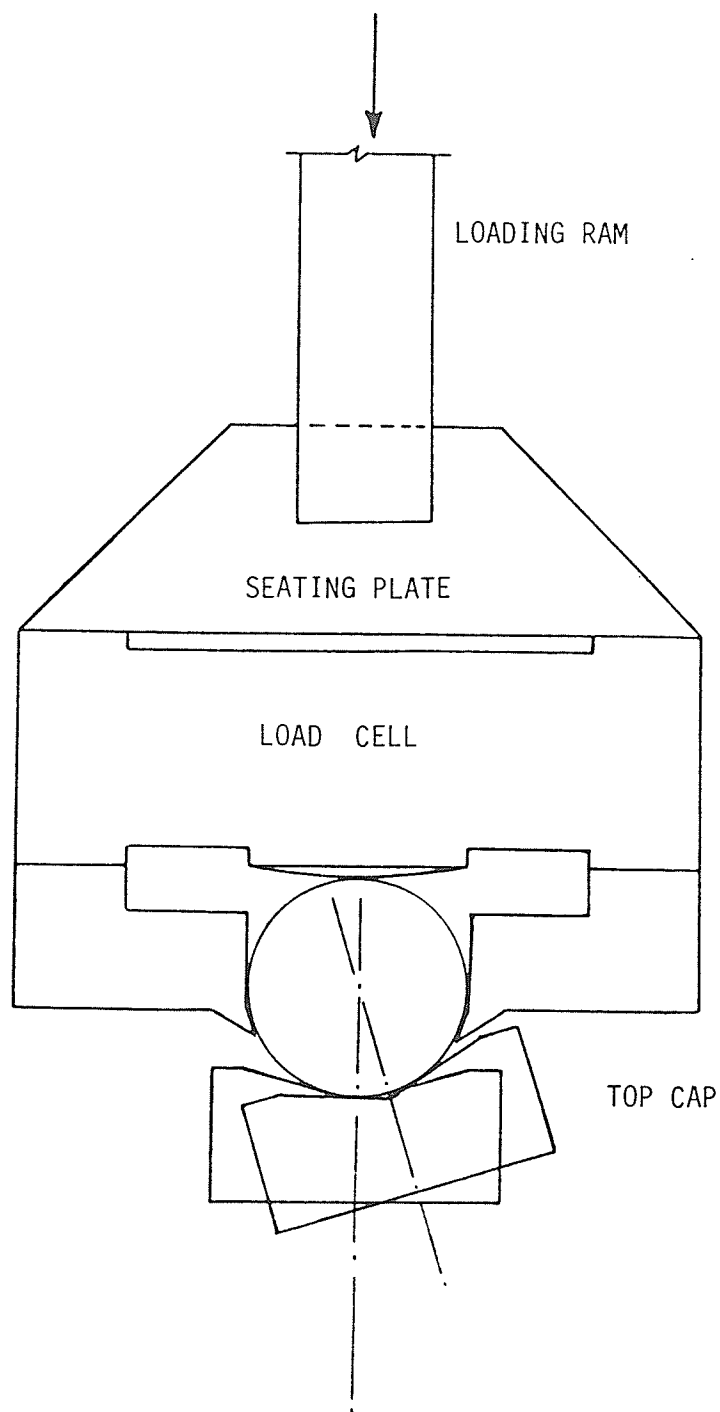


FIGURE 3.2 - LOAD CELL AND BALL BEARING ASSEMBLY

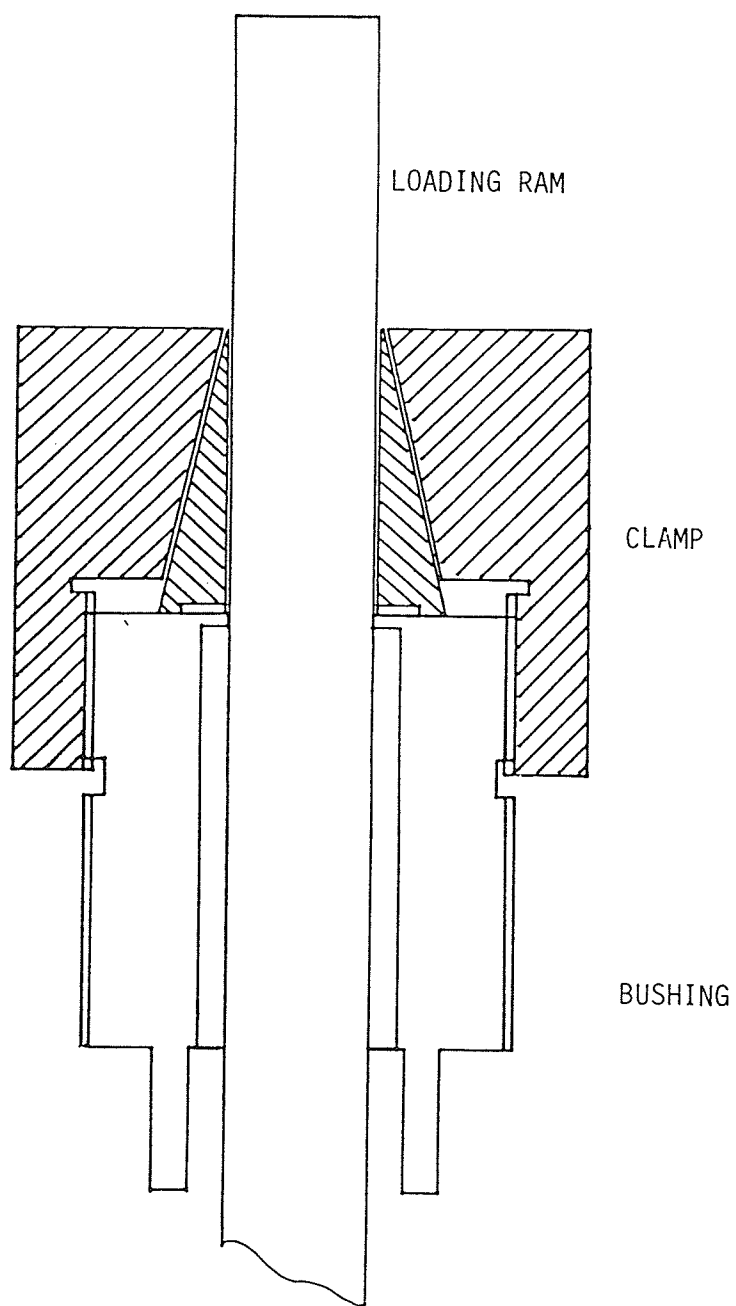


FIGURE 3.3 - LOADING RAM CLAMPING ARRANGEMENT

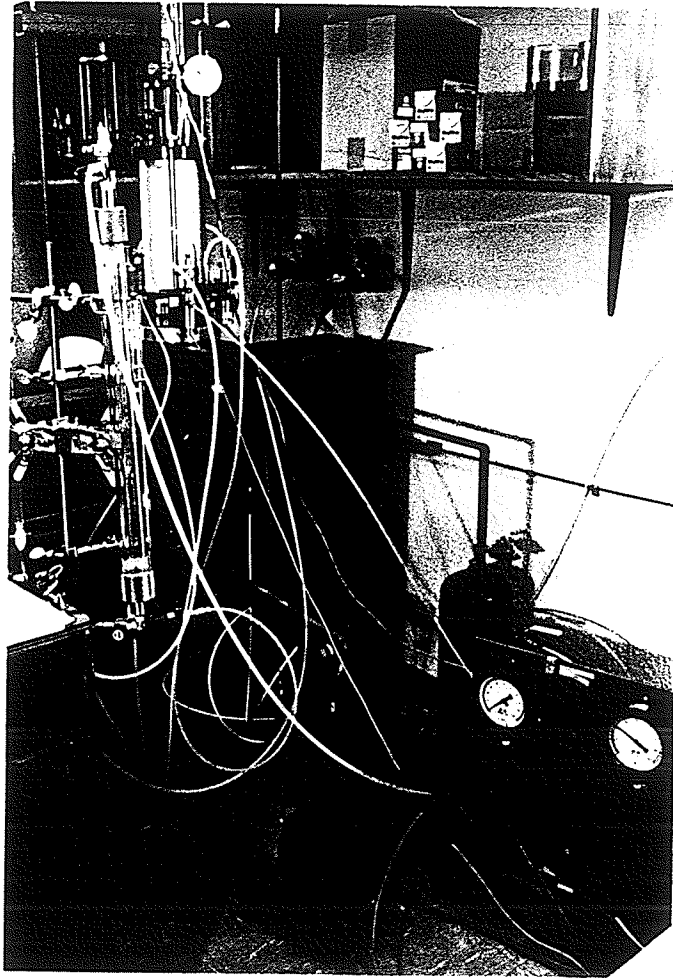


FIGURE 3.4 - TEST SET-UP USING BRAINARD KILMAN (B-K) TRIAXIAL CELL

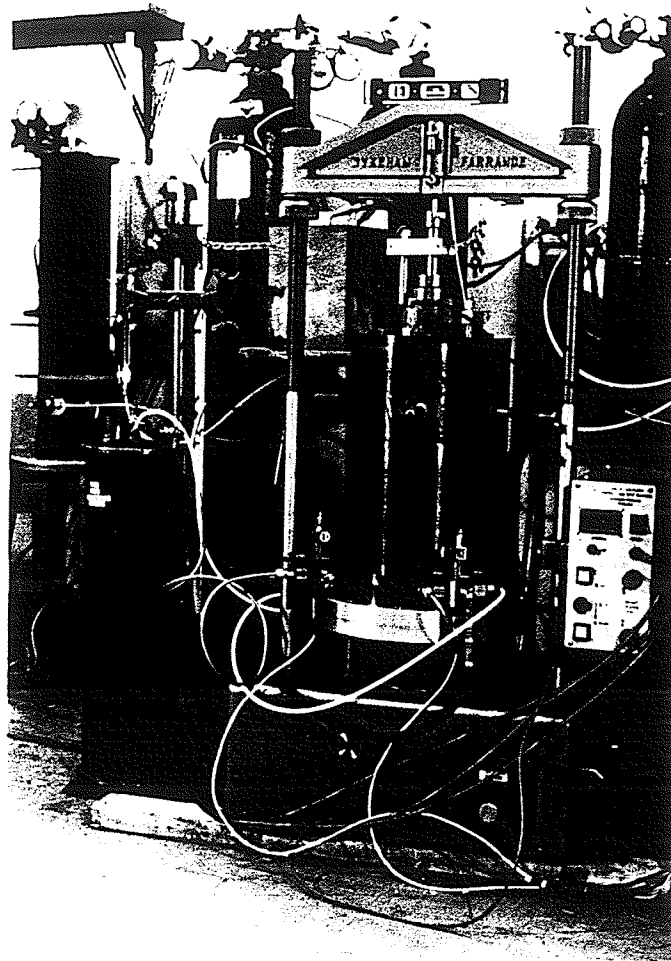


FIGURE 3.5 - SPECIALLY DESIGNED HIGH PRESSURE TRIAXIAL CELL

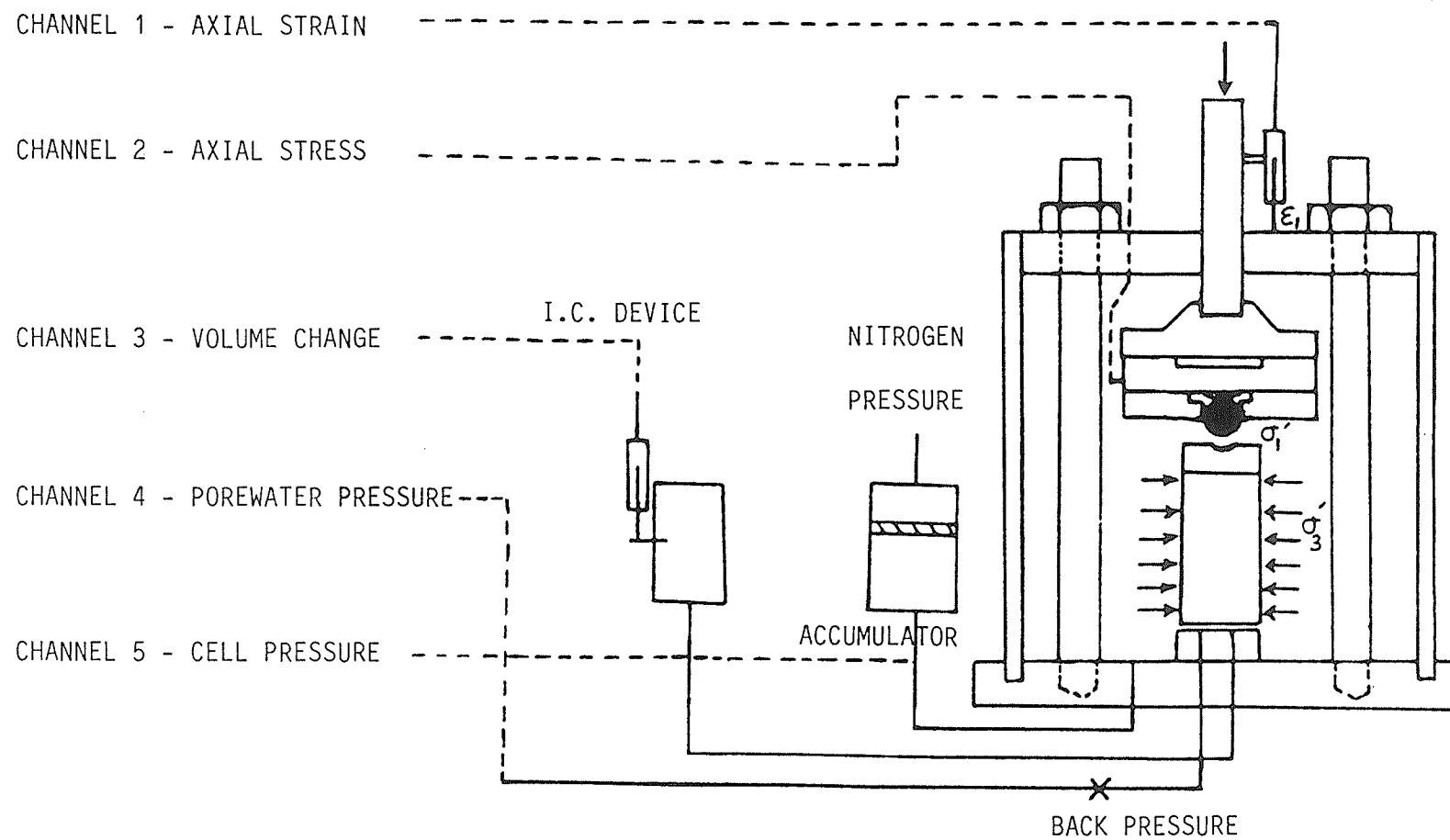


FIGURE 3.6 - LAYOUT OF THE COMPUTER-AIDED TRIAXIAL SYSTEM

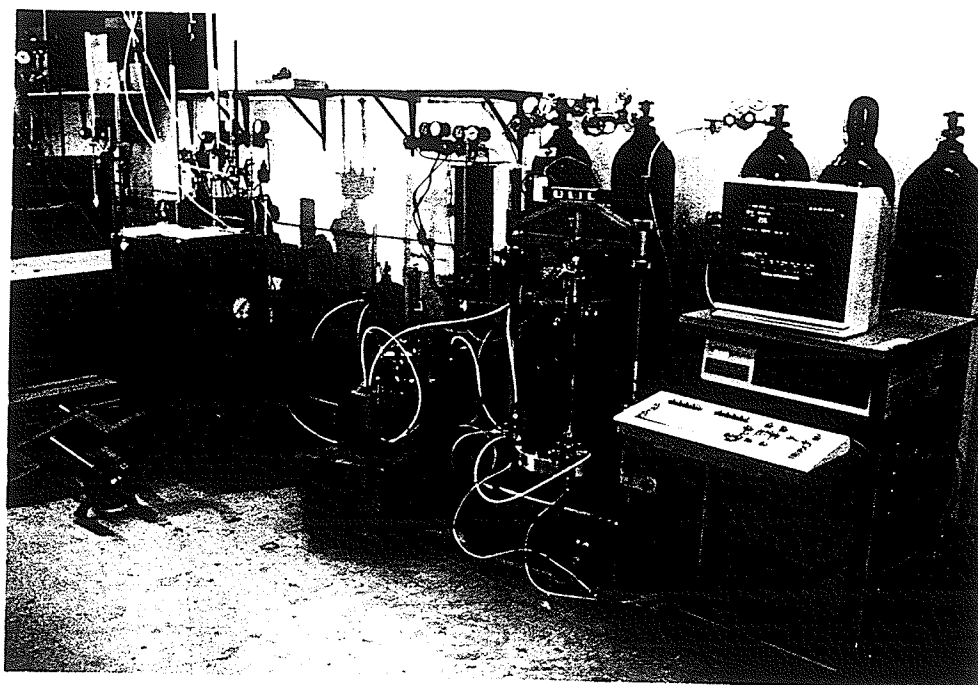


FIGURE 3.7 - COMPUTER-AIDED HIGH PRESSURE TRIAXIAL SYSTEM

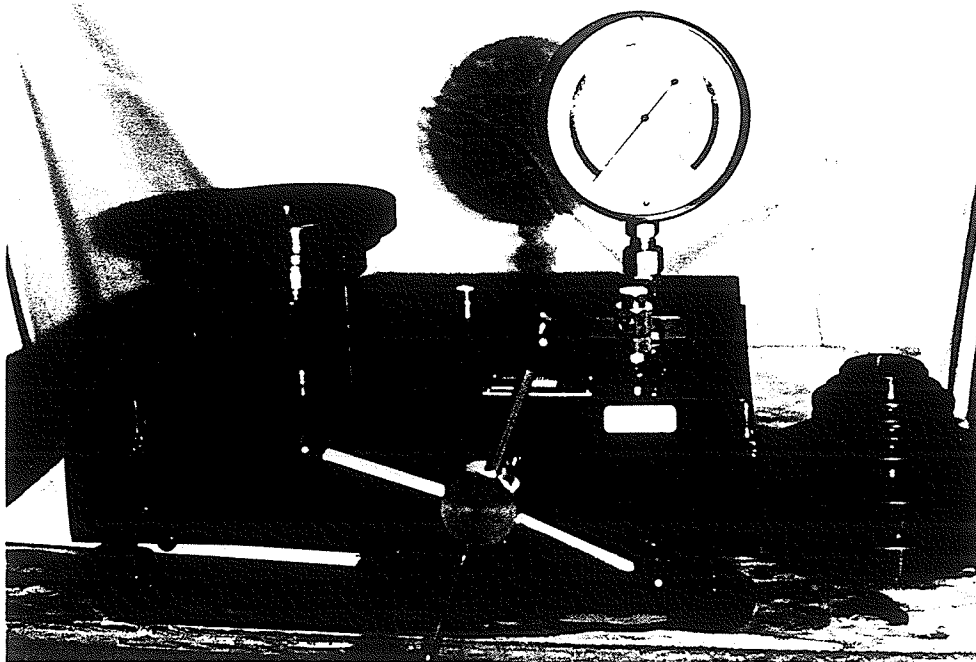


FIGURE 3.8 - CALIBRATION DEVICE FOR PRESSURE TRANSDUCERS, COMMERCIALY
KNOWN AS ASHCROFT PORTABLE DEAD WEIGHT TESTER

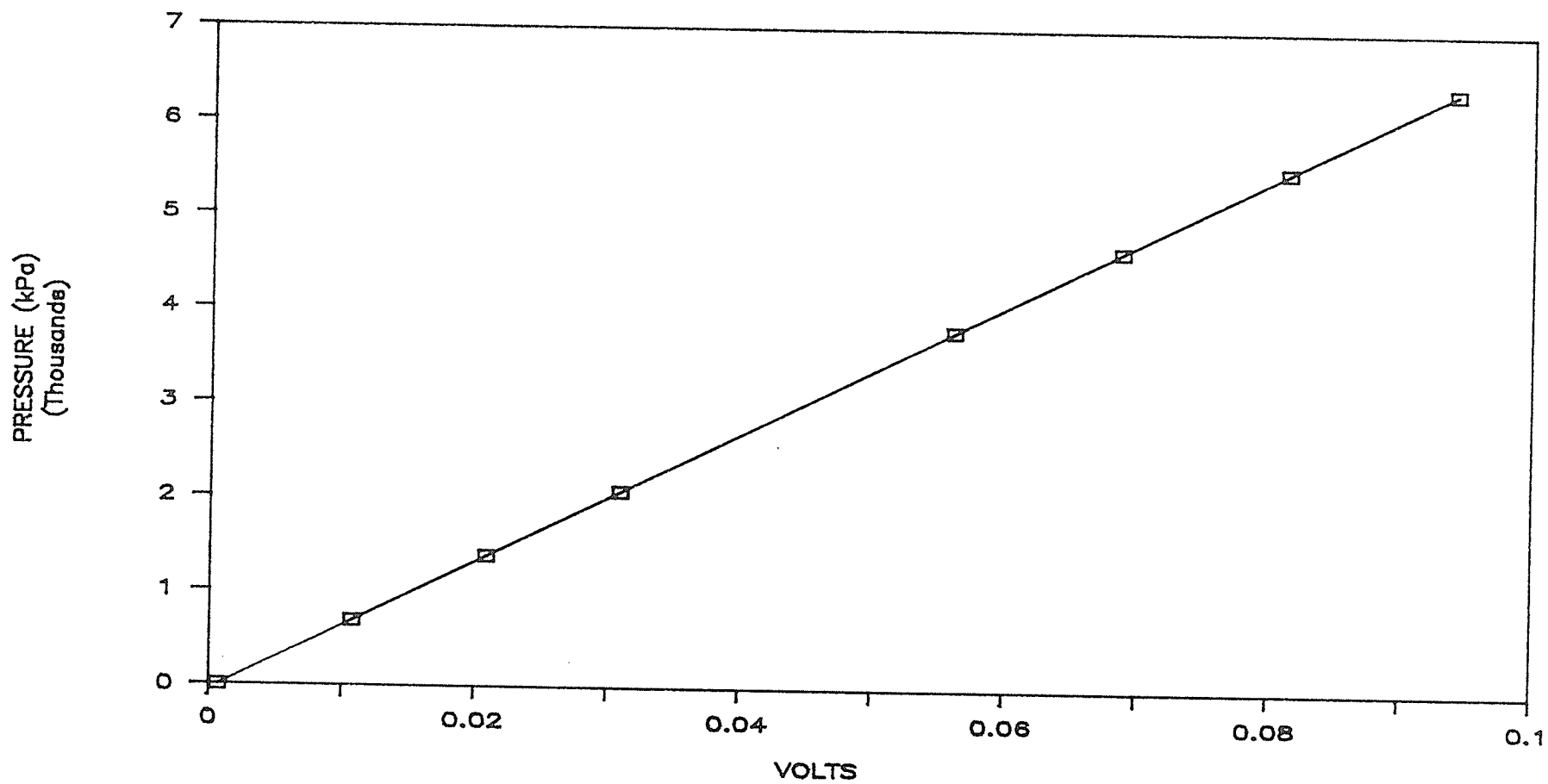


FIGURE 3.9 - AN EXAMPLE CALIBRATION CURVE FOR A PRESSURE TRANSDUCER

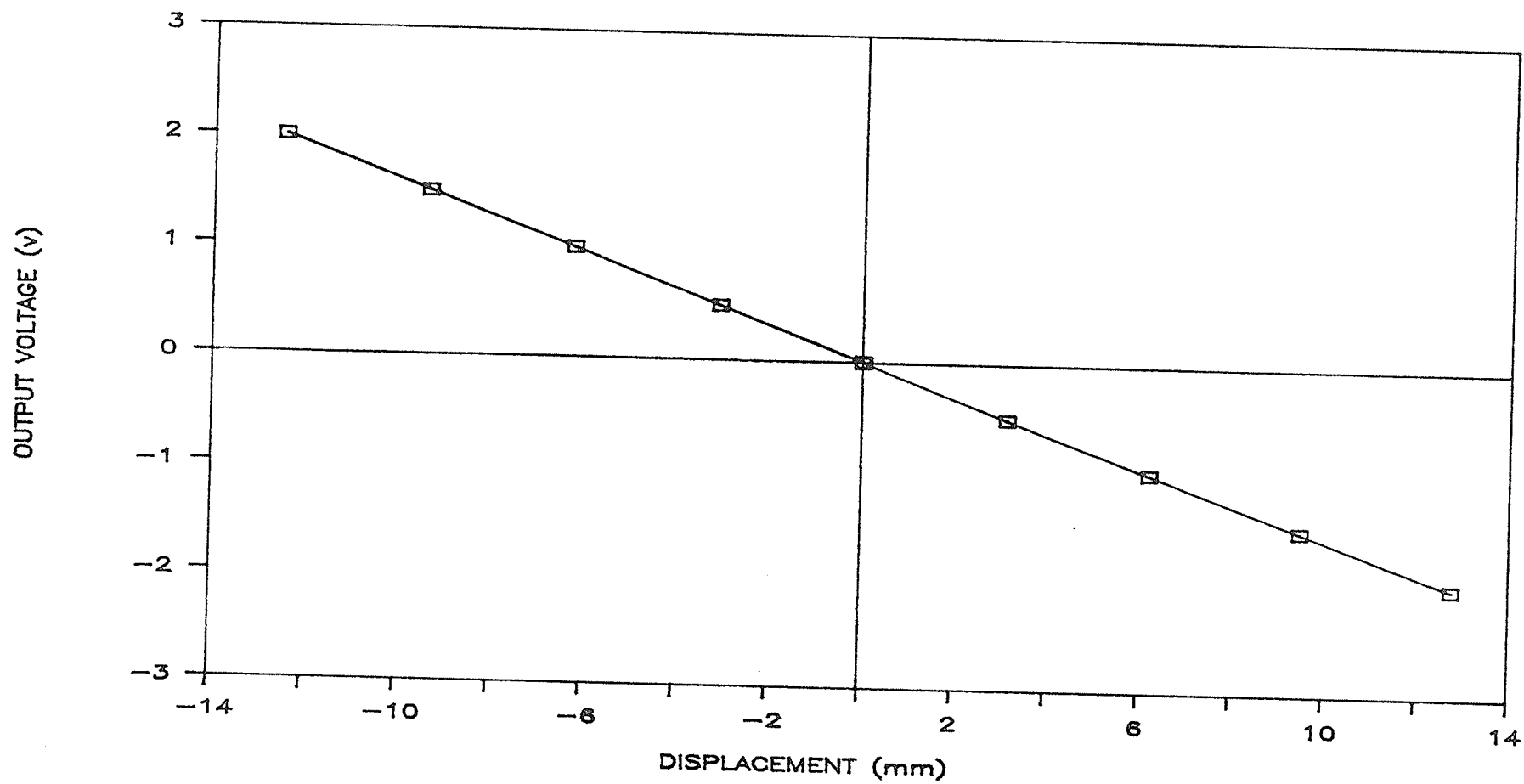


FIGURE 3.10- AN EXAMPLE CALIBRATION CURVE FOR A LINEAR VARIATION DIFFERENTIAL TRANSDUCER (LVDT)

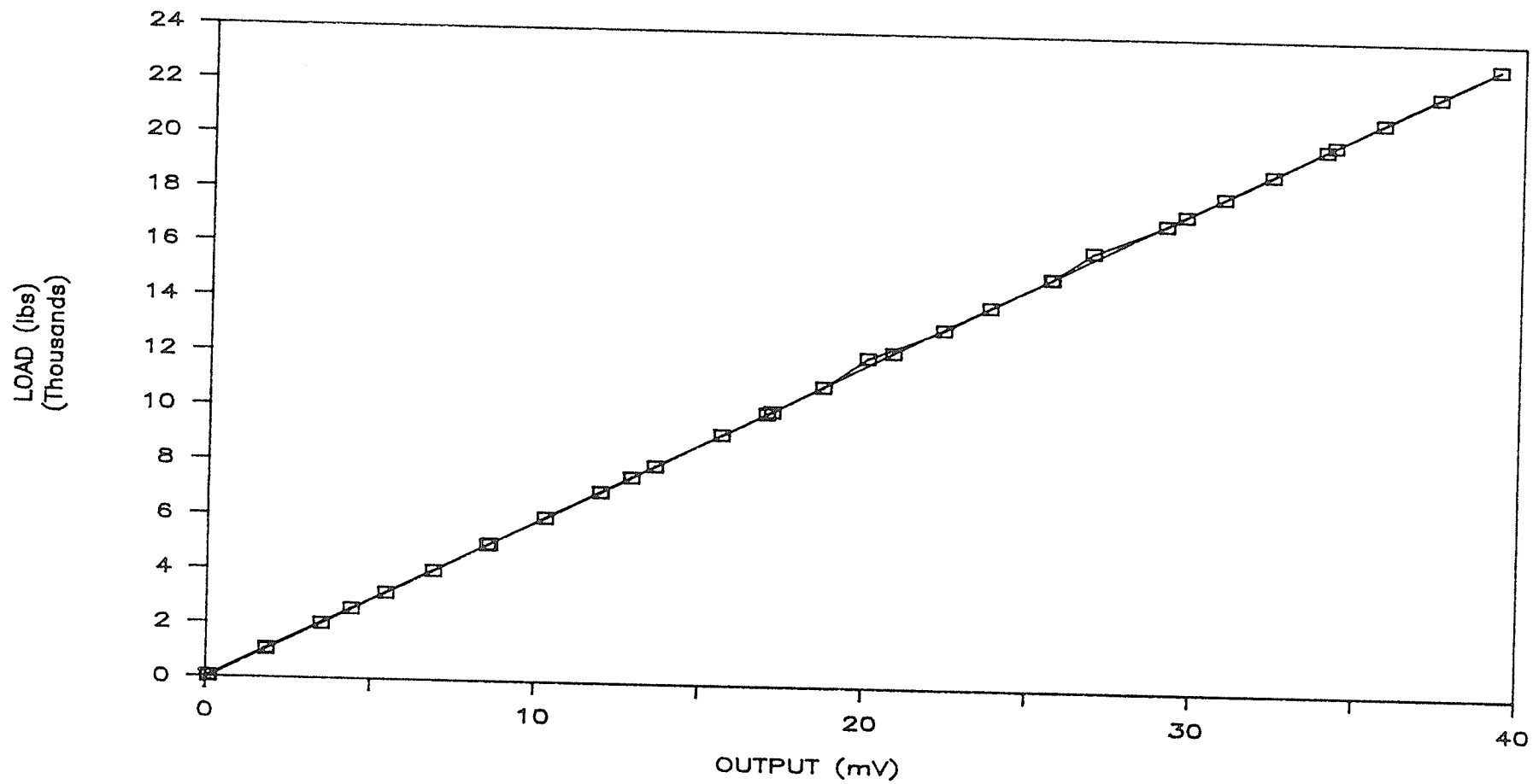


FIGURE 3.11- AN EXAMPLE CALIBRATION CURVE FOR THE HIGH CAPACITY INTERFACE LOAD CELL UNDER 10 MPa CONFINING PRESSURE DURING LOADING AND UNLOADING

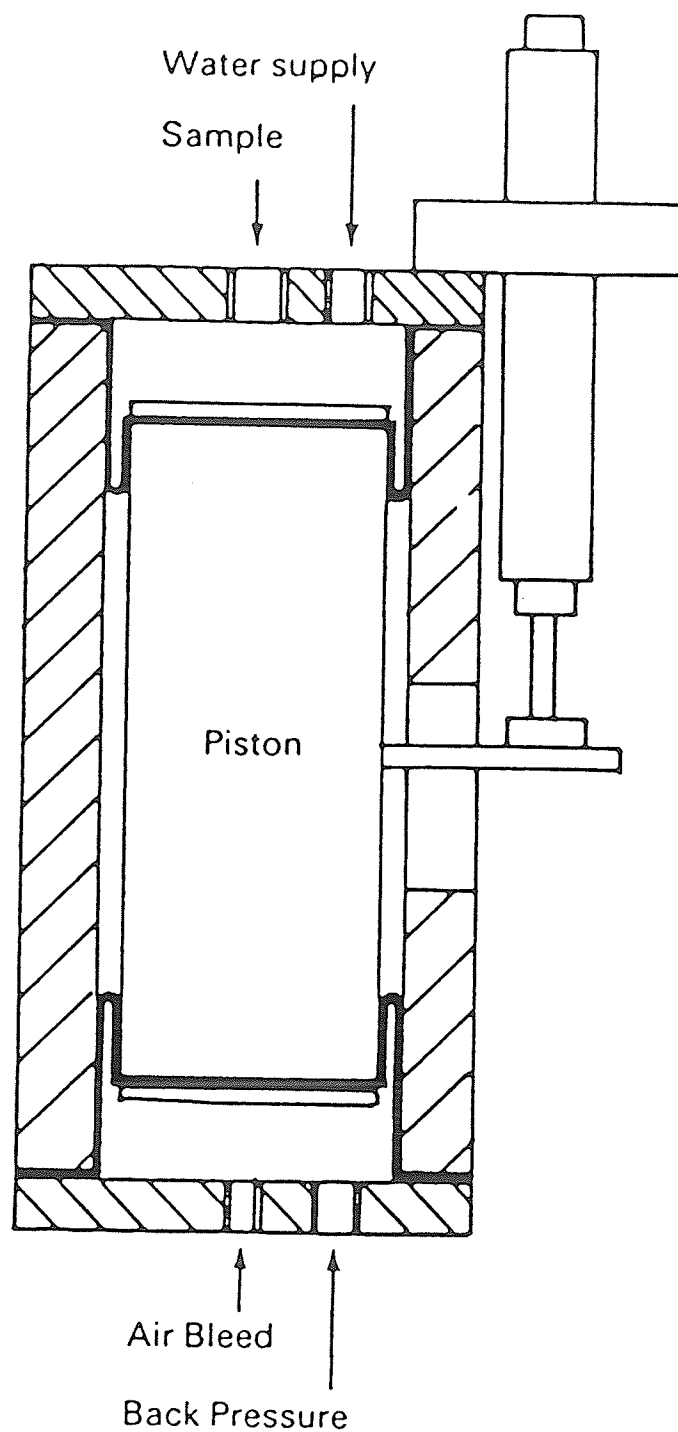


FIGURE 3.12 - IMPERIAL COLLEGE VOLUME CHANGE TRANSDUCER FROM SHAPE INSTRUMENTS

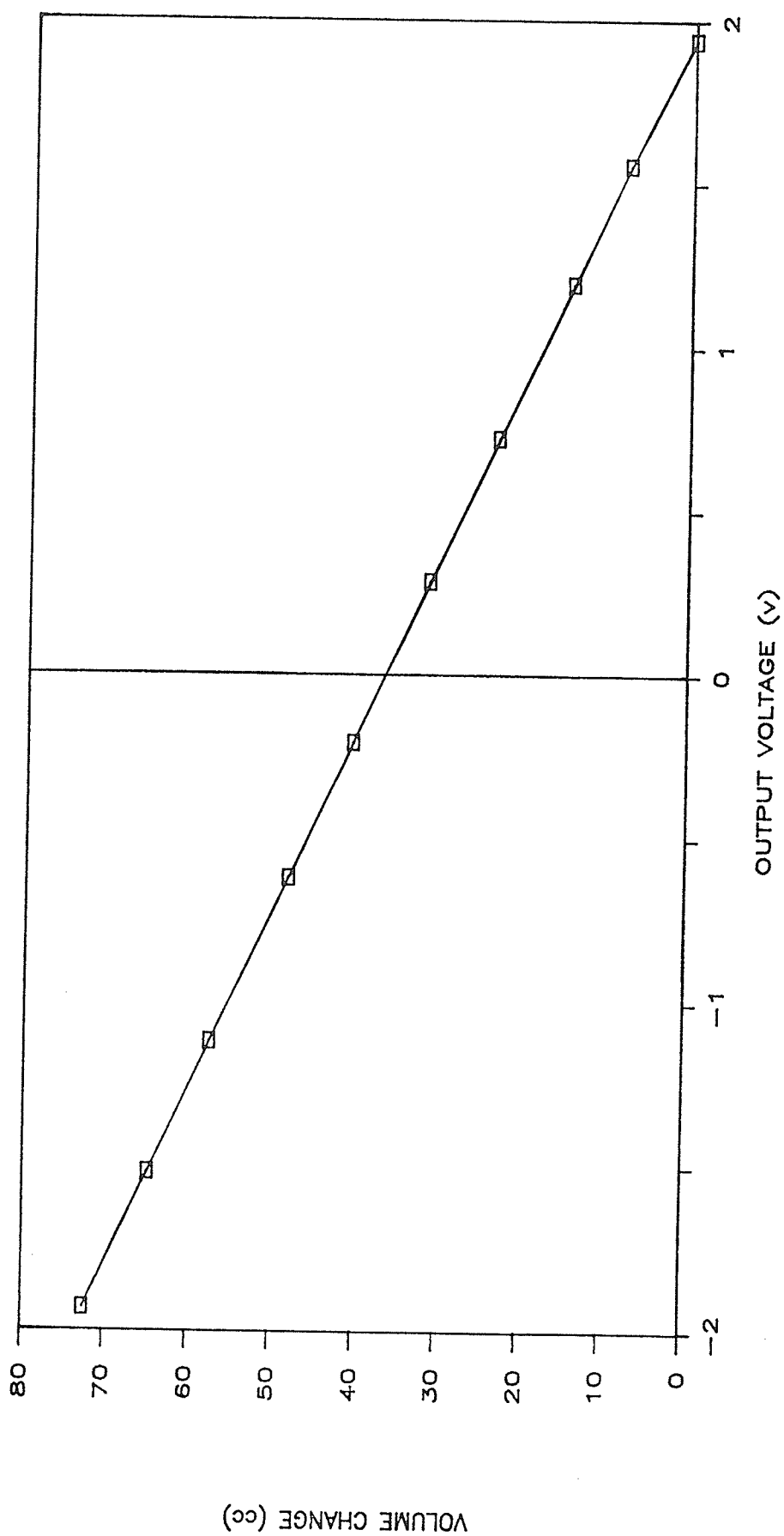


FIGURE 3.13 - AN EXAMPLE CALIBRATION CURVE FOR THE VOLUME CHANGE DEVICE UNDER 1 MPa PRESSURE

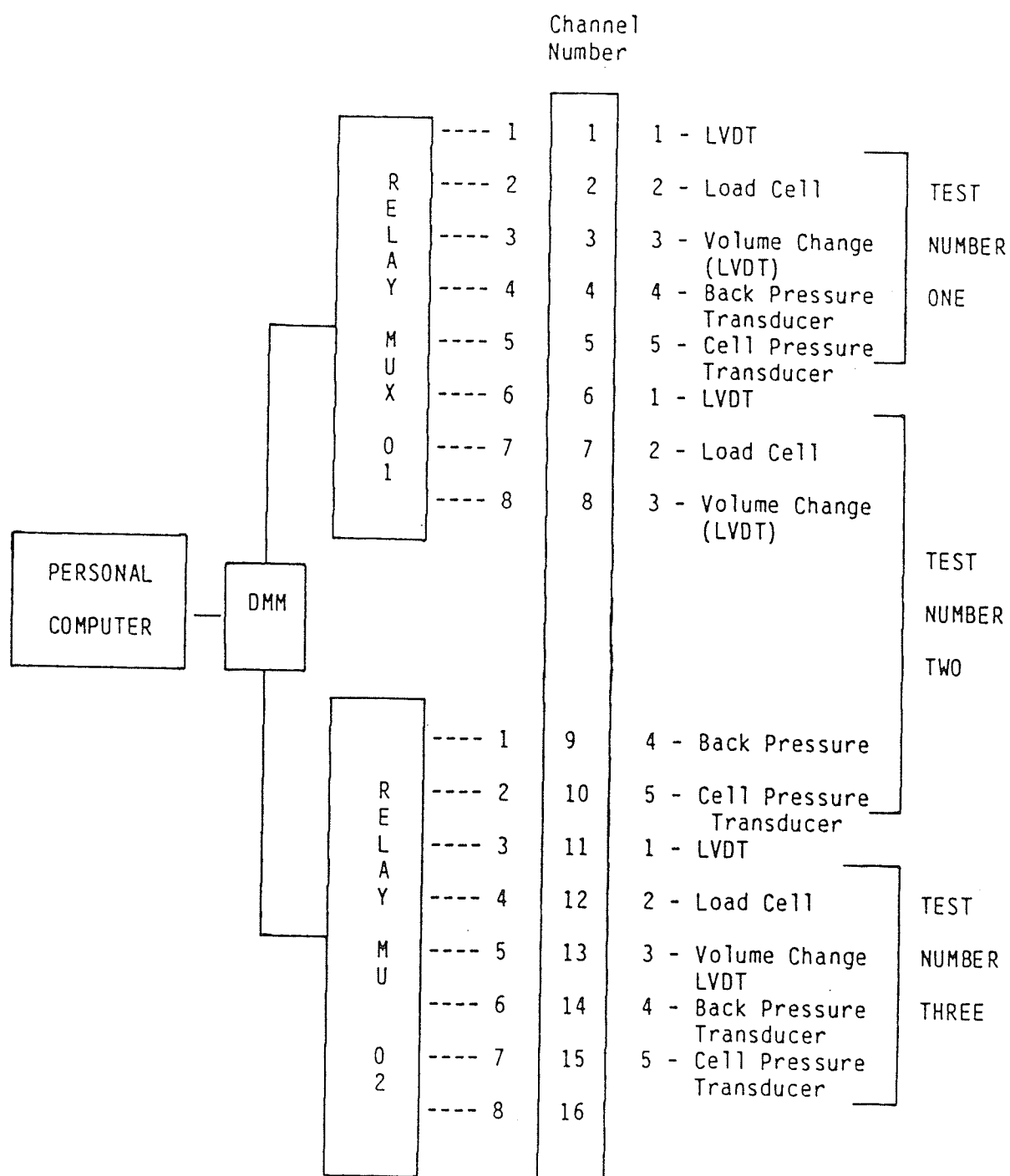


FIGURE 3.14 - INTERFACING DEVICES IN DATA ACQUISITION SYSTEM (DAS)

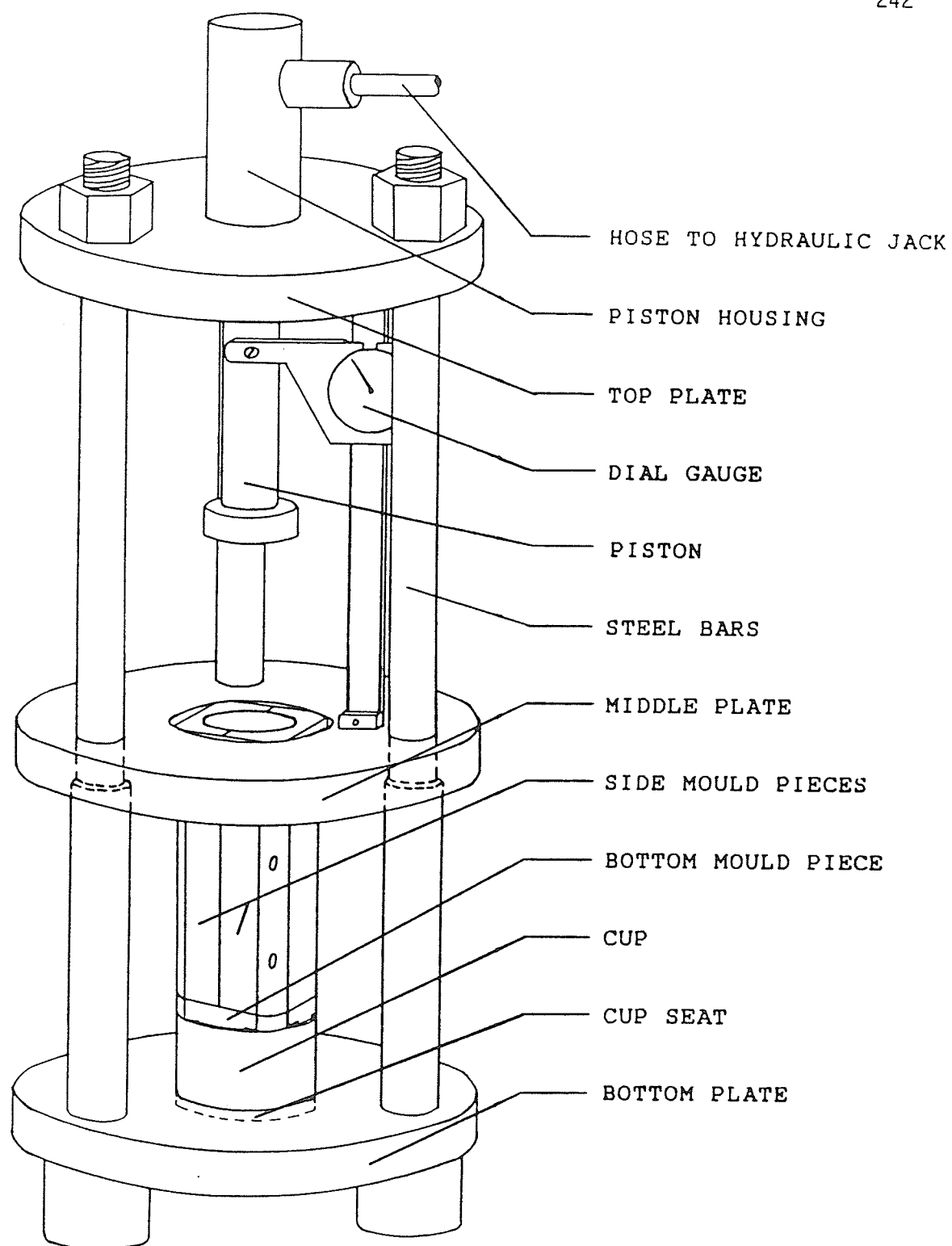


FIGURE 4.1 - NEW COMPACTION FRAME (AFTER YARACHEWSKI 1988)

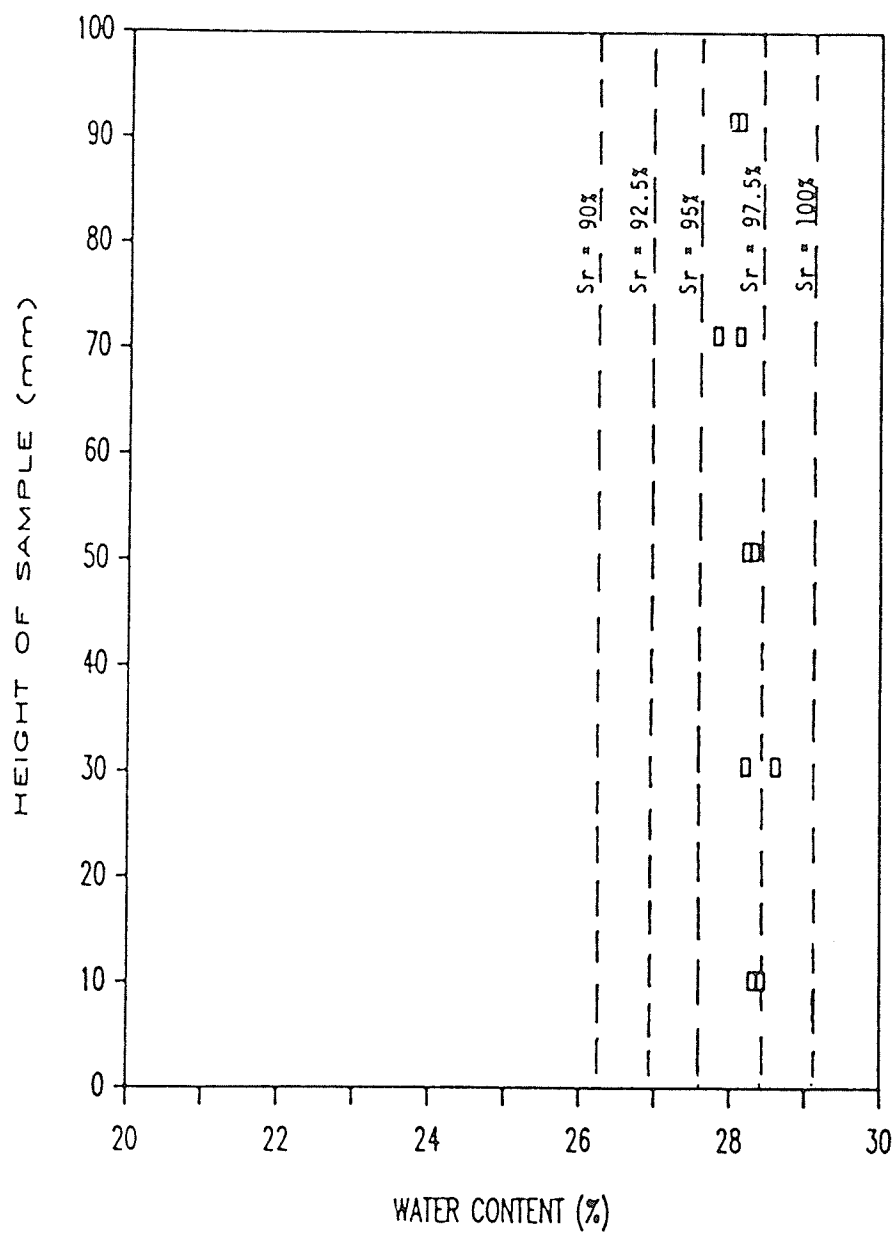


FIGURE 4.2 - VARIATION OF WATER CONTENT ALONG THE HEIGHT OF A SAMPLE

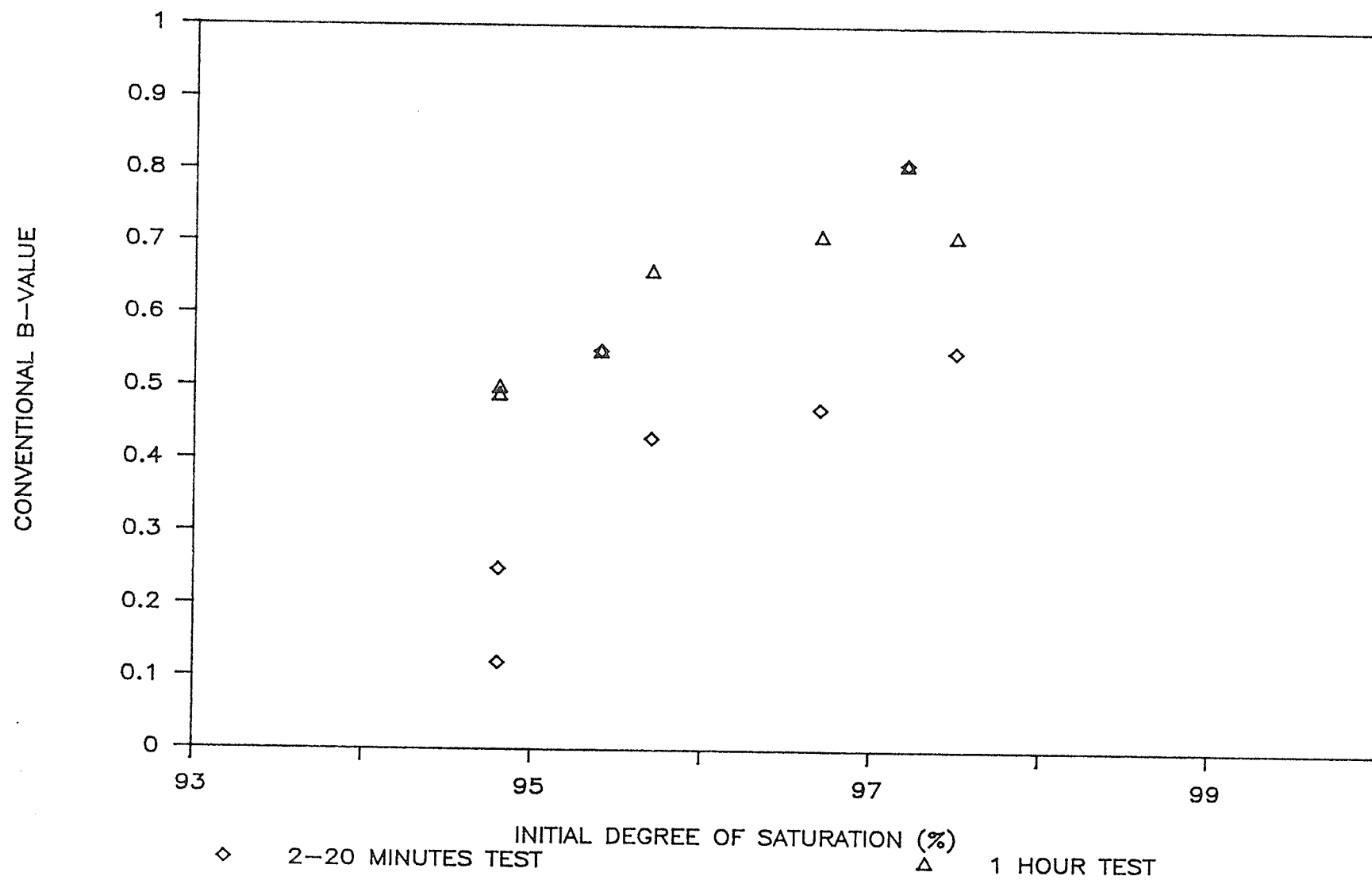


FIGURE 4.3 - VARIATION OF ATTAINED B-VALUE WITH INITIAL DEGREE OF SATURATION (%)

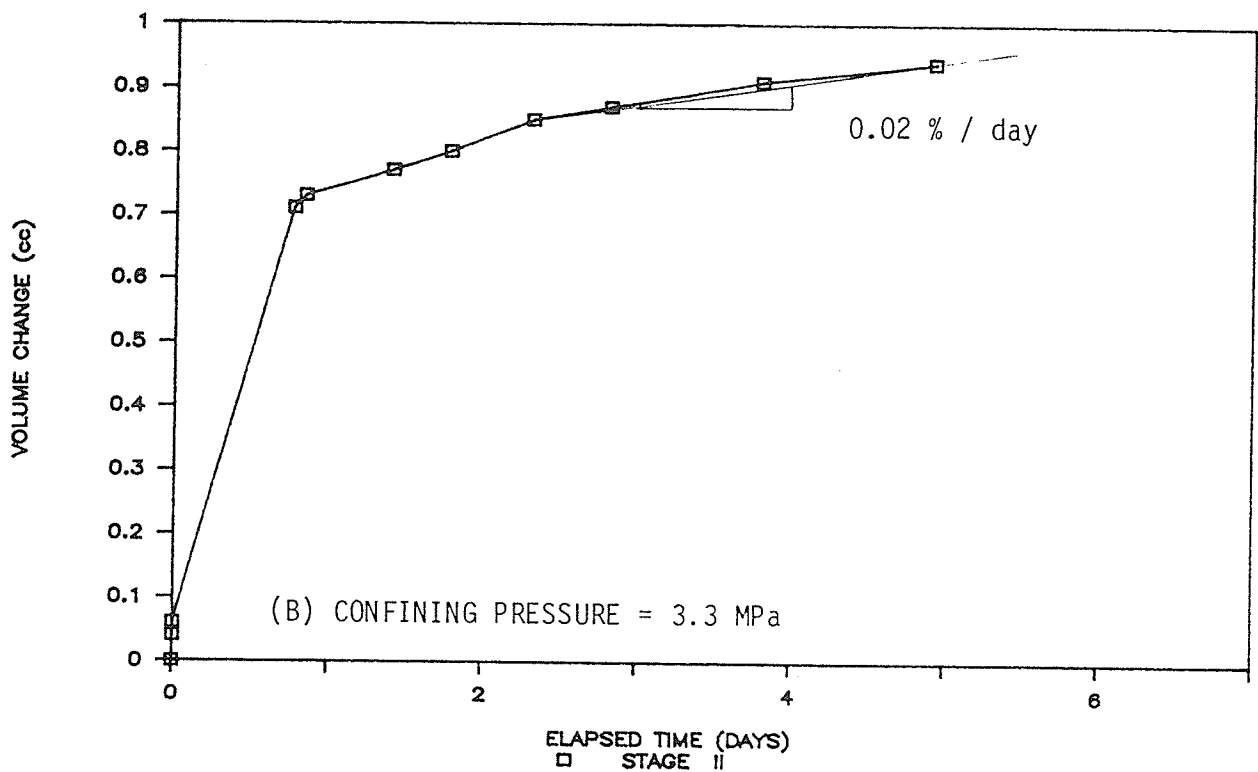
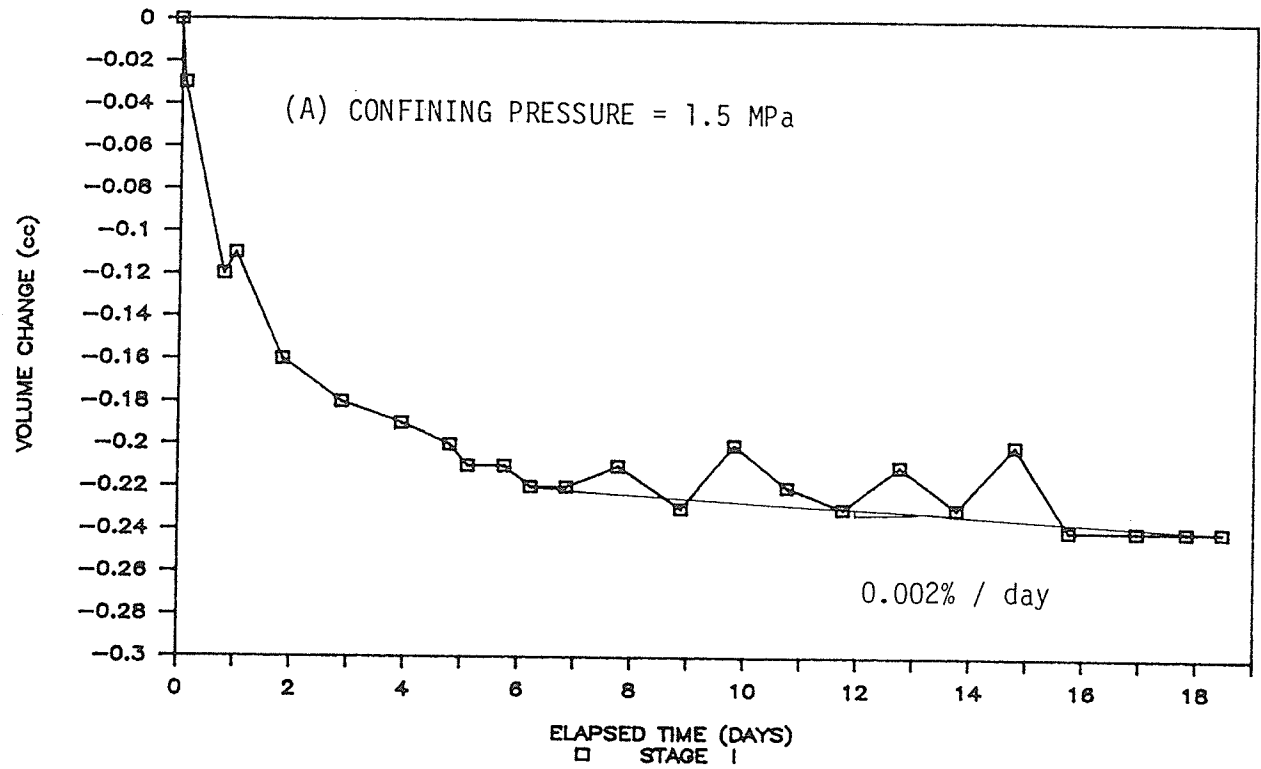


FIGURE 4.4 - SYSTEM VOLUME CHANGE VERSUS TIME - LEAKAGE TEST USING TWO 0.36 mm THICK MEMBRANES

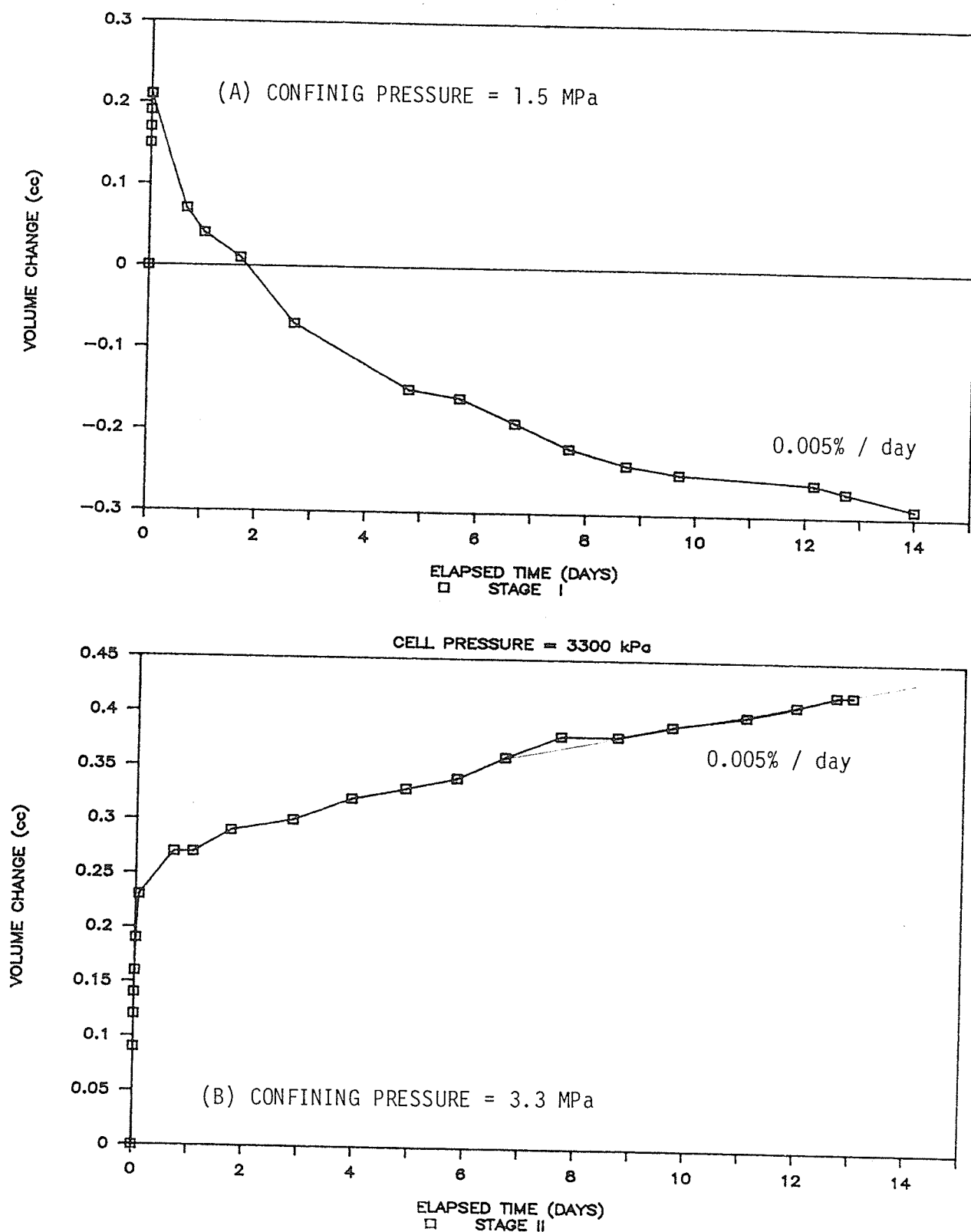


FIGURE 4.5 - SYSTEM VOLUME CHANGE VERSUS TIME - LEAKAGE TEST USING TWO 0.64 mm THICK MEMBRANES

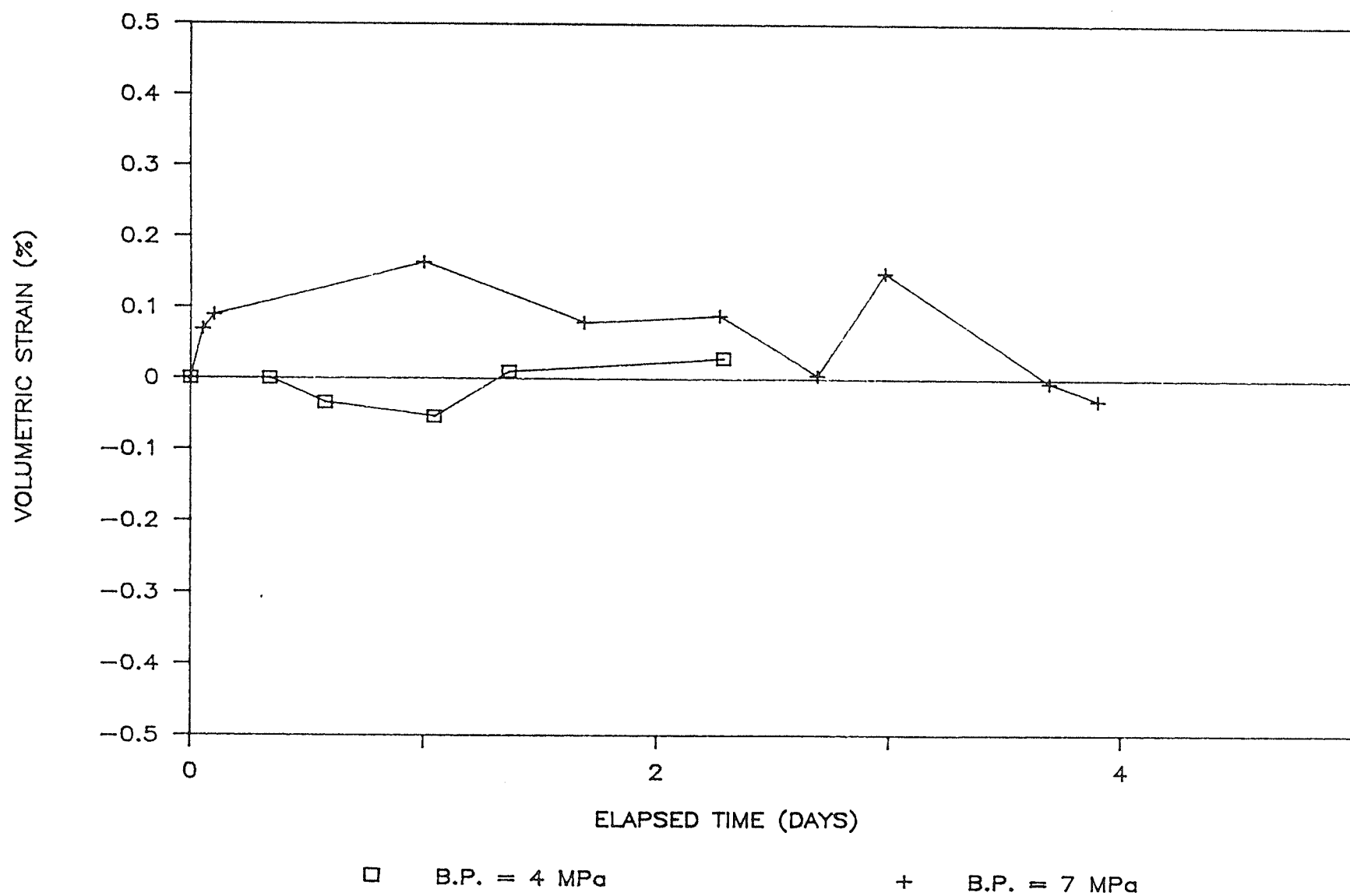


FIGURE 4.6 - SYSTEM VOLUME CHANGE VERSUS ELAPSED TIME - LEAKAGE TEST USING 1.27 mm THICK MEMBRANES

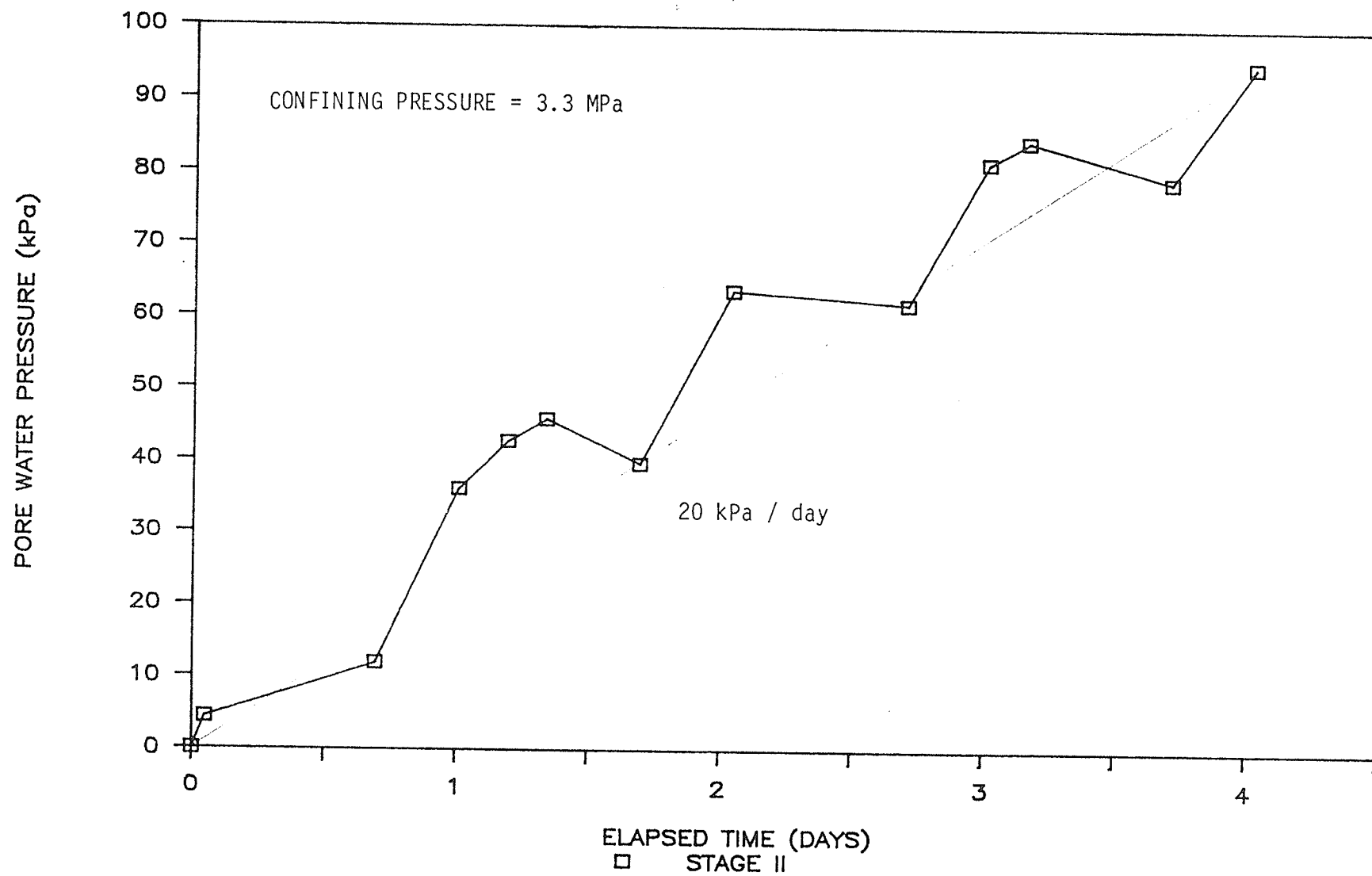


FIGURE 4.7 -POREWATER PRESSURE BUILD-UP VERSUS TIME - LEAKAGE TEST USING TWO 0.64 mm THICK MEMBRANES

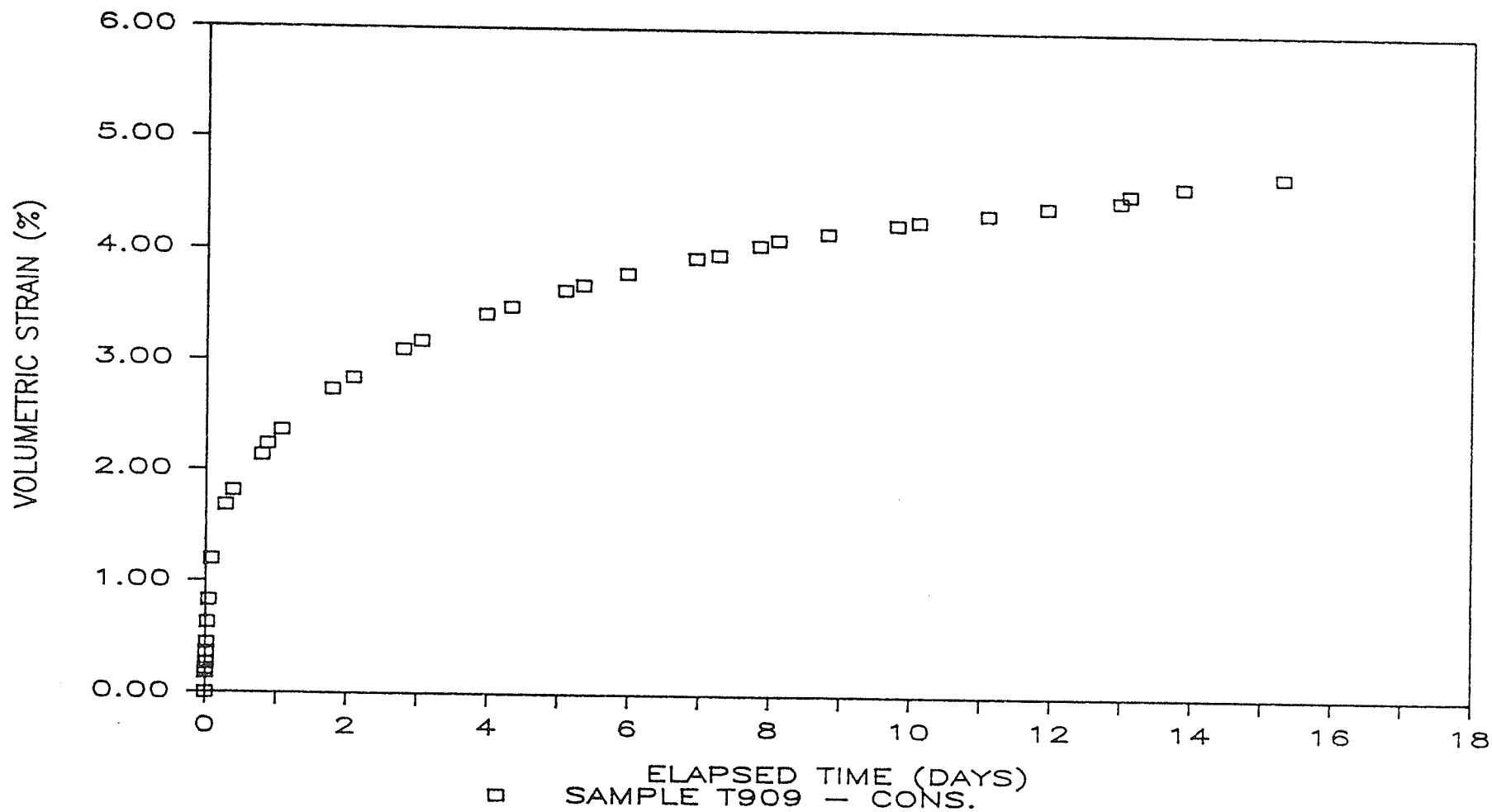


FIGURE 5.1 - VOLUMETRIC STRAIN VERSUS CONSOLIDATION DURATION - T909

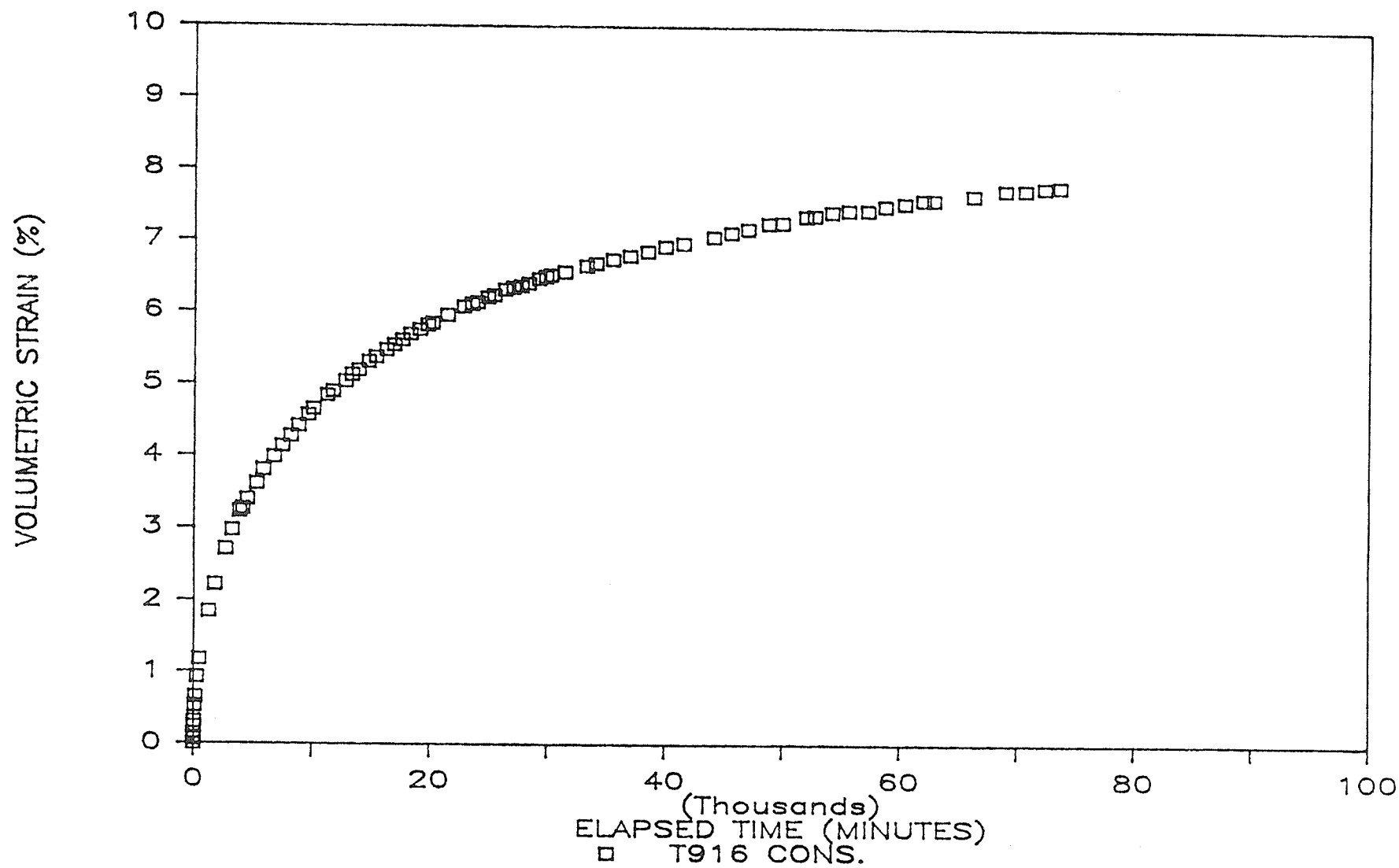


FIGURE 5.2 - VOLUMETRIC STRAIN VERSUS CONSOLIDATION DURATION - T916

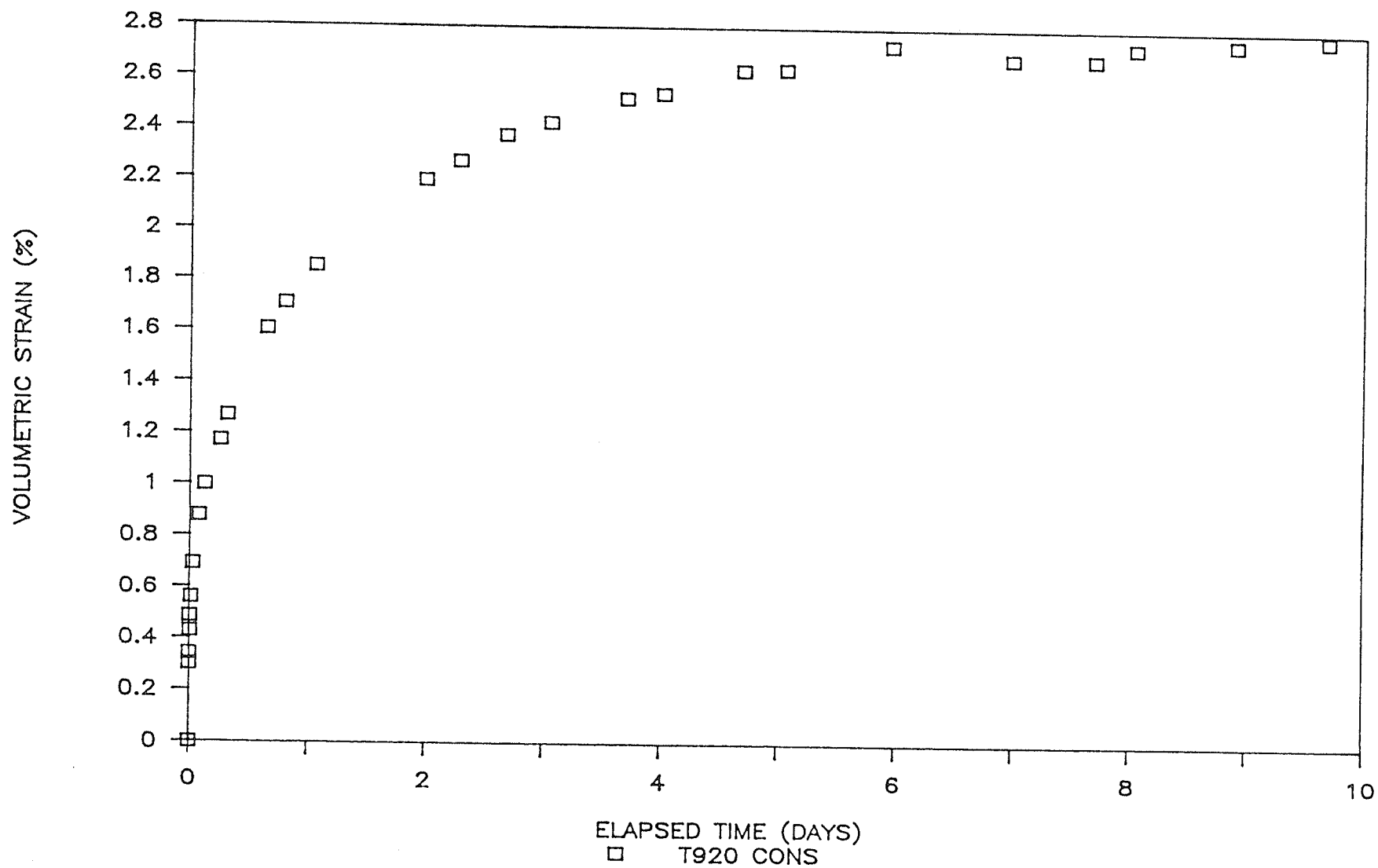


FIGURE 5.3 - VOLUMETRIC STRAIN VERSUS CONSOLIDATION DURATION - T920

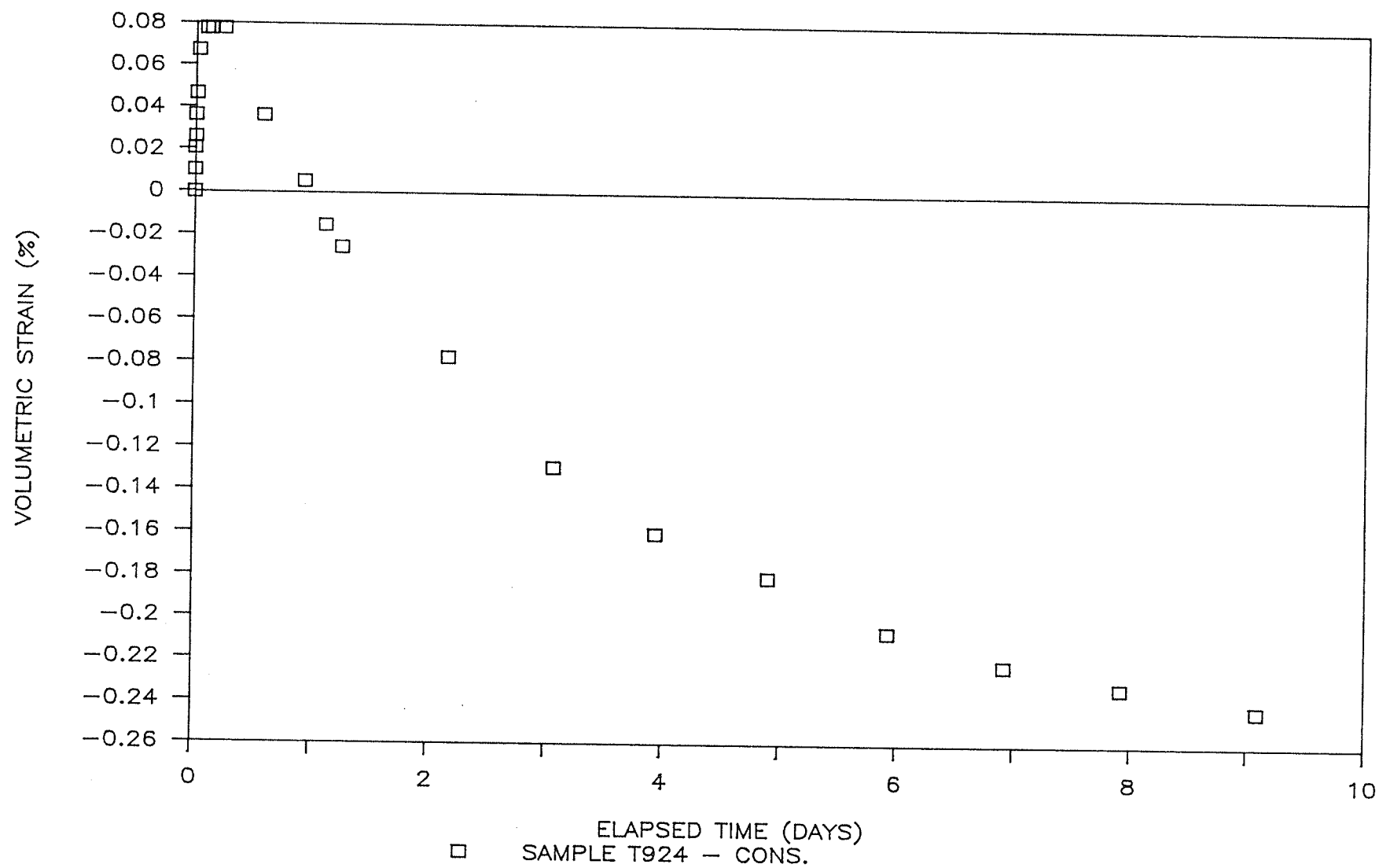


FIGURE 5.4 - VOLUMETRIC STRAIN VERSUS CONSOLIDATION DURATION - T924

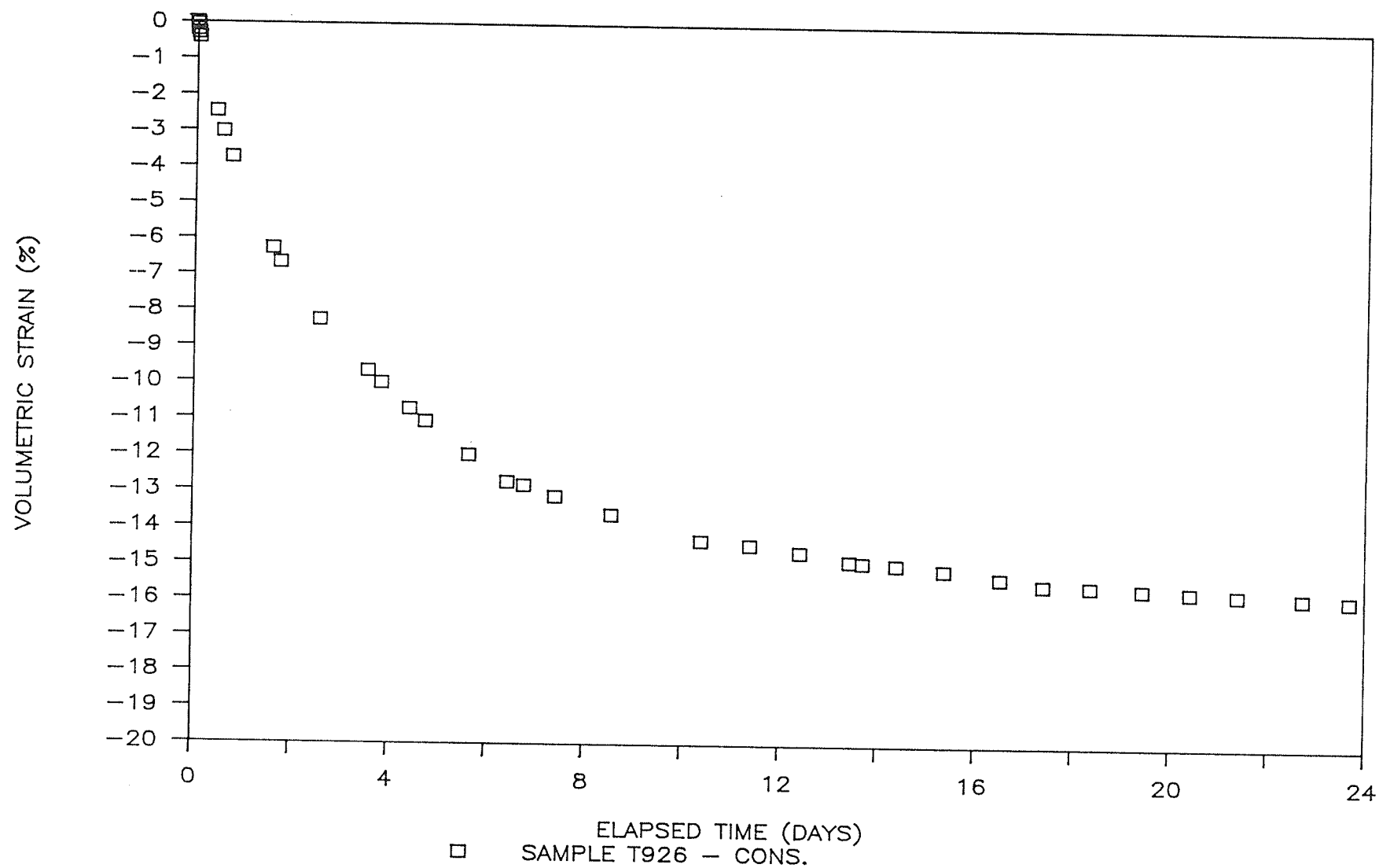


FIGURE 5.5 - VOLUMETRIC STRAIN VERSUS CONSOLIDATION DURATION - T926

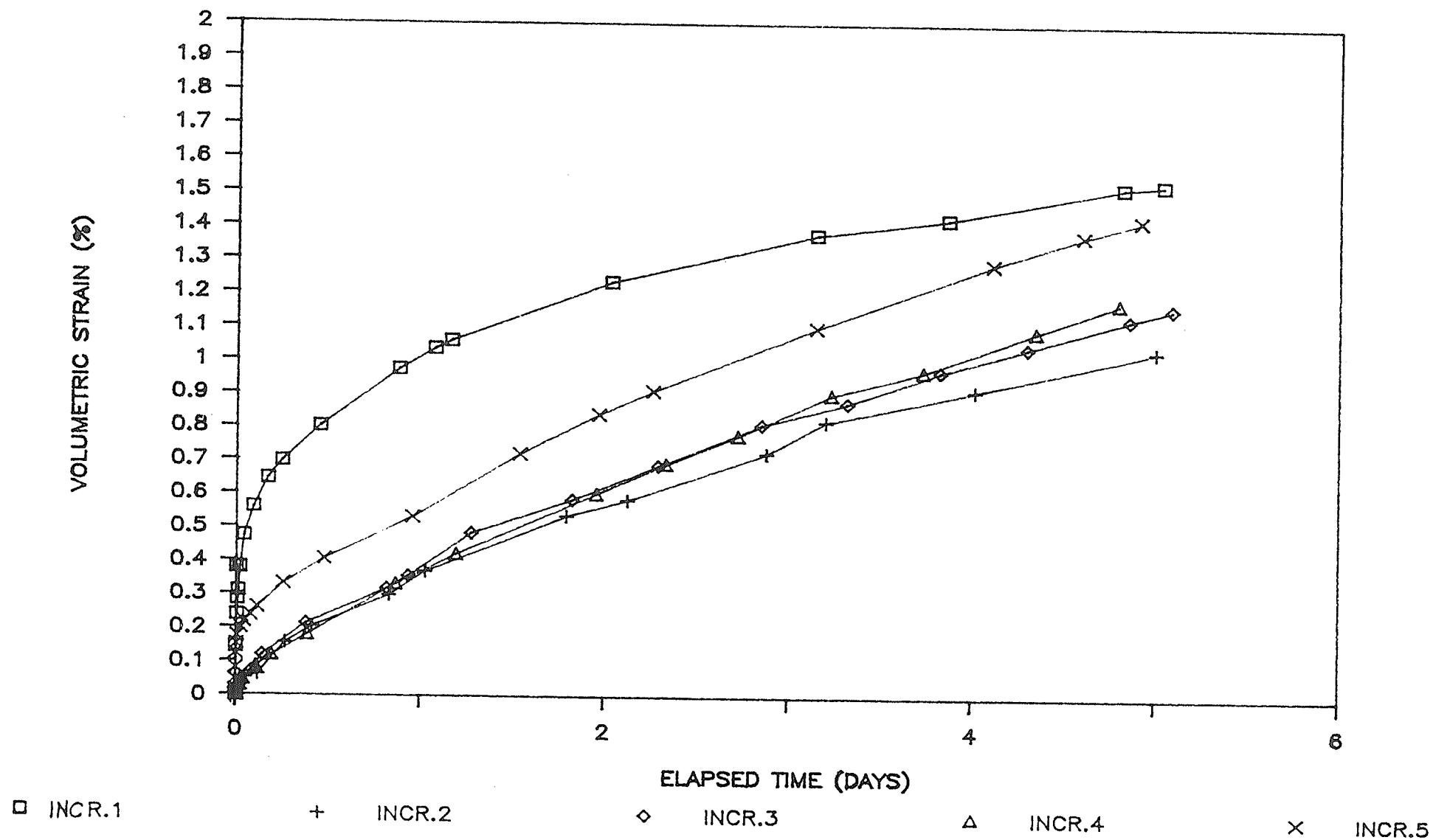


FIGURE 5.6 - VOLUMETRIC STRAIN VERSUS CONSOLIDATION DURATION - T915 (INCREMENTAL ISOTROPIC LOADING)

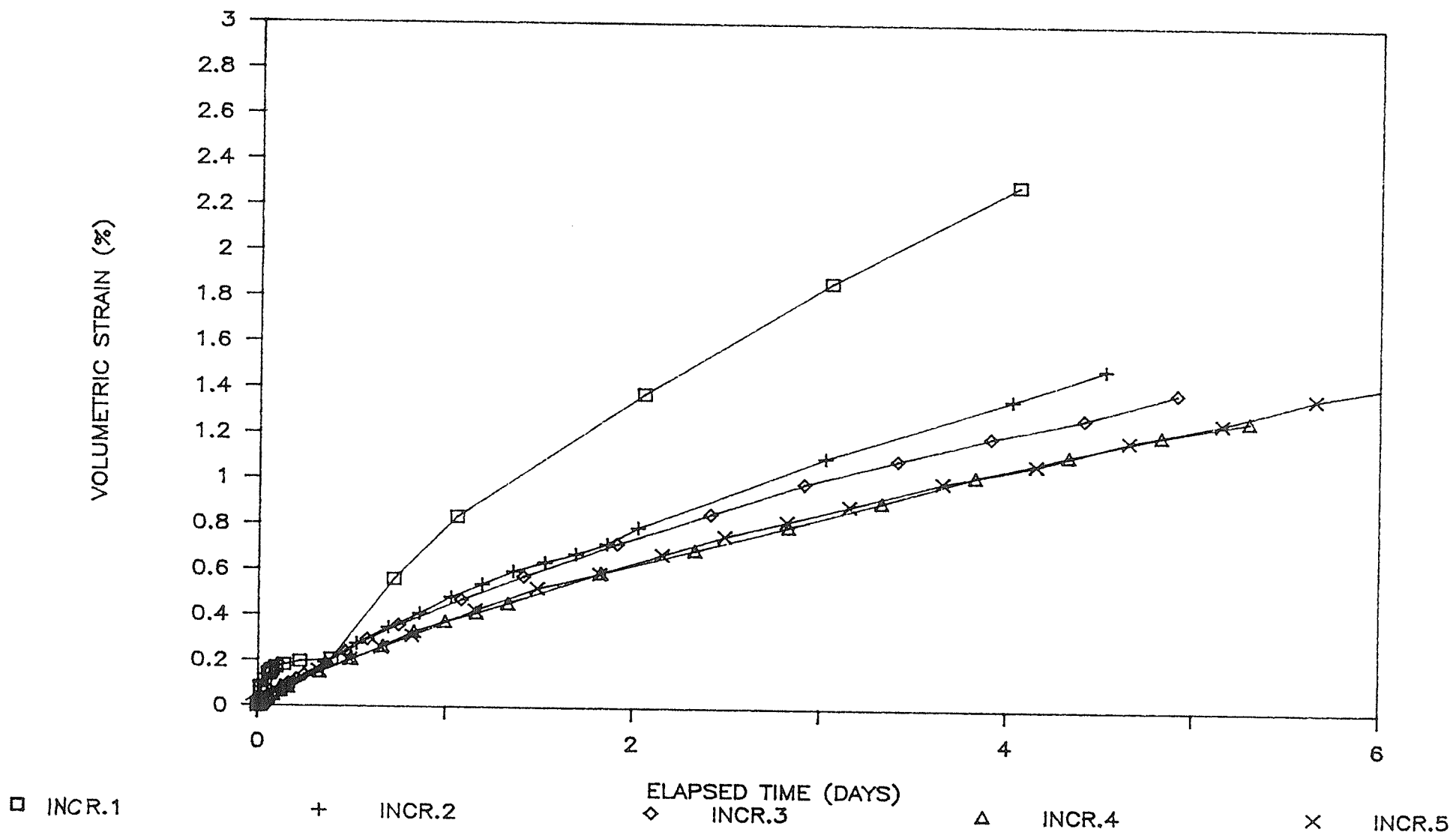


FIGURE 5.7 - VOLUMETRIC STRAIN VERSUS CONSOLIDATION DURATION - T925 (INCREMENTAL ISOTROPIC LOADING)

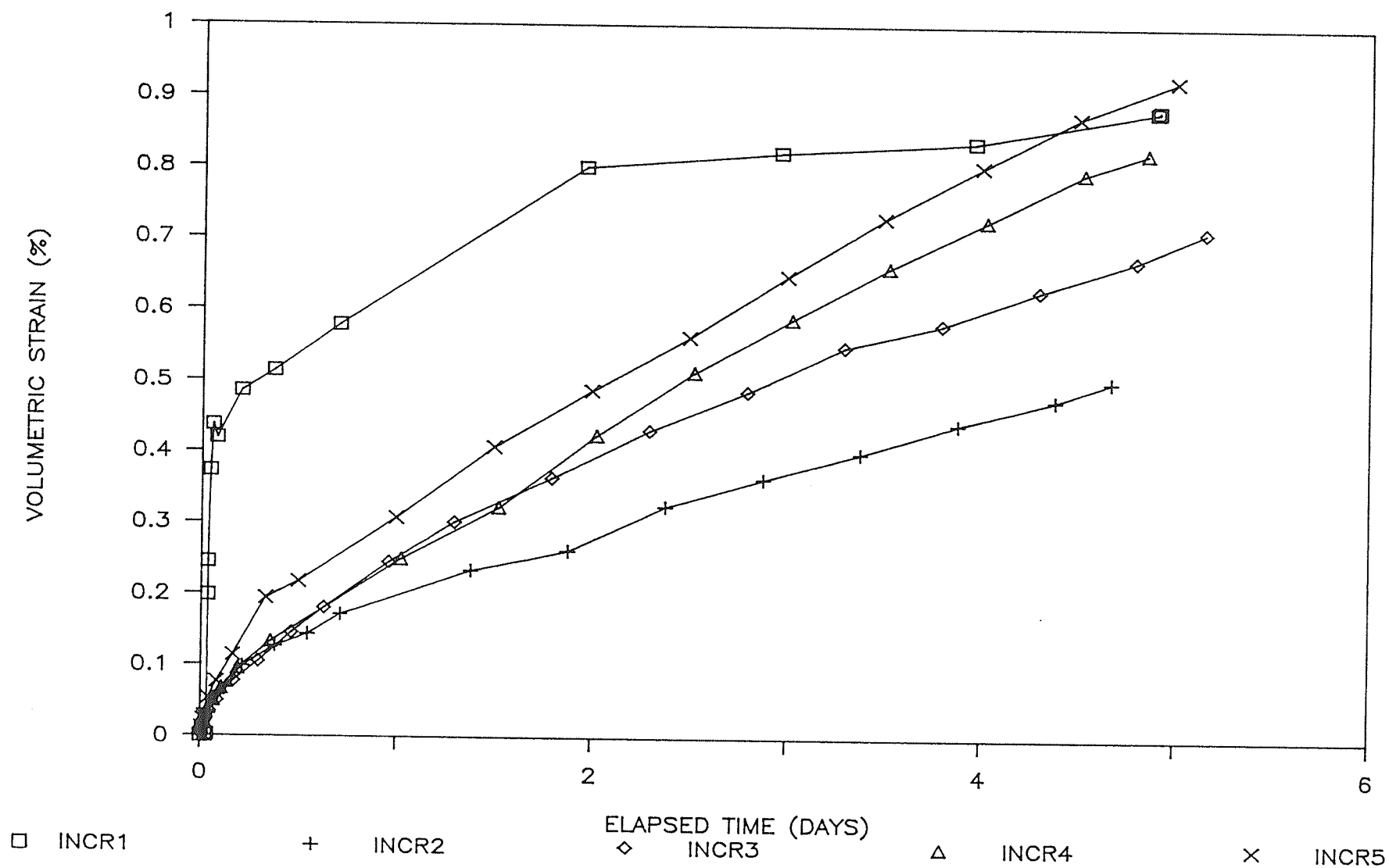


FIGURE 5.8 - VOLUMETRIC STRAIN VERSUS CONSOLIDATION DURATION - T930 (INCREMENTAL ISOTROPIC LOADING)

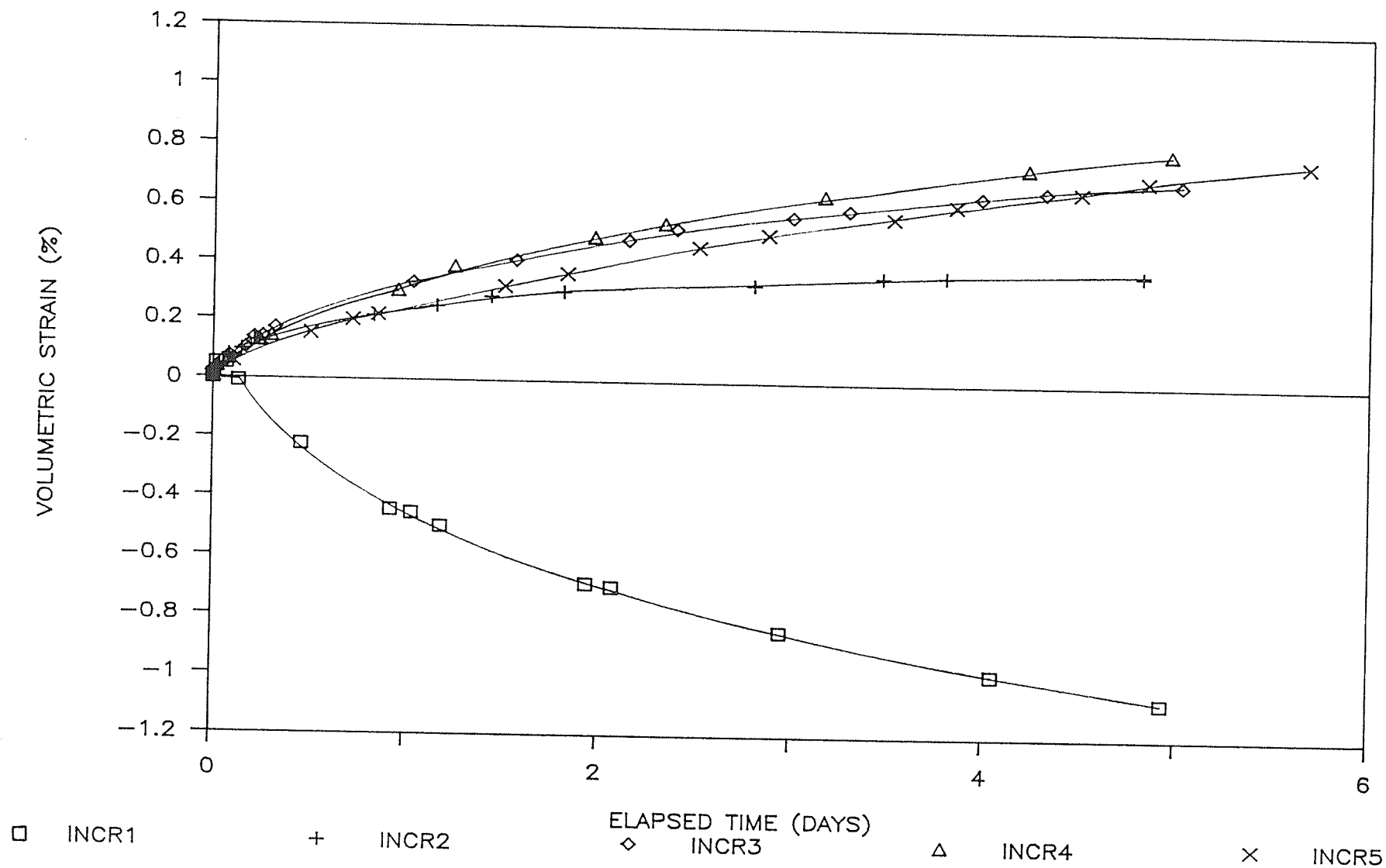


FIGURE 5.9 - VOLUMETRIC STRAIN VERSUS CONSOLIDATION DURATION - T929 (INCREMENTAL ISOTROPIC LOADING)

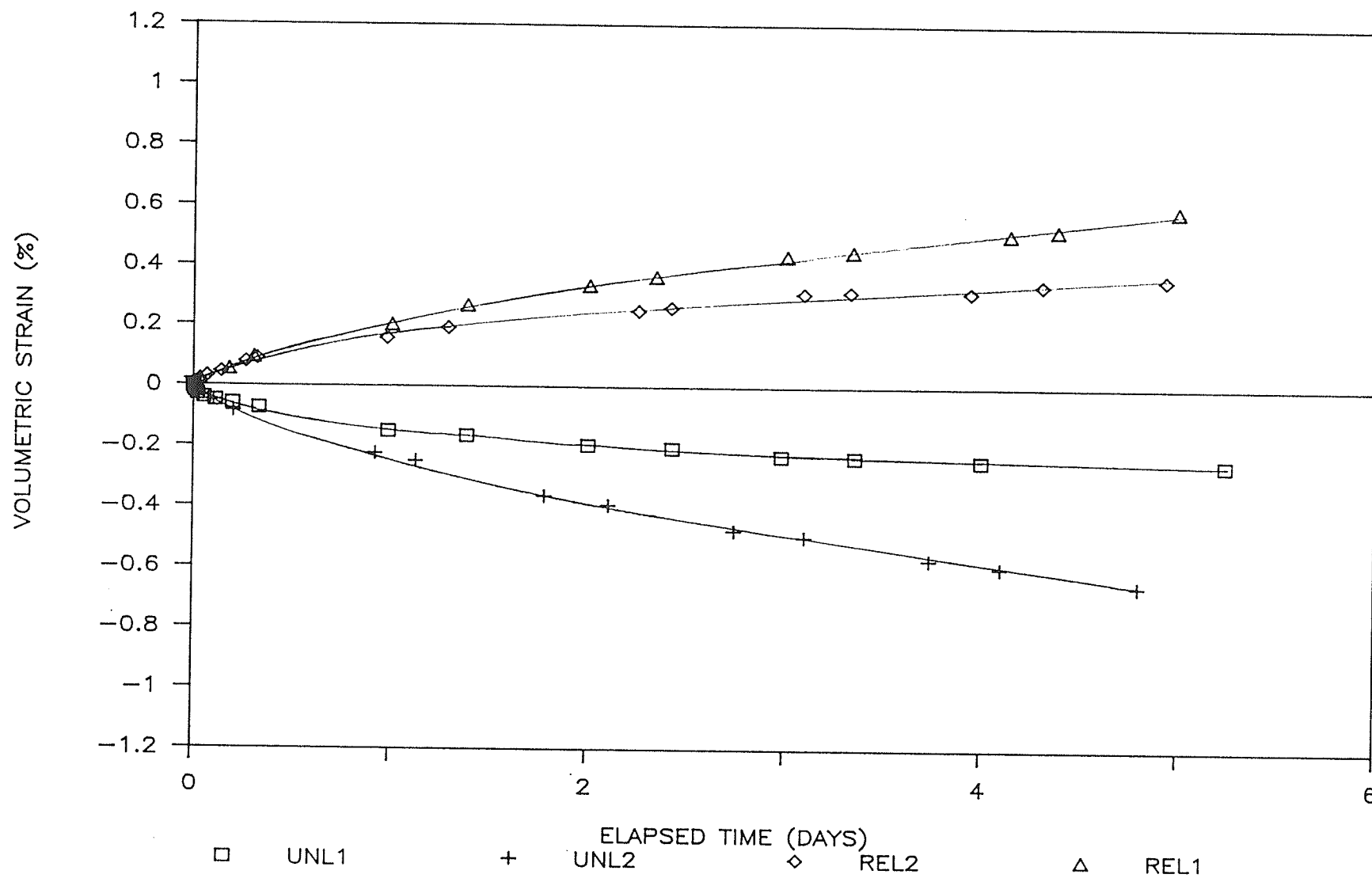


FIGURE 5.10- VOLUMETRIC STRAIN VERSUS CONSOLIDATION DURATION - T929 (INCREMENTAL/DECREMENTAL ISOTROPIC UNLOAD-RELOAD CYCLE)

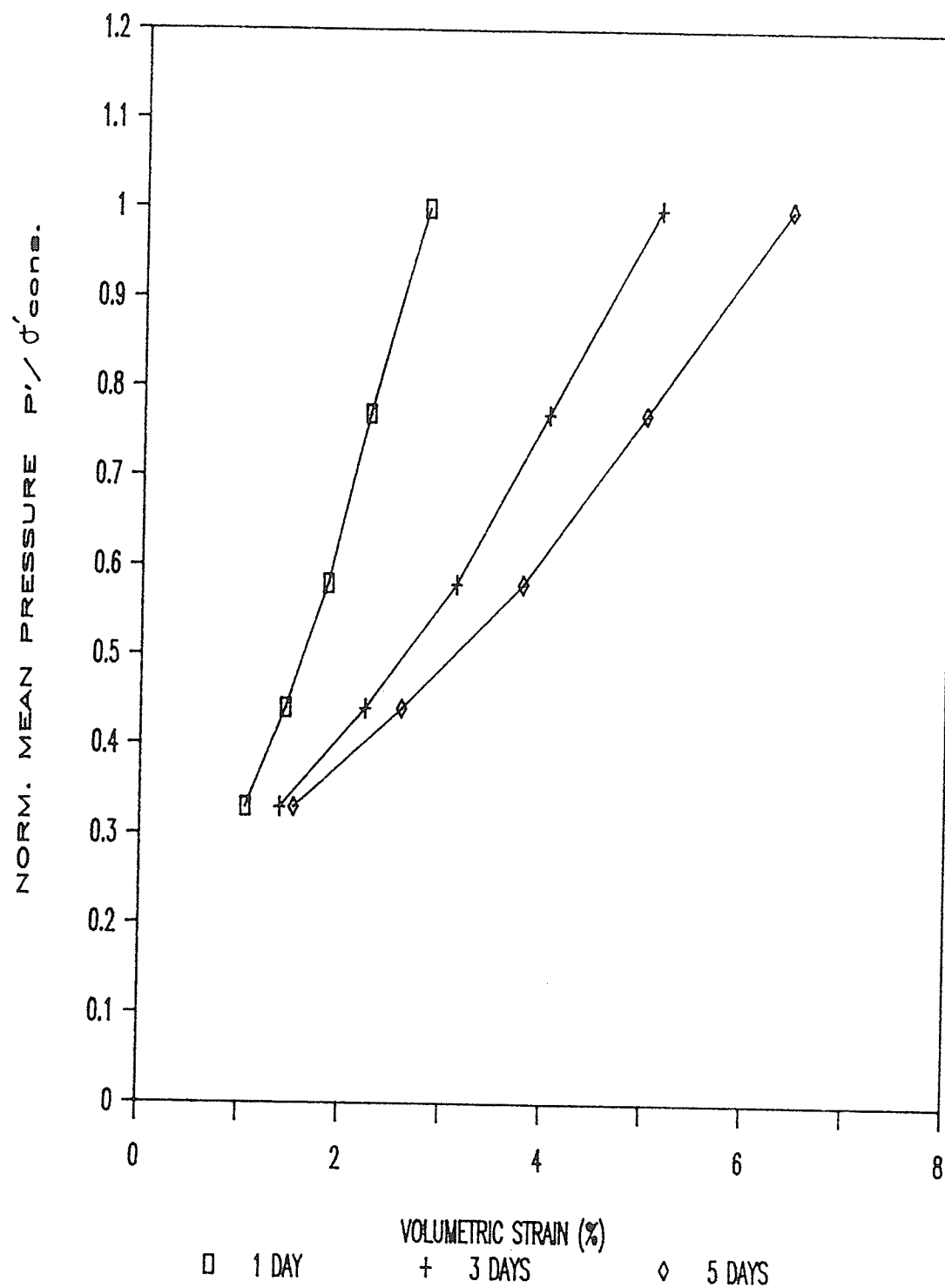


FIGURE 5.11 - NORMALIZED ISOTROPIC COMPRESSION CURVES p' VERSUS VOLUMETRIC STRAIN (1 DAY, 3 DAYS AND 5 DAYS STRAINS)-T915

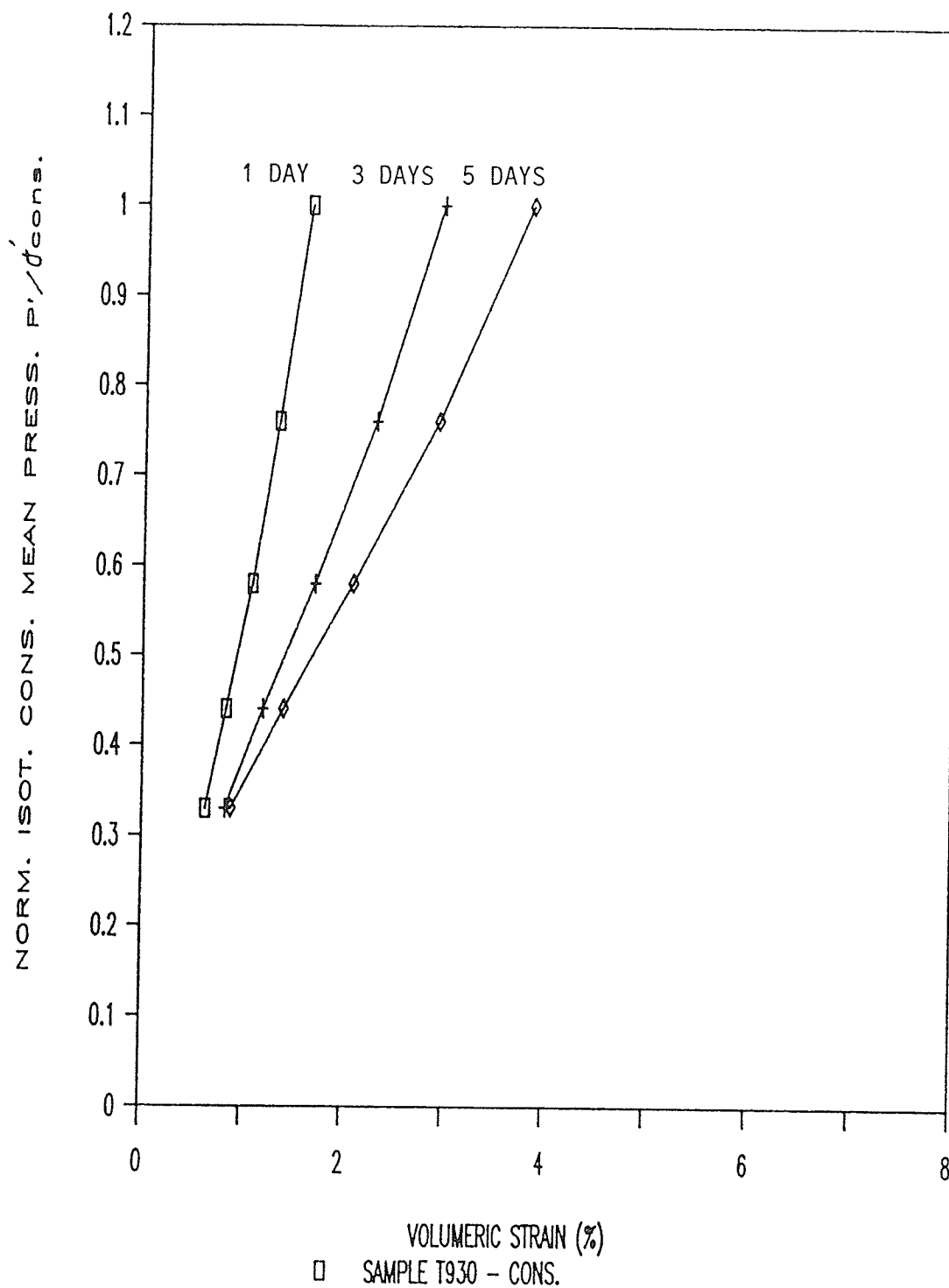


FIGURE 5.12 - NORMALIZED ISOTROPIC COMPRESSION CURVES p' VERSUS VOLUMETRIC STRAIN (1 DAY, 3DAYS AND 5DAYS) - T930

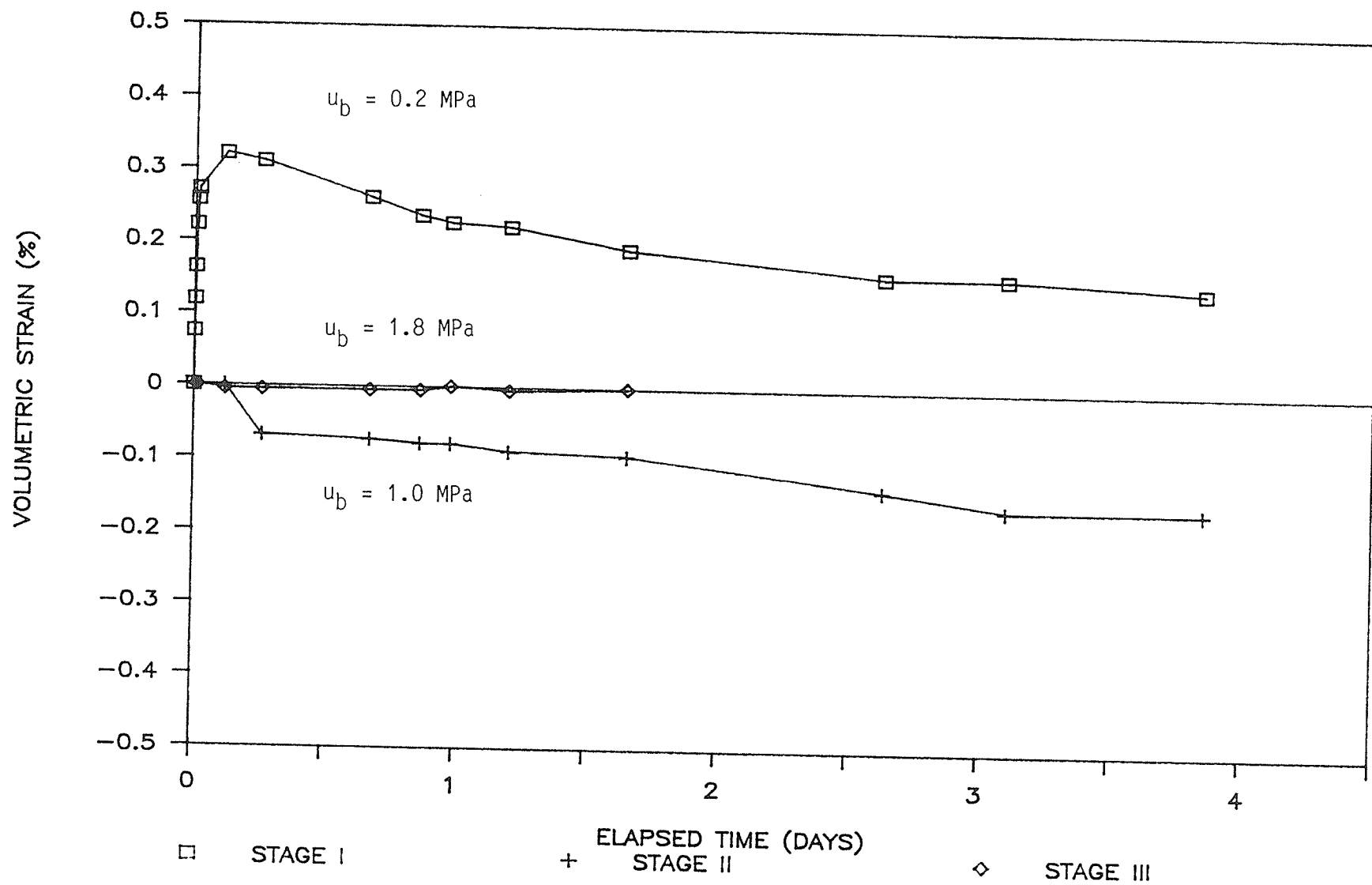


FIGURE 5.13 - VOLUMETRIC STRAIN VERSUS TIME RELATIONSHIP UNDER ELEVATED POREWATER PRESSURE - T927

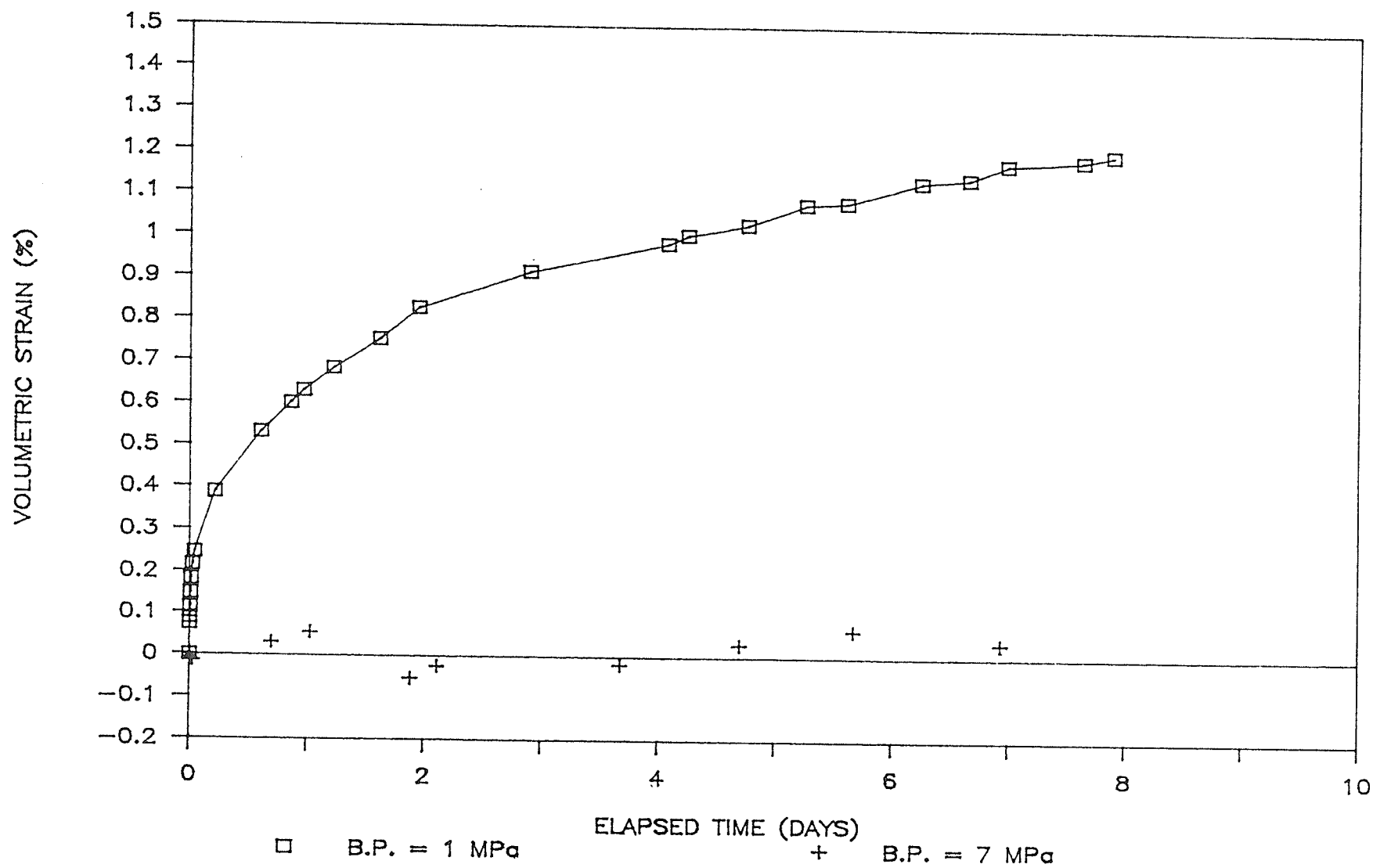


FIGURE 5.14 - VOLUMETRIC STRAIN VERSUS TIME RELATIONSHIP UNDER ELEVATED POREWATER PRESSURE - T933

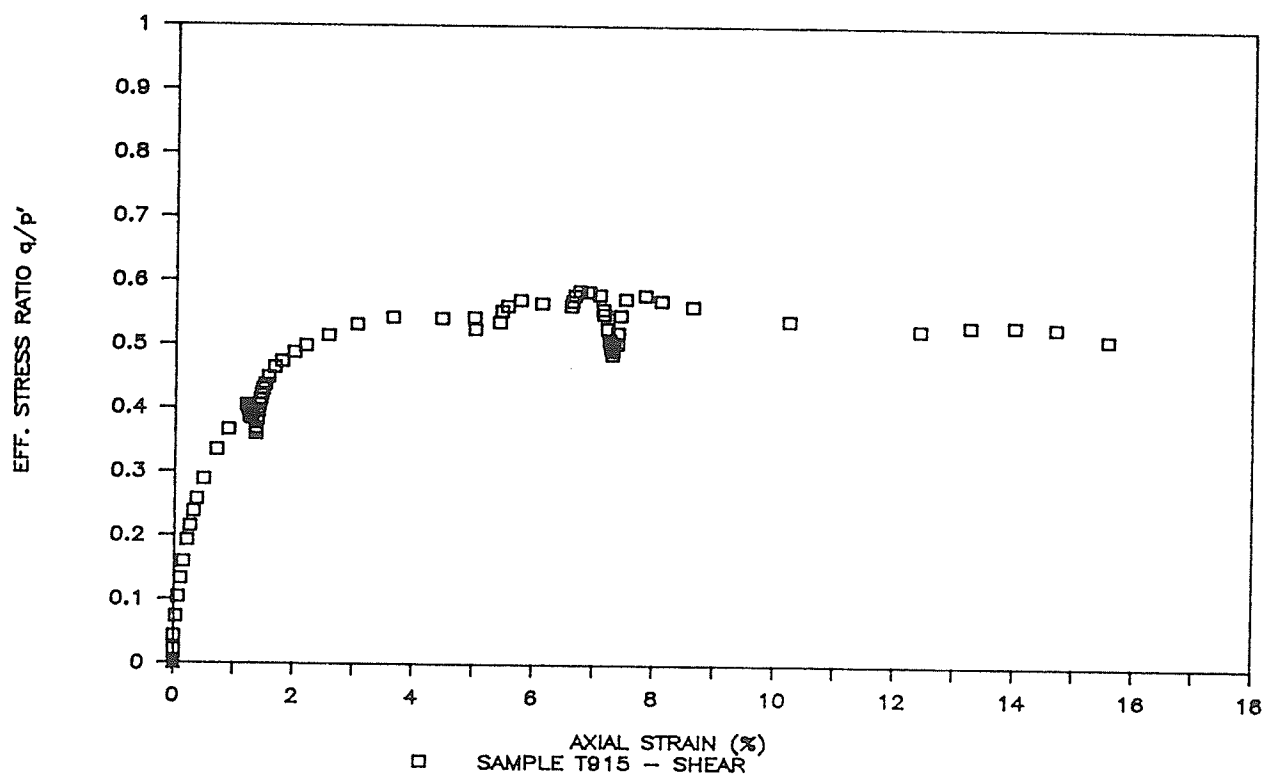
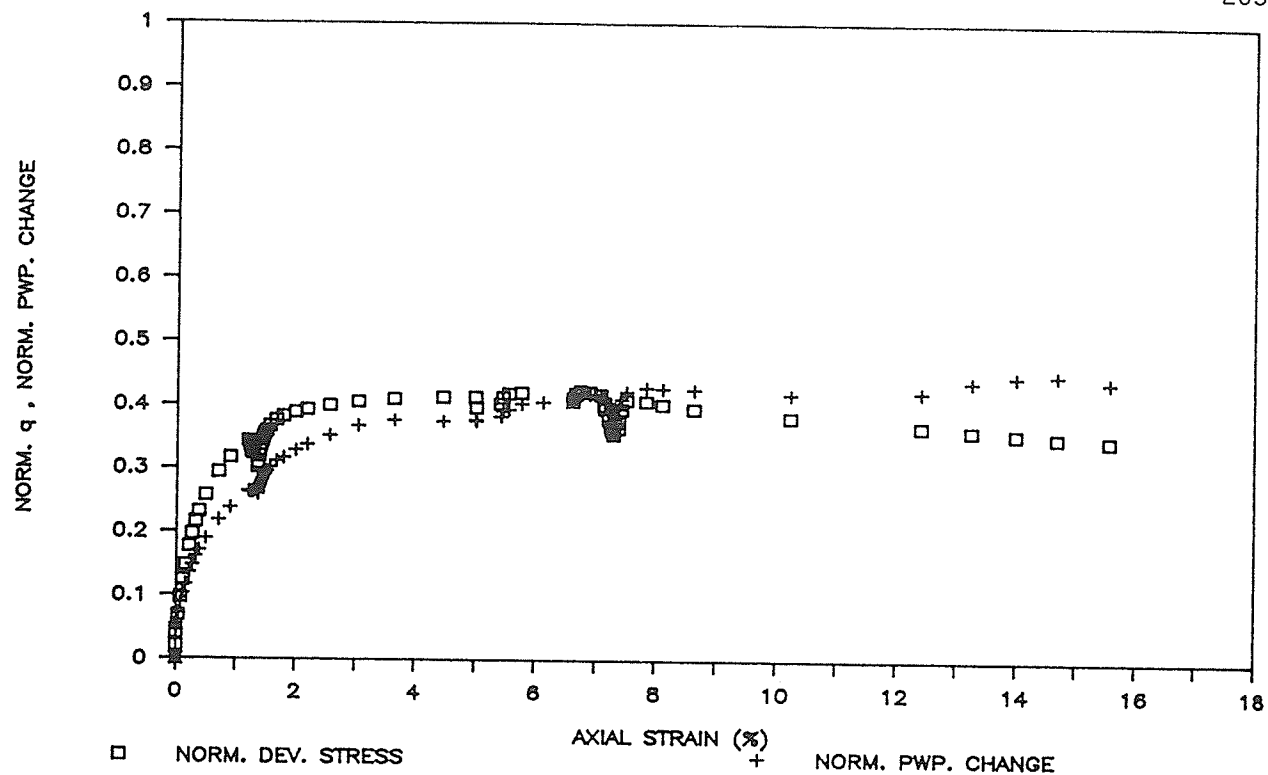


FIGURE 5.15 - STRESS-STRAIN-POREWATER PRESSURE-STRESS RATIO CURVES
IN UNDRAINED TRIAXIAL COMPRESSION - T915

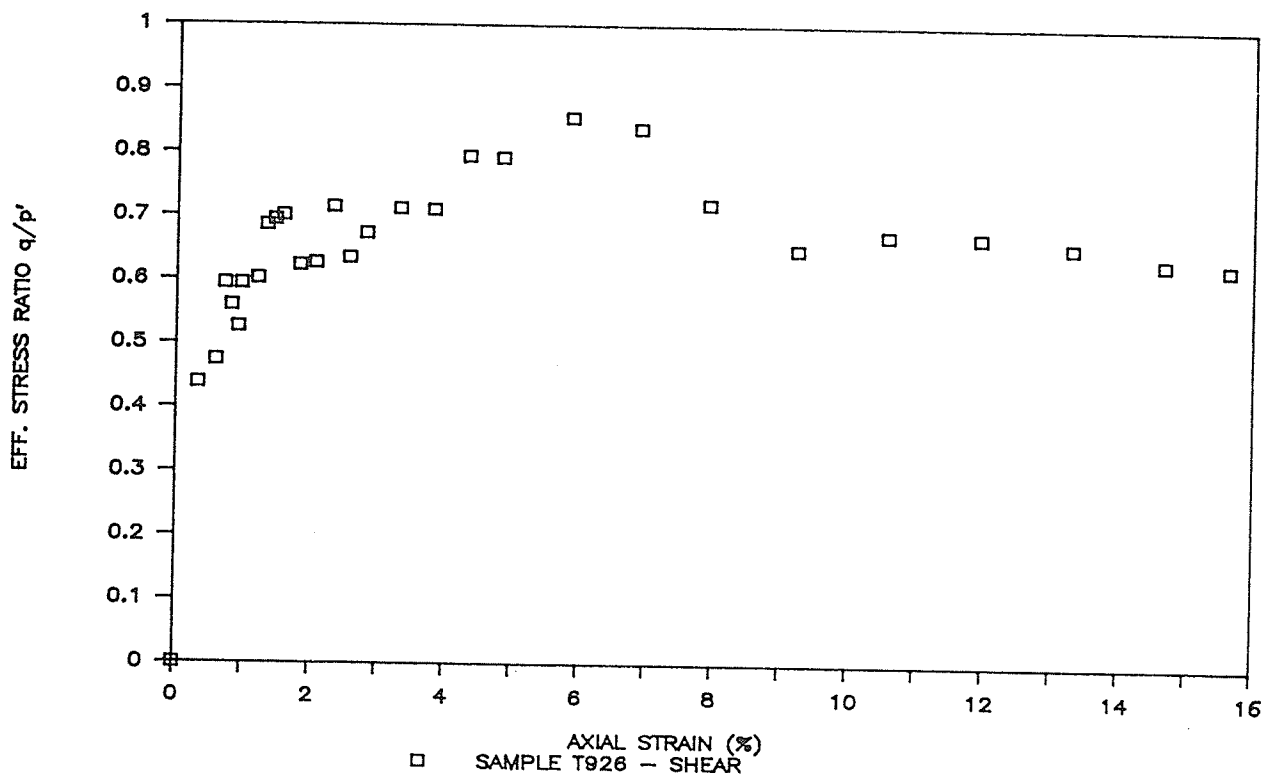
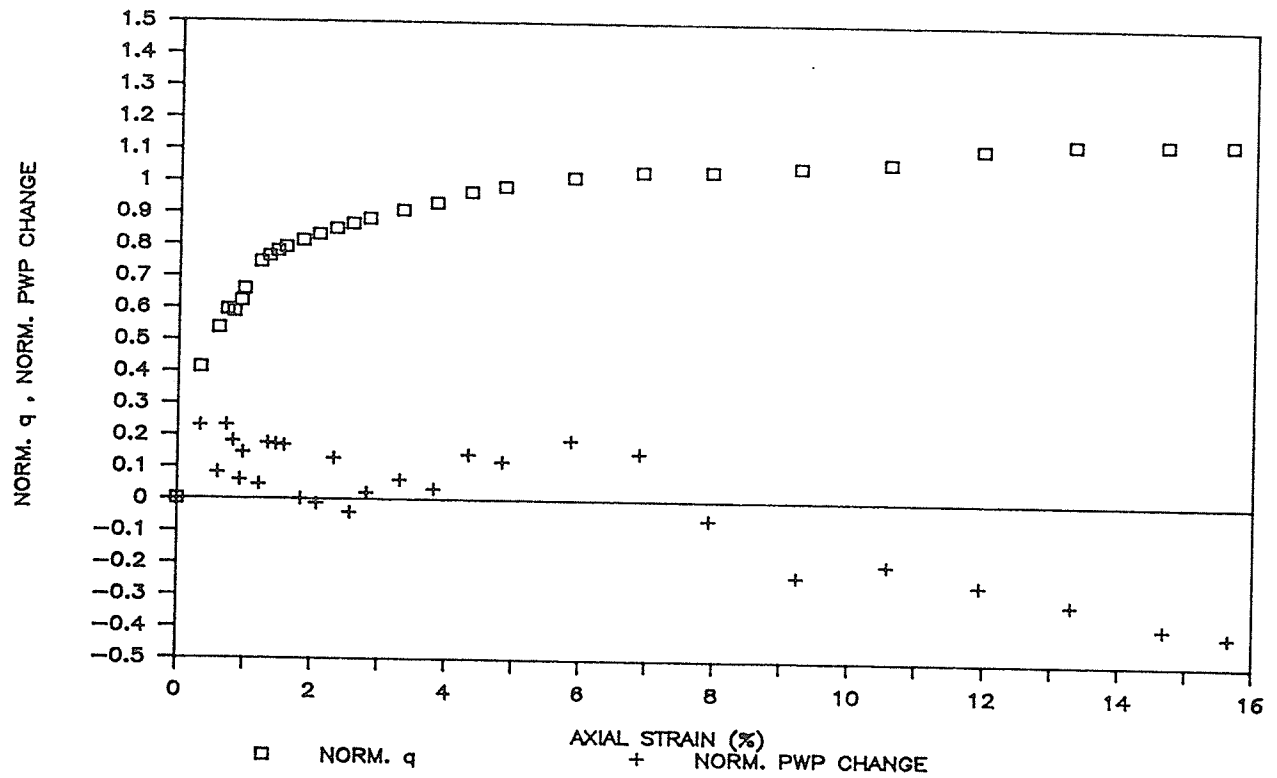


FIGURE 5.16 - STRESS-STRAIN-POREWATER PRESSURE-STRESS RATIO CURVES
IN UNDRAINED TRIAXIAL COMPRESSION - T926

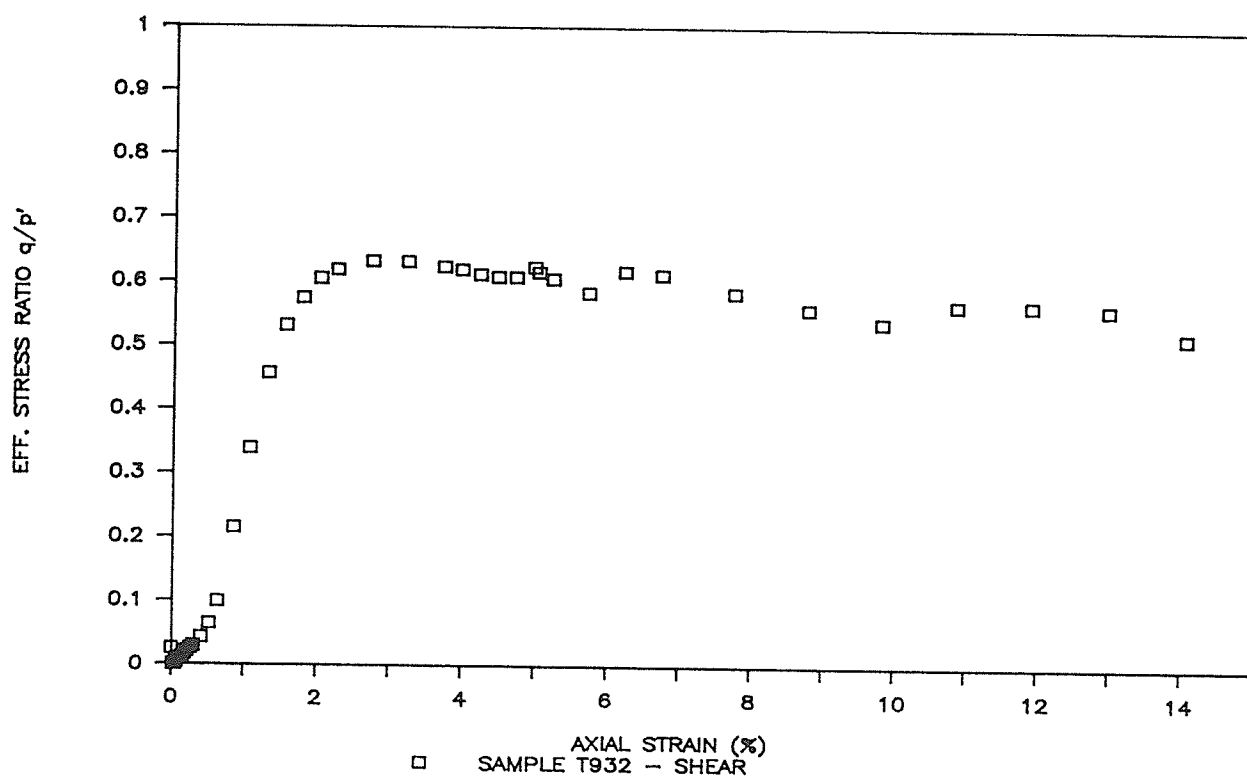
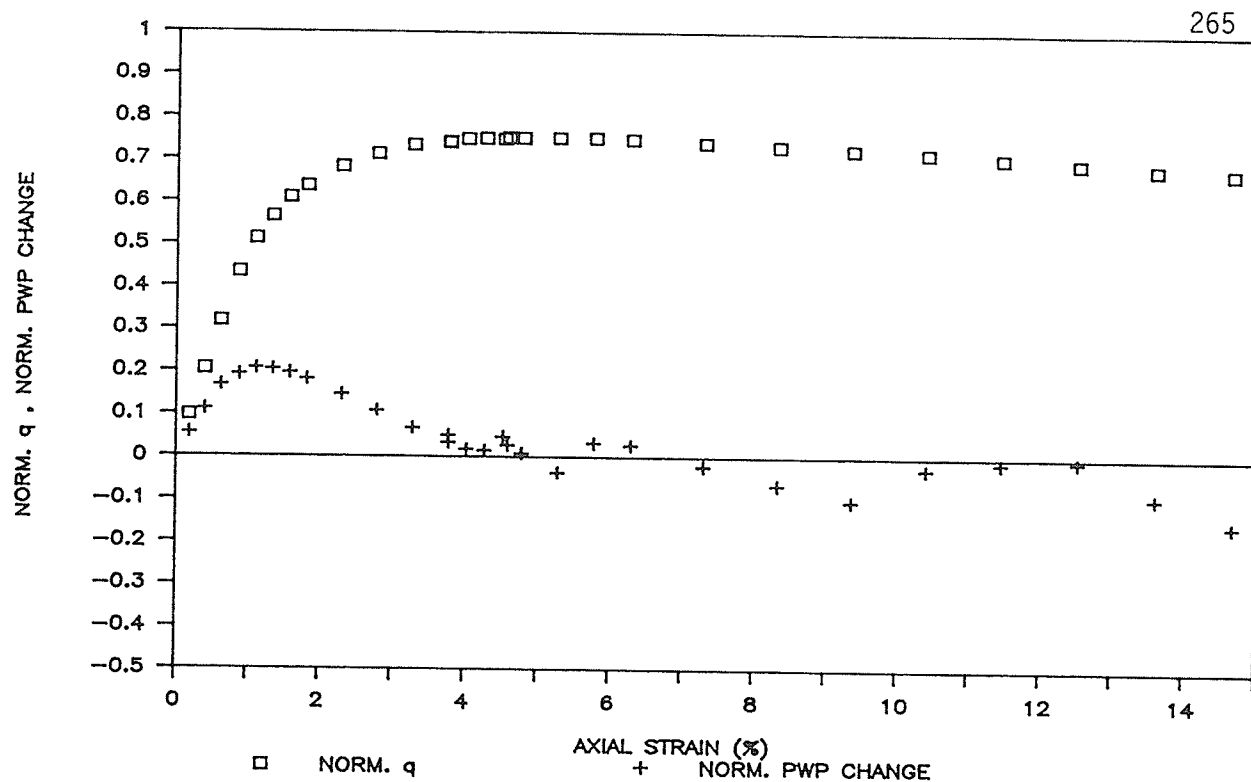


FIGURE 5.17 - STRESS-STRAIN-POREWATER PRESSURE-STRESS RATIO CURVES
IN UNDRAINED TRIAXIAL COMPRESSION - T932

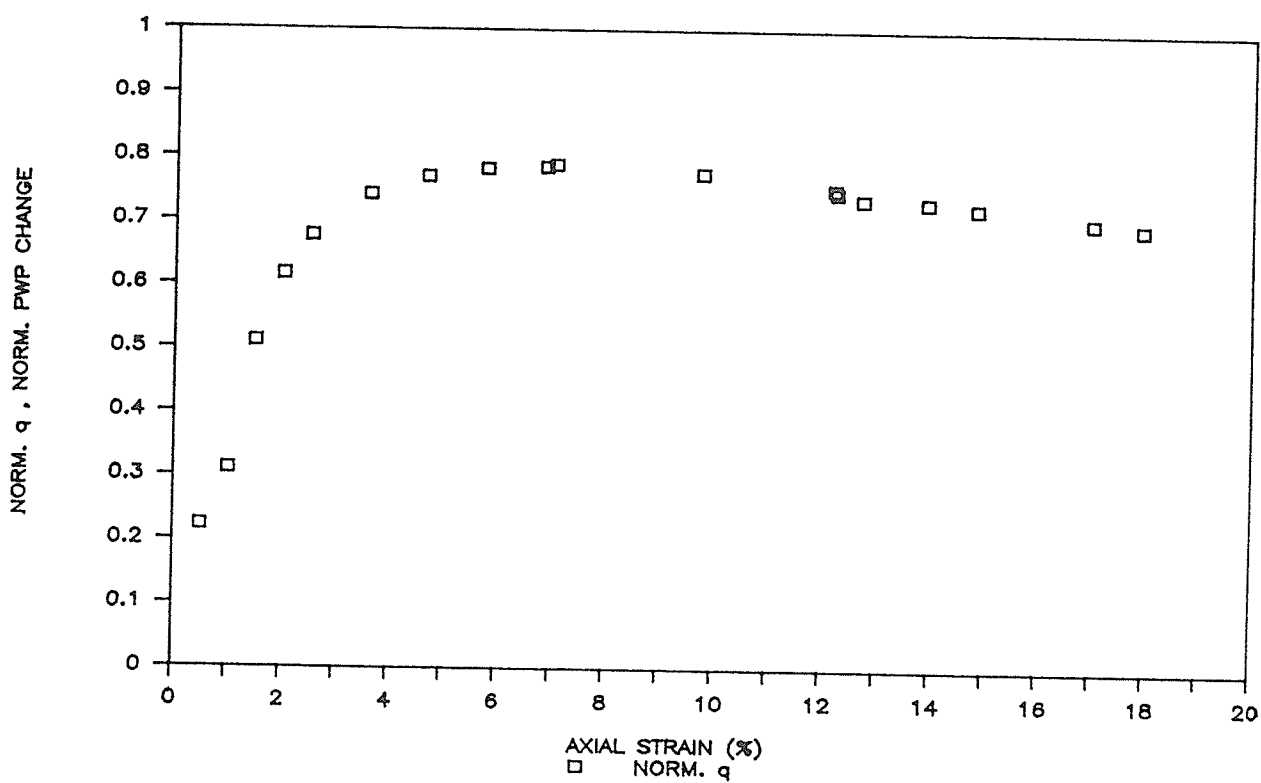
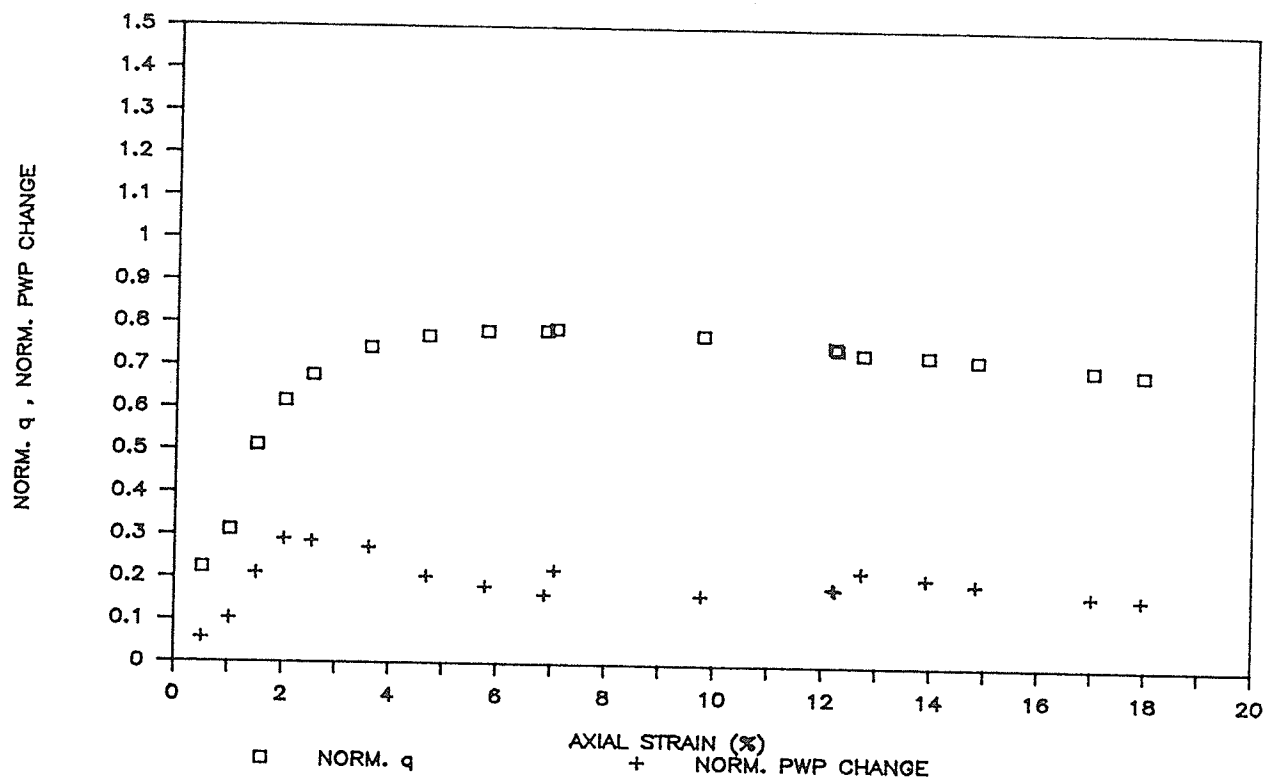


FIGURE 5.18 - STRESS-STRAIN-POREWATER PRESSURE-STRESS RATIO CURVES
IN UNDRAINED TRIAXIAL COMPRESSION - T927

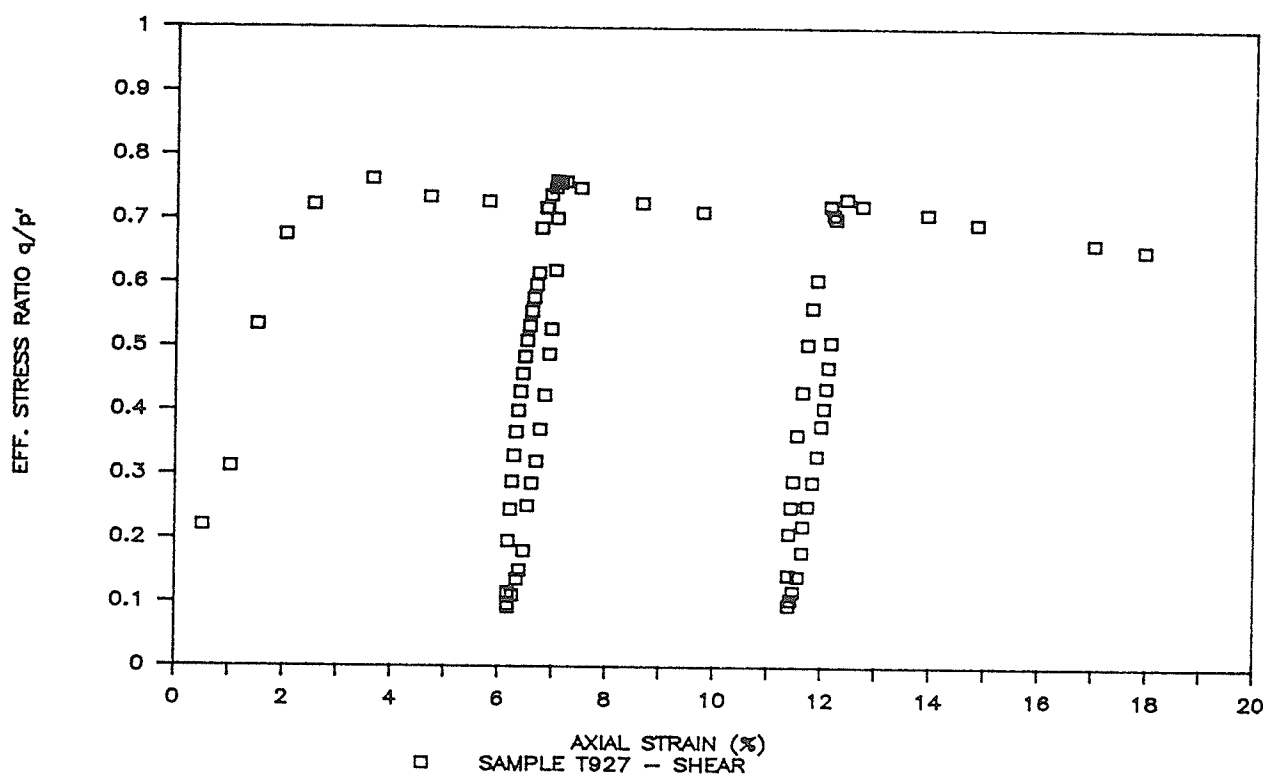
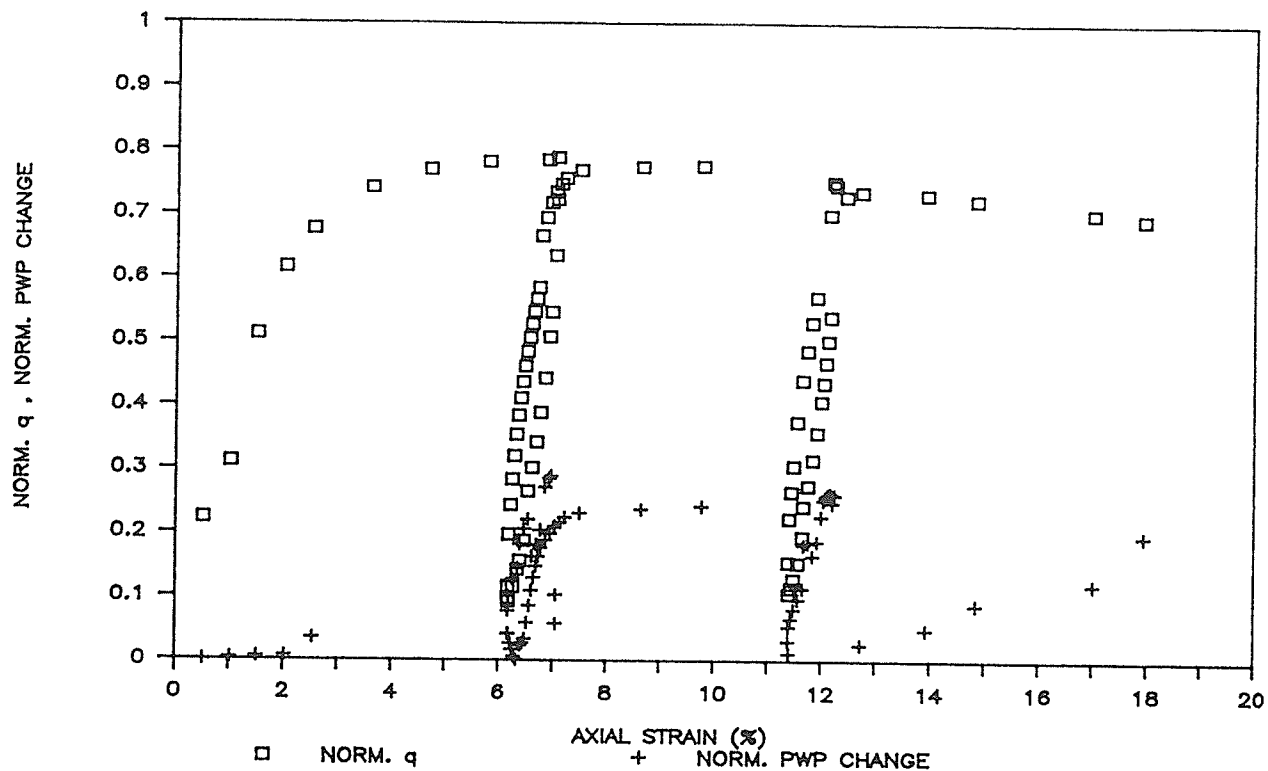


FIGURE 5.18 - STRESS-STRAIN-POREWATER PRESSURE-STRESS RATIO CURVES
IN UNDRAINED TRIAXIAL COMPRESSION - T927

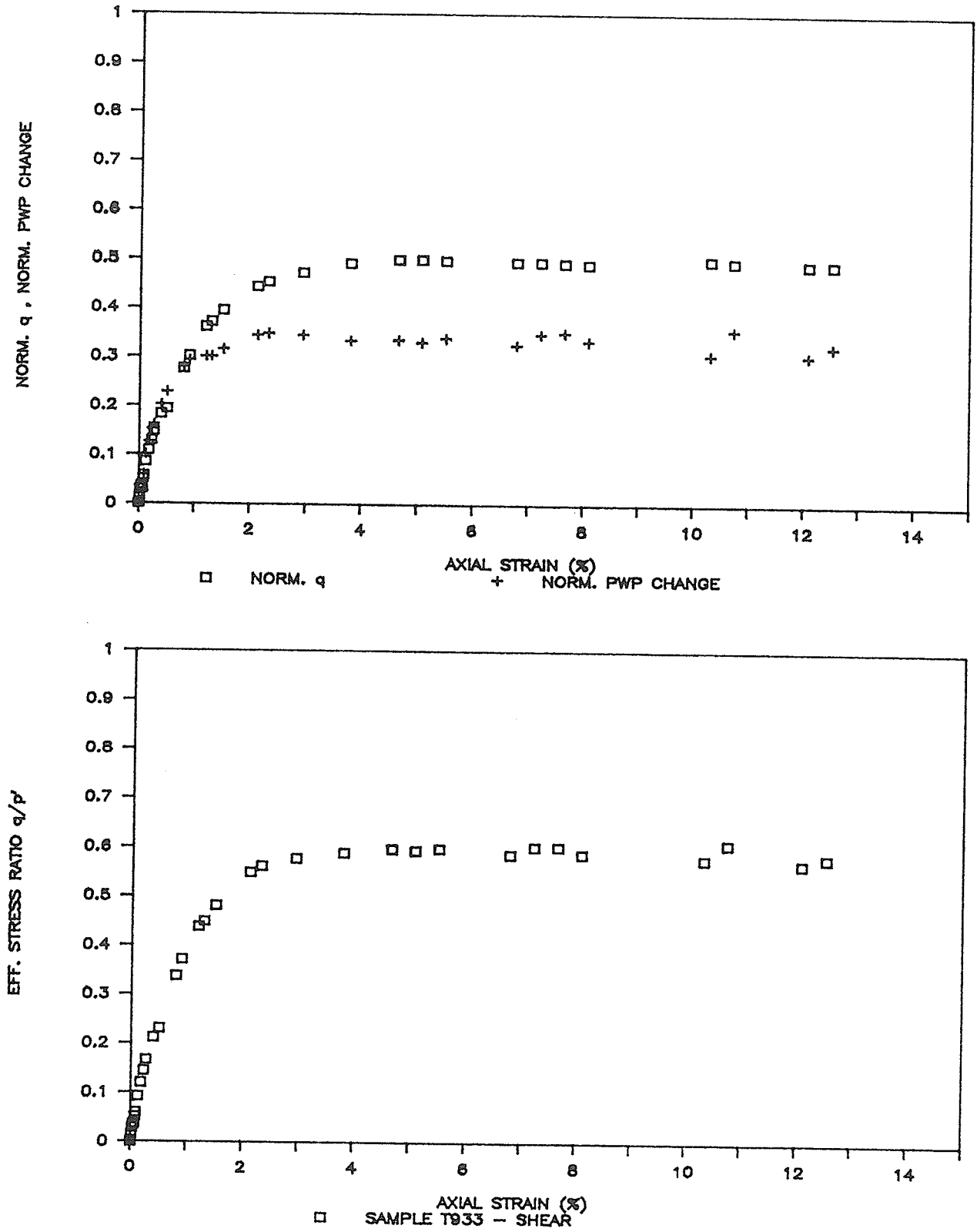


FIGURE 5.19 - STRESS-STRAIN-POREWATER PRESSURE-STRESS RATIO CURVES
IN UNDRAINED TRIAXIAL COMPRESSION - T933

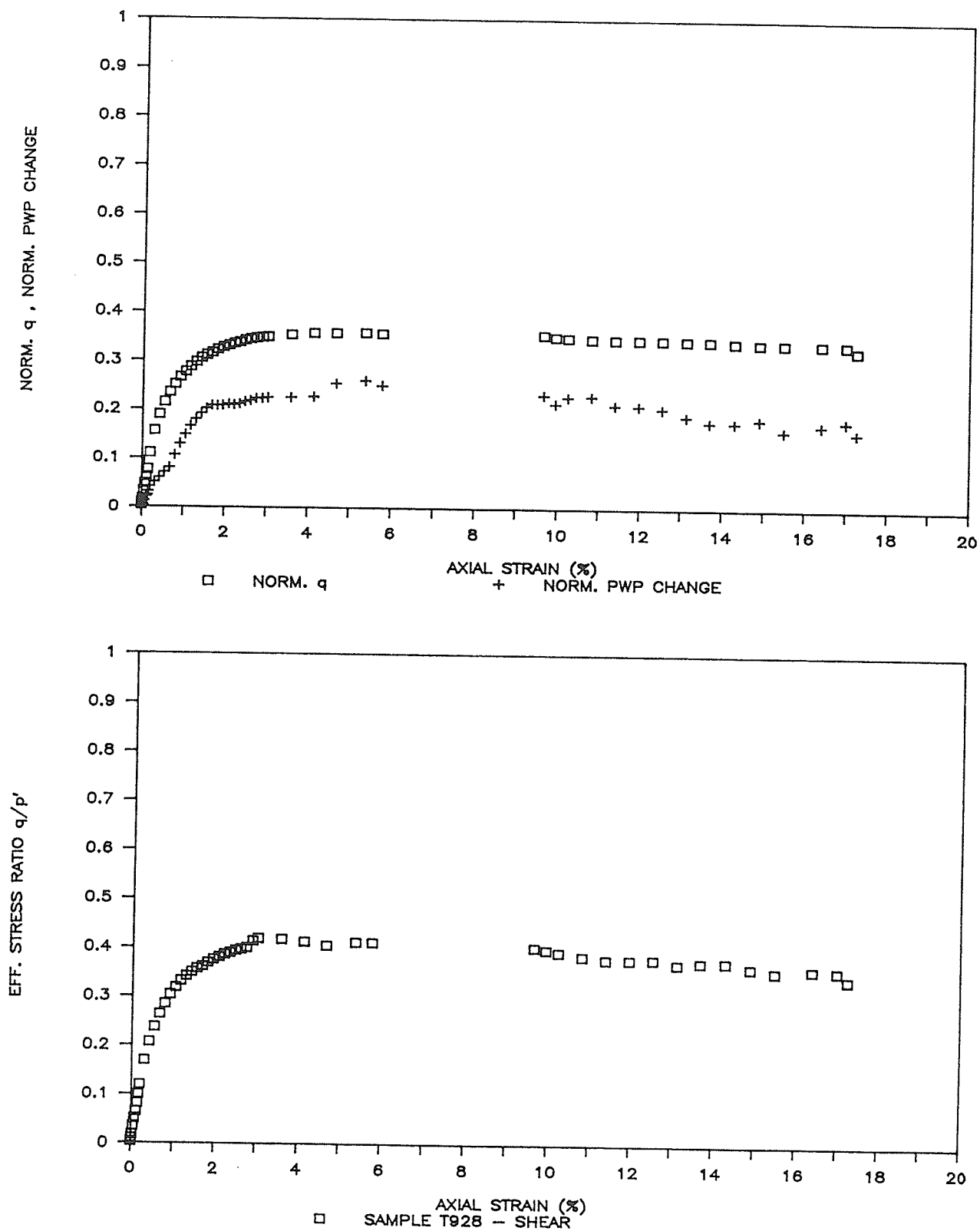


FIGURE 5.20 - STRESS-STRAIN-POREWATER PRESSURE-STRESS RATIO CURVES
IN UNDRAINED TRIAXIAL COMPRESSION - T928

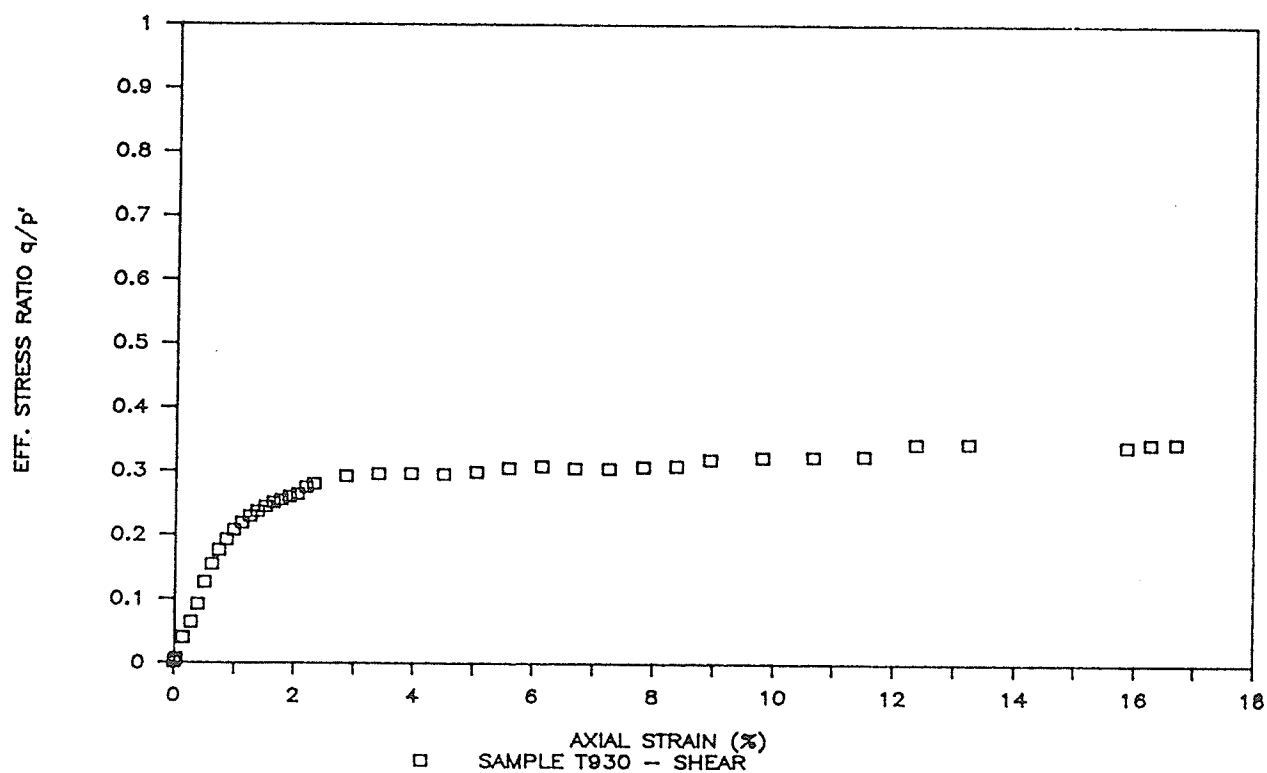
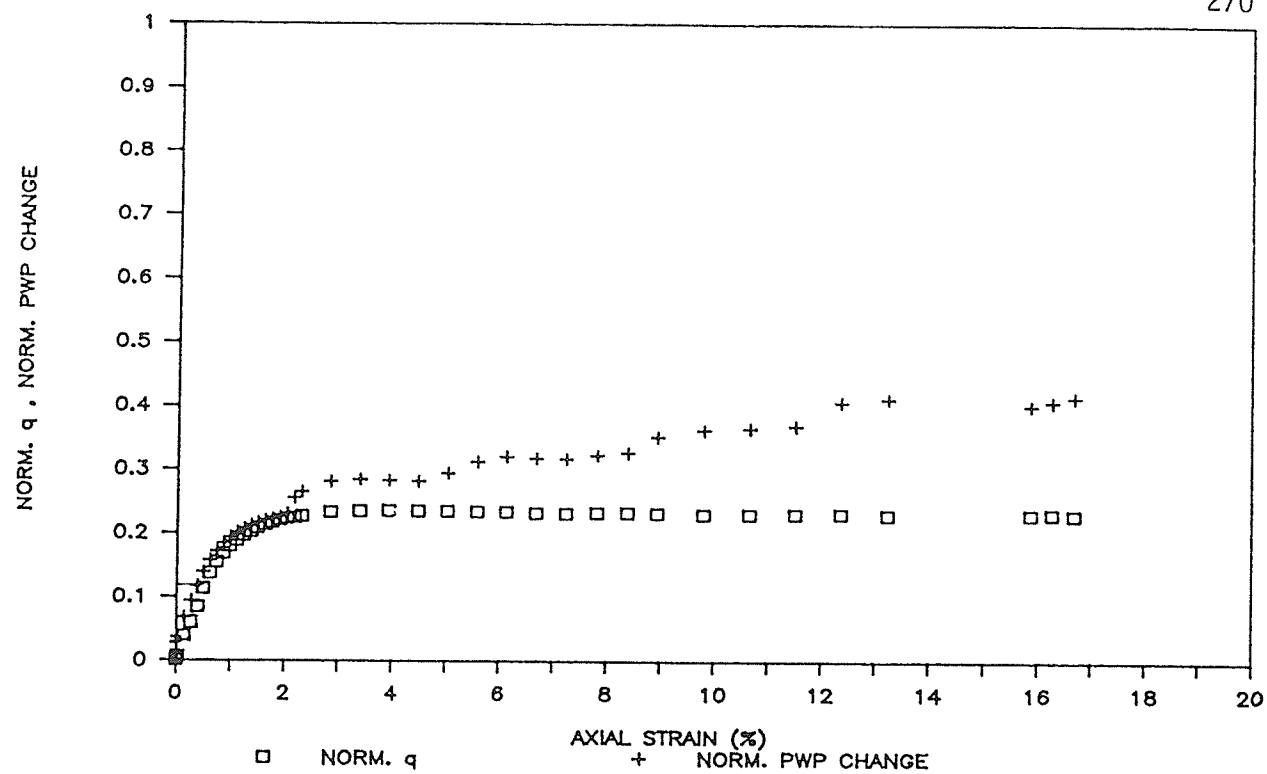


FIGURE 5.21 - STRESS-STRAIN-POREWATER PRESSURE-STRESS RATIO CURVES
IN UNDRAINED TRIAXIAL COMPRESSION - T930

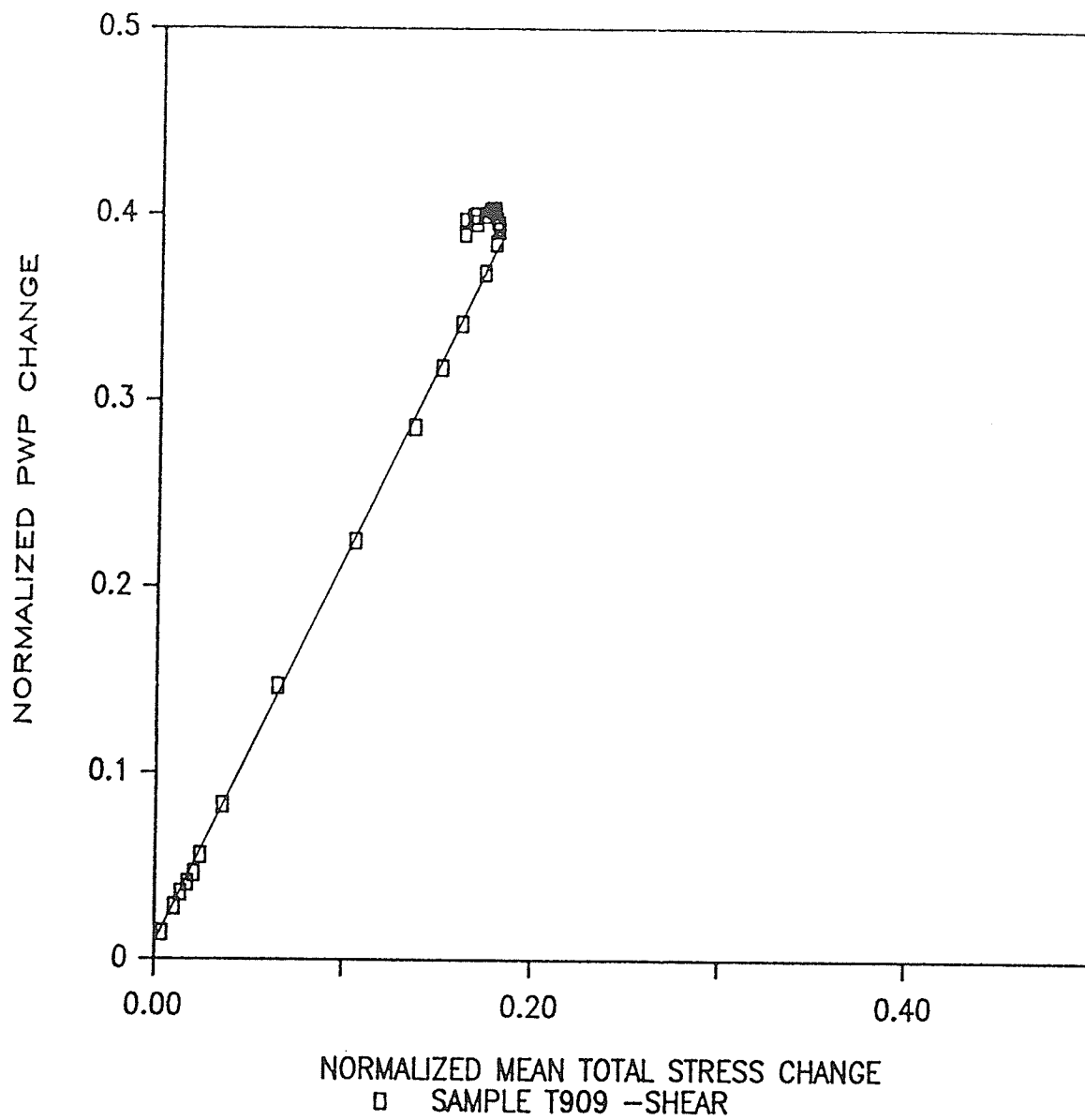


FIGURE 5.22 - NORMALIZED POREWATER PRESSURE CHANGES VERSUS MEAN
TOTAL PRESSURE CHANGES - T909

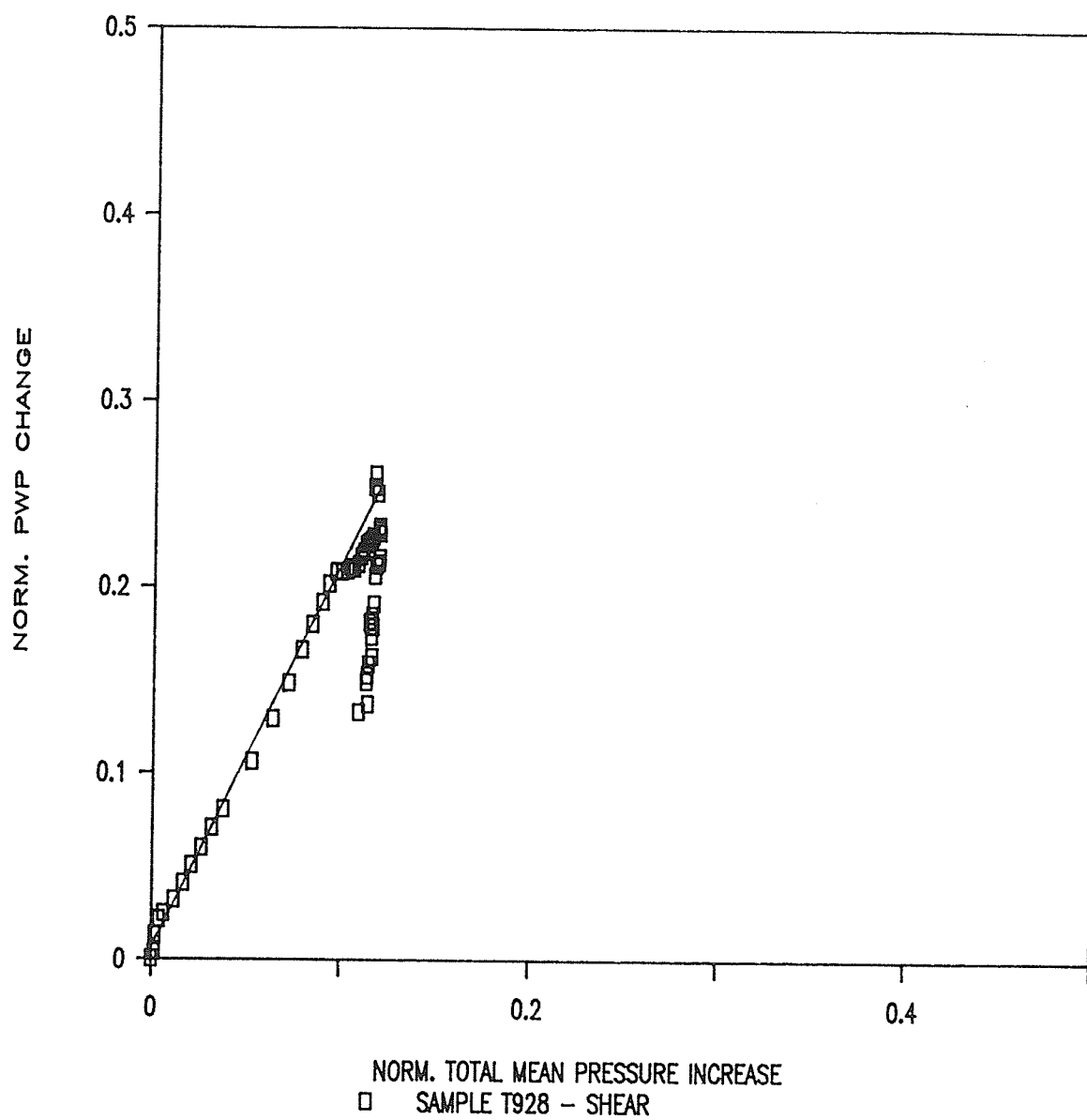


FIGURE 5.23 - NORMALIZED POREWATER PRESSURE CHANGES VERSUS MEAN TOTAL PRESSURE CHANGES - T928

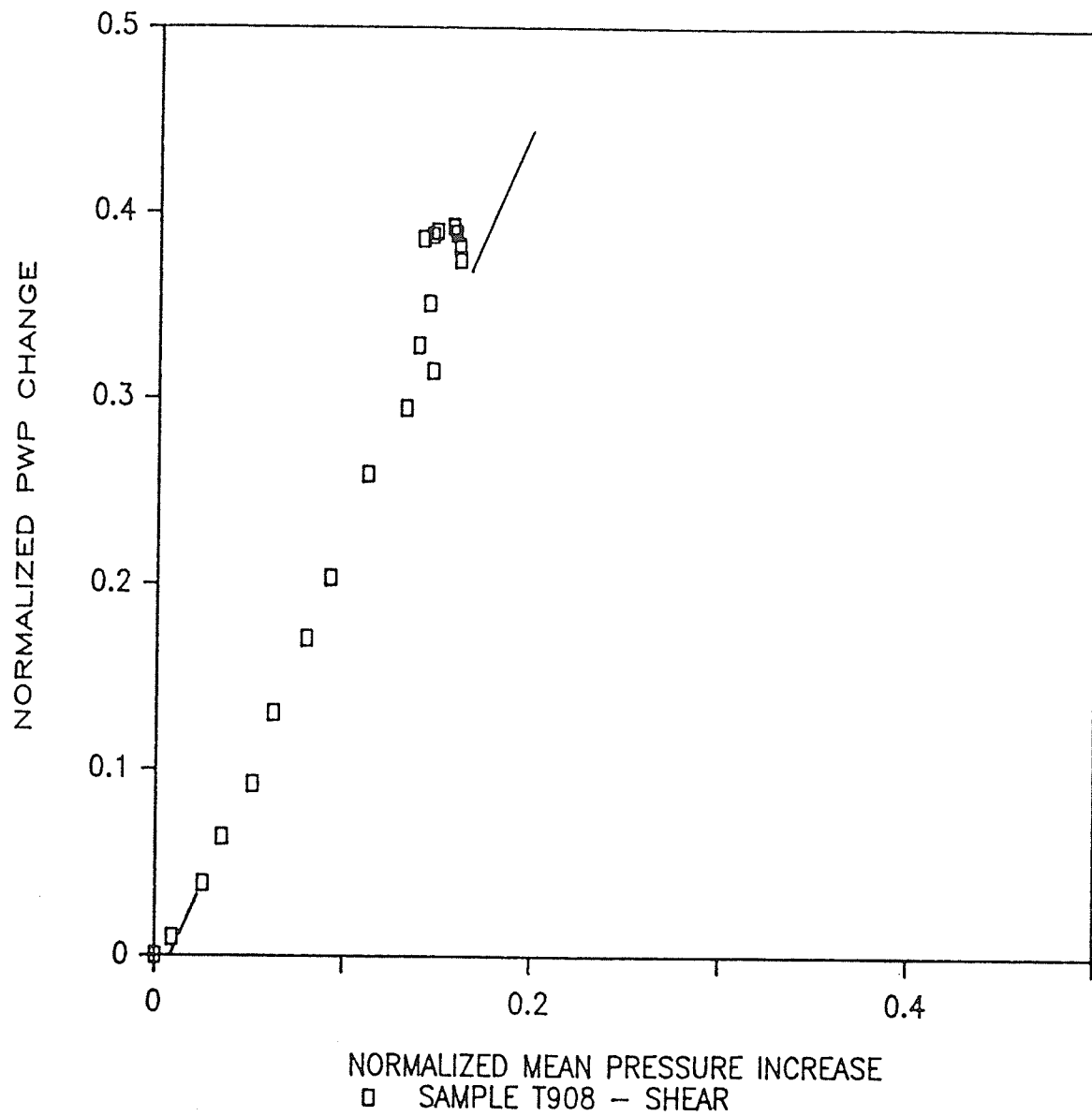


FIGURE 5.24 - NORMALIZED POREWATER PRESSURE CHANGES VERSUS MEAN
TOTAL PRESSURE CHANGES - T908

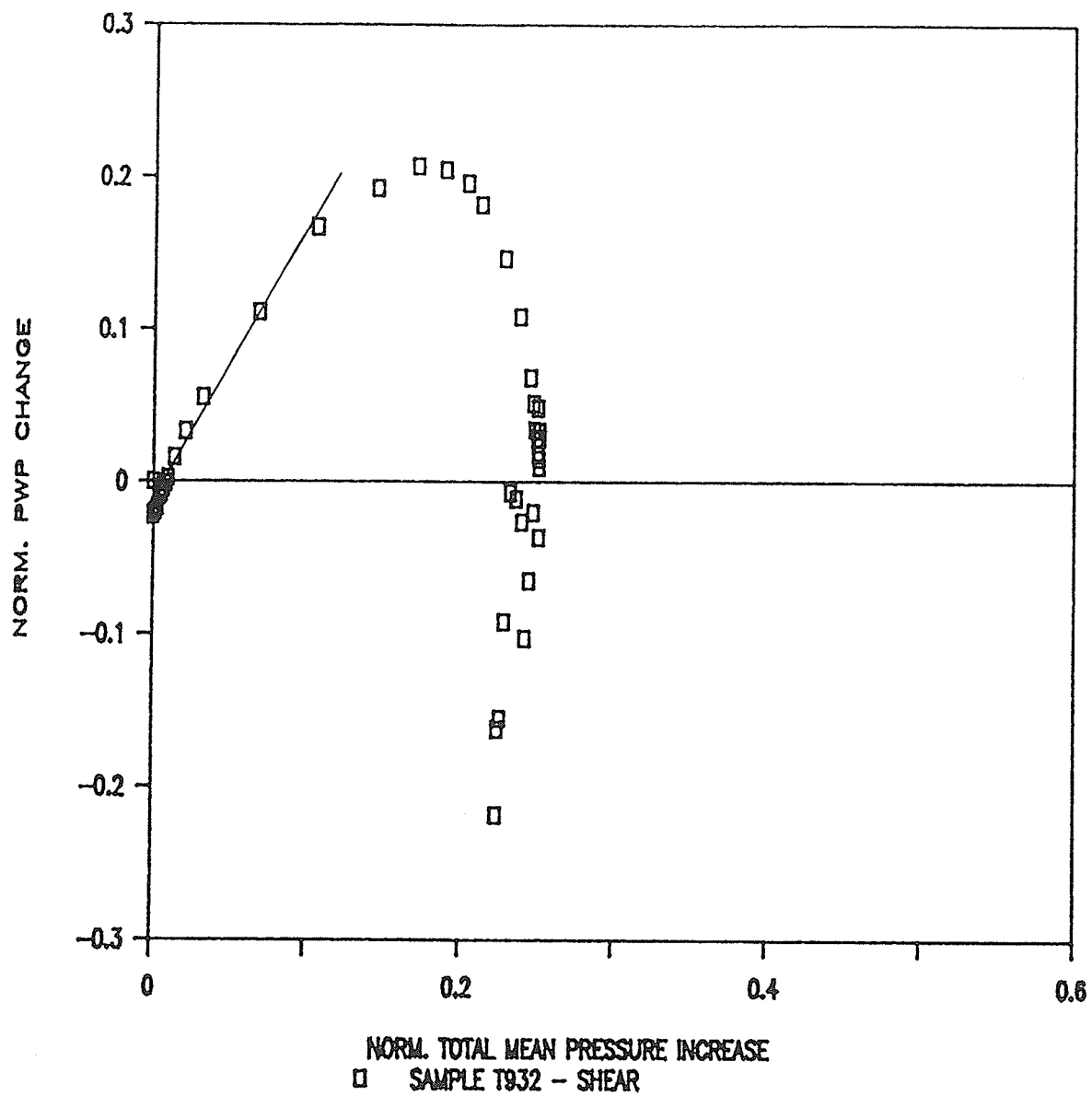


FIGURE 5.25 - NORMALIZED POREWATER PRESSURE CHANGES VERSUS MEAN
TOTAL PRESSURE CHANGES - T932

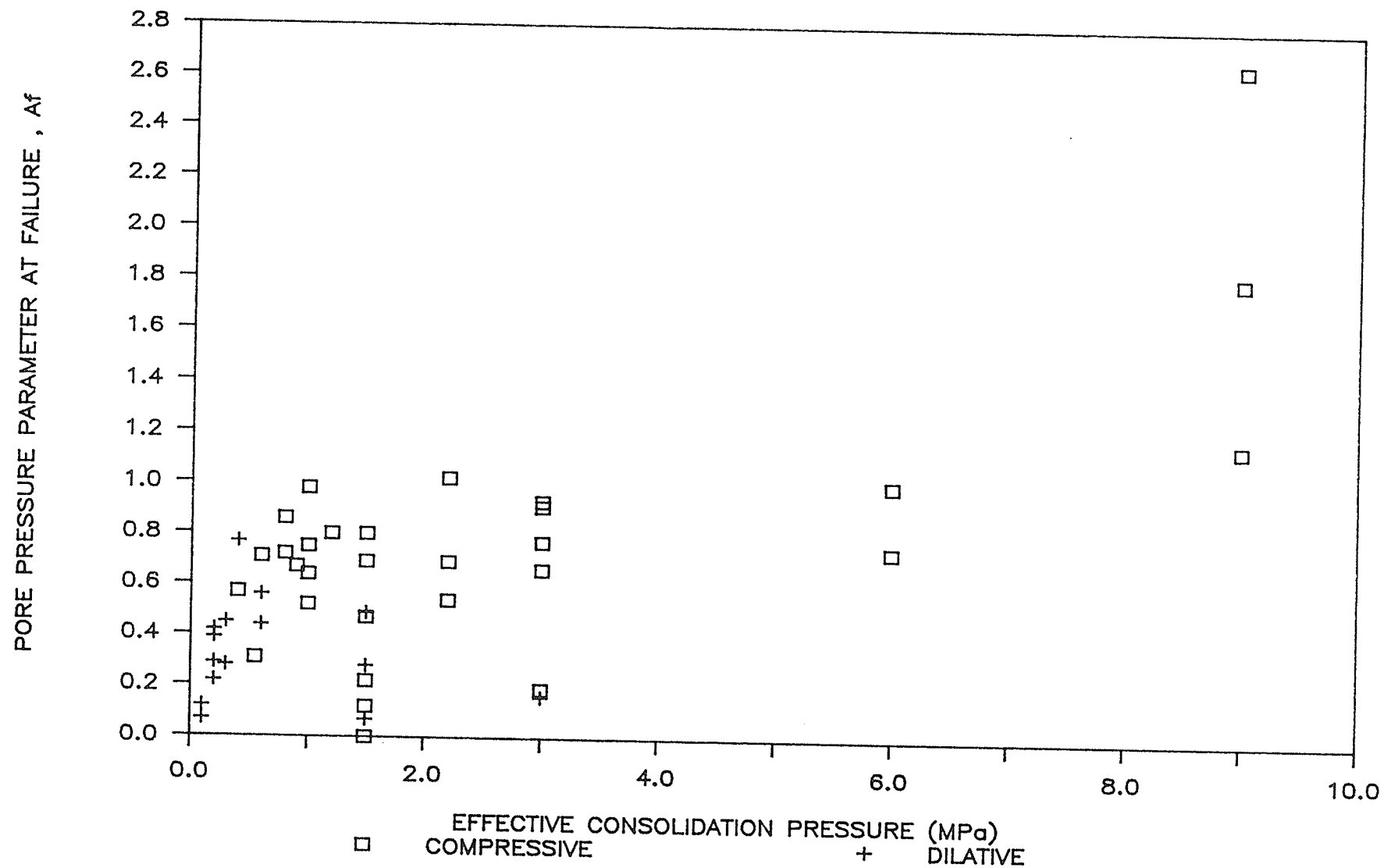


FIGURE 5.26 - RELATIONSHIP BETWEEN POREWATER PRESSURE PARAMETER AT FAILURE A_f AND EFFECTIVE CONSOLIDATION PRESSURE

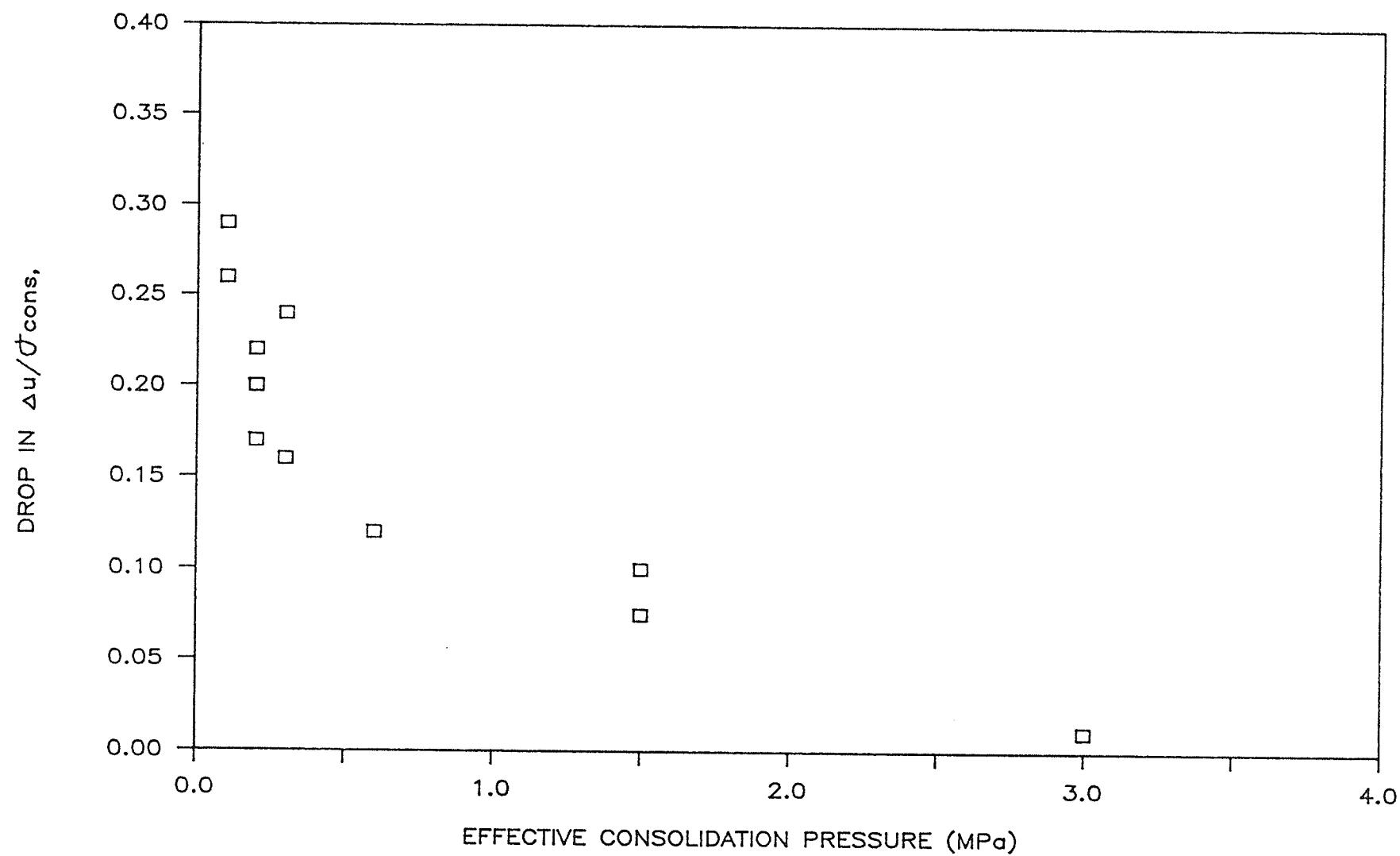


FIGURE 5.27 - RELATIONSHIP BETWEEN NORMALIZED DROP IN POREWATER PRESSURE $\Delta u / \sigma_{\text{cons}}$ BETWEEN PEAK AND 12% AXIAL STRAIN AND CONSOLIDATION PRESSURE

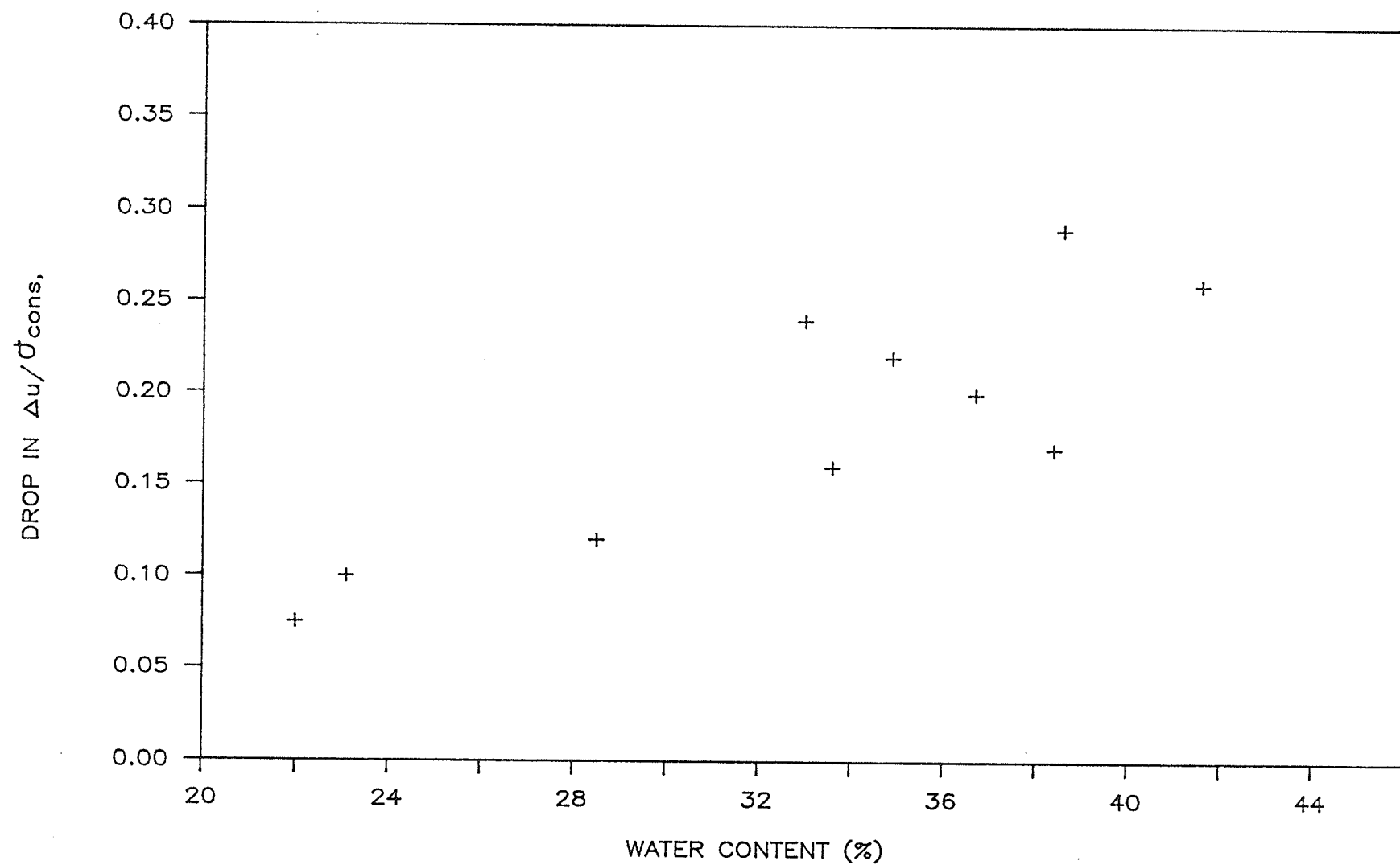


FIGURE 5.28 - RELATIONSHIP BETWEEN NORMALIZED DROP IN POREWATER PRESSURE $\Delta u / \sigma_{\text{cons}}$ BETWEEN PEAK AND 12% AXIAL STRAIN, AND WATER CONTENT

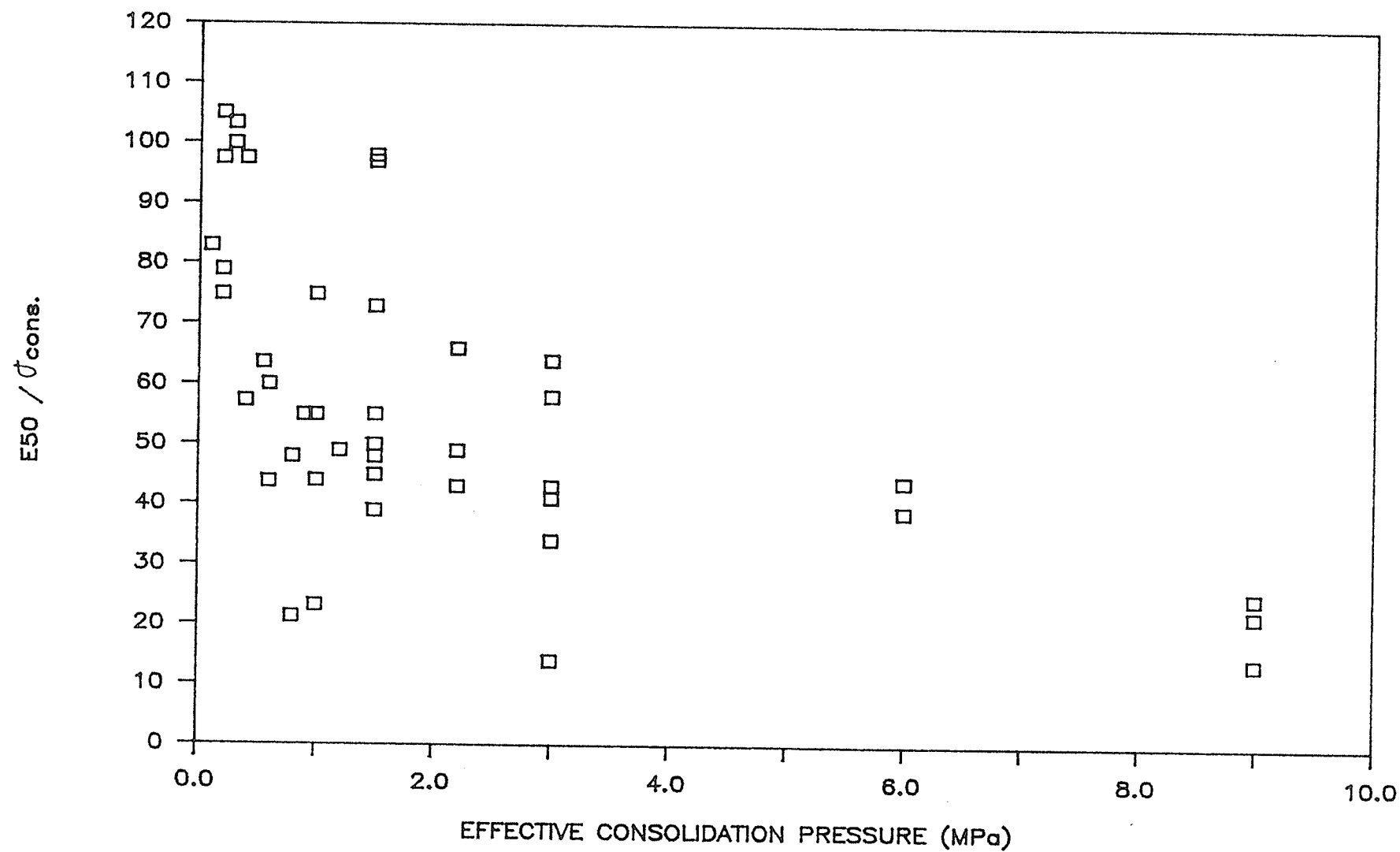


FIGURE 5.29 - RELATIONSHIP BETWEEN NORMALIZED UNDRAINED YOUNG'S MODULUS E_{50} / σ_{cons} AND CONSOLIDATION PRESSURE

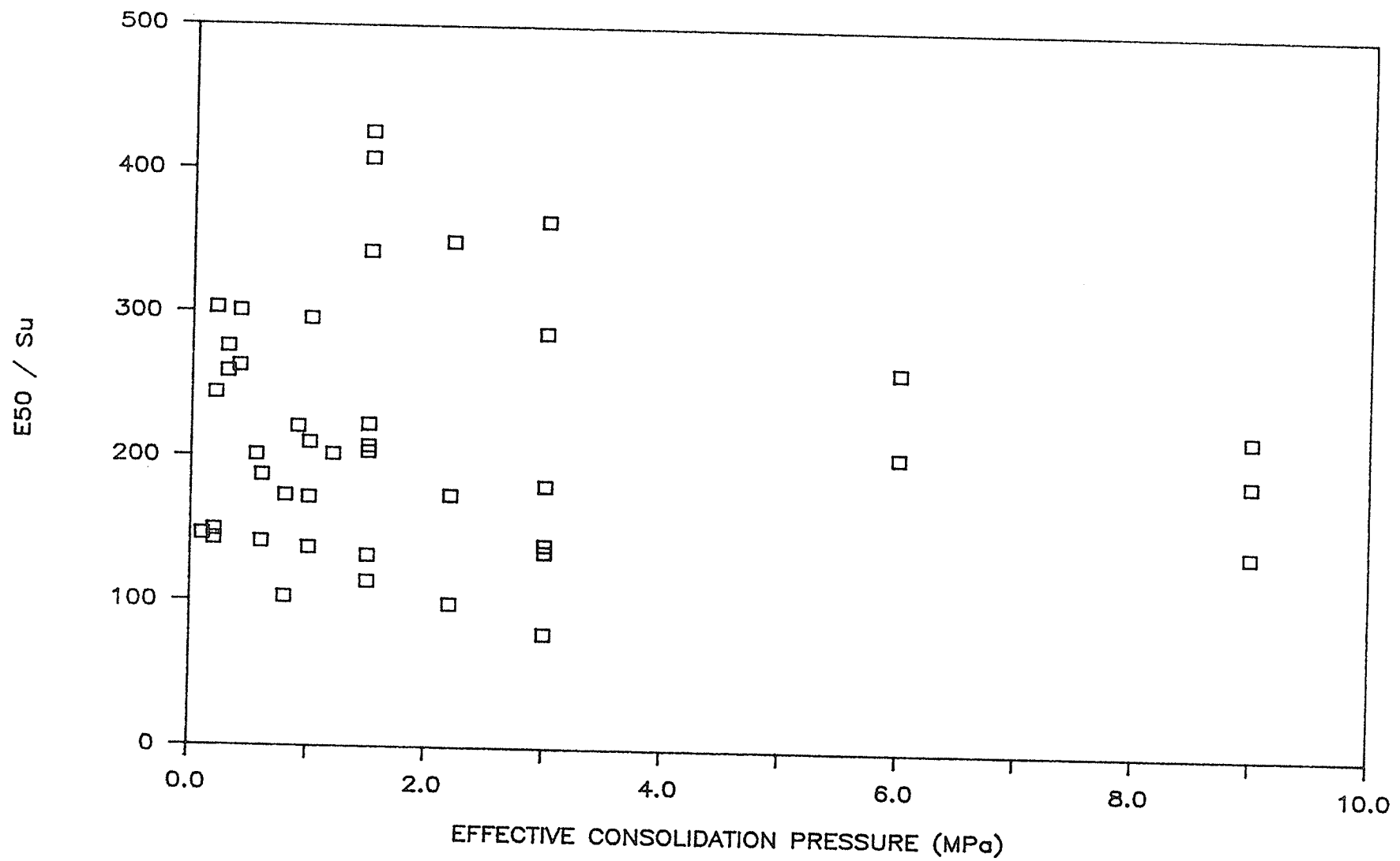


FIGURE 5.30 - RELATIONSHIP BETWEEN NORMALIZED UNDRAINED YOUNG'S MODULUS, E_{50}/S_u AND CONSOLIDATION PRESSURE

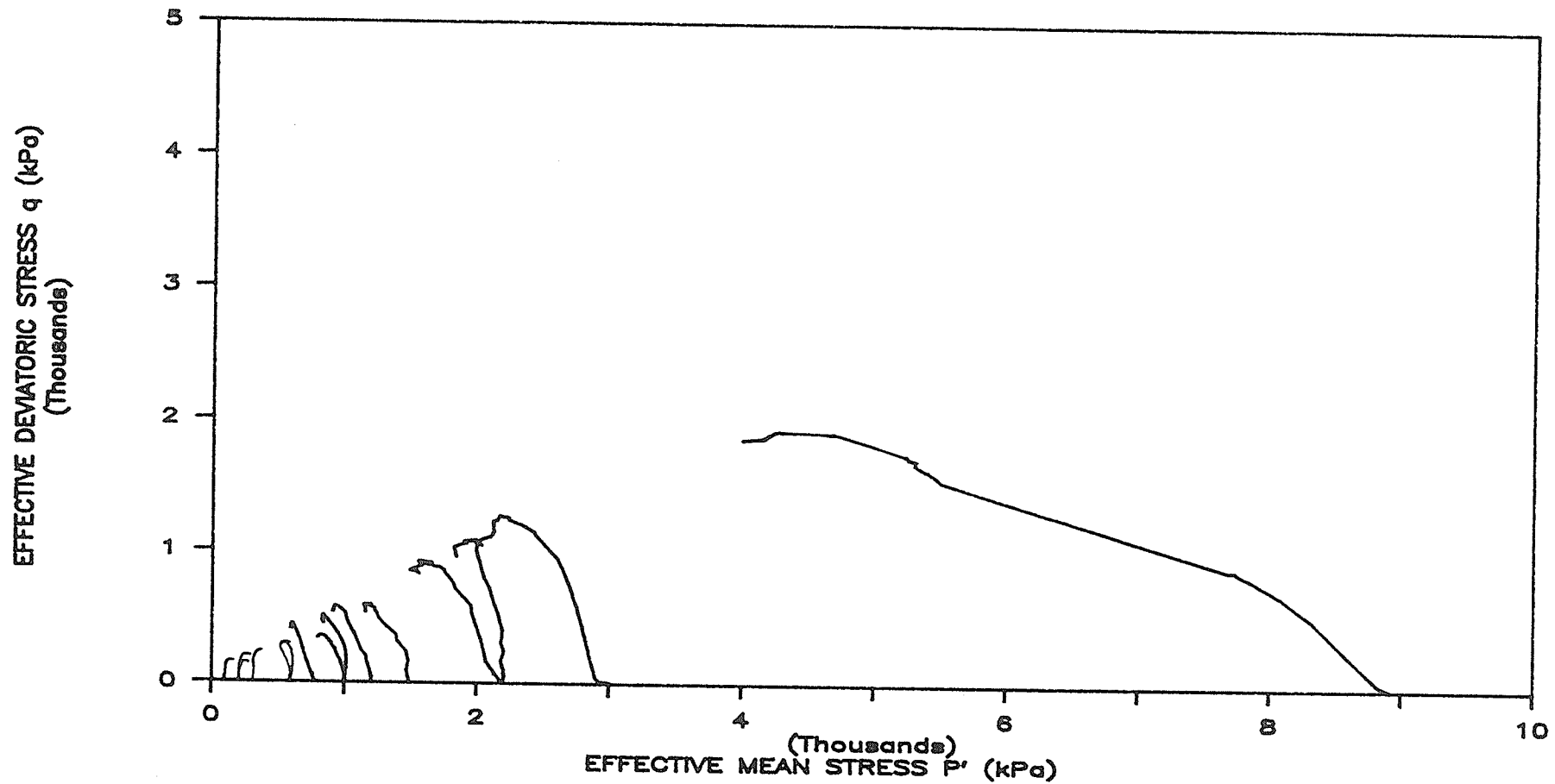


FIGURE 5.31 - EFFECTIVE STRESS PATHS FROM UNDRAINED SHEAR TESTS - LOW DENSITY SPECIMENS

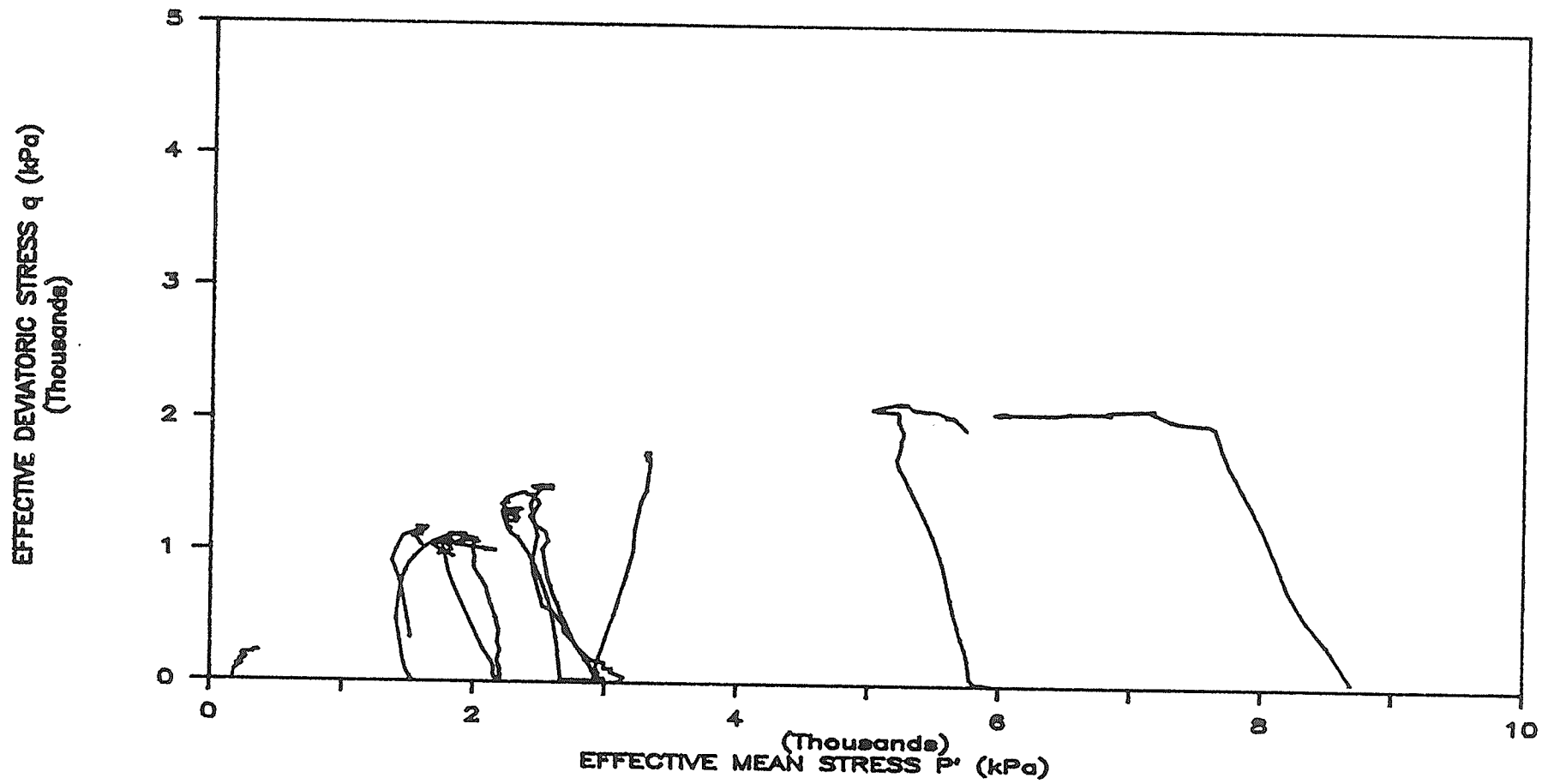


FIGURE 5.32 - EFFECTIVE STRESS PATHS FROM UNDRAINED SHEAR TESTS - HIGH DENSITY SPECIMENS

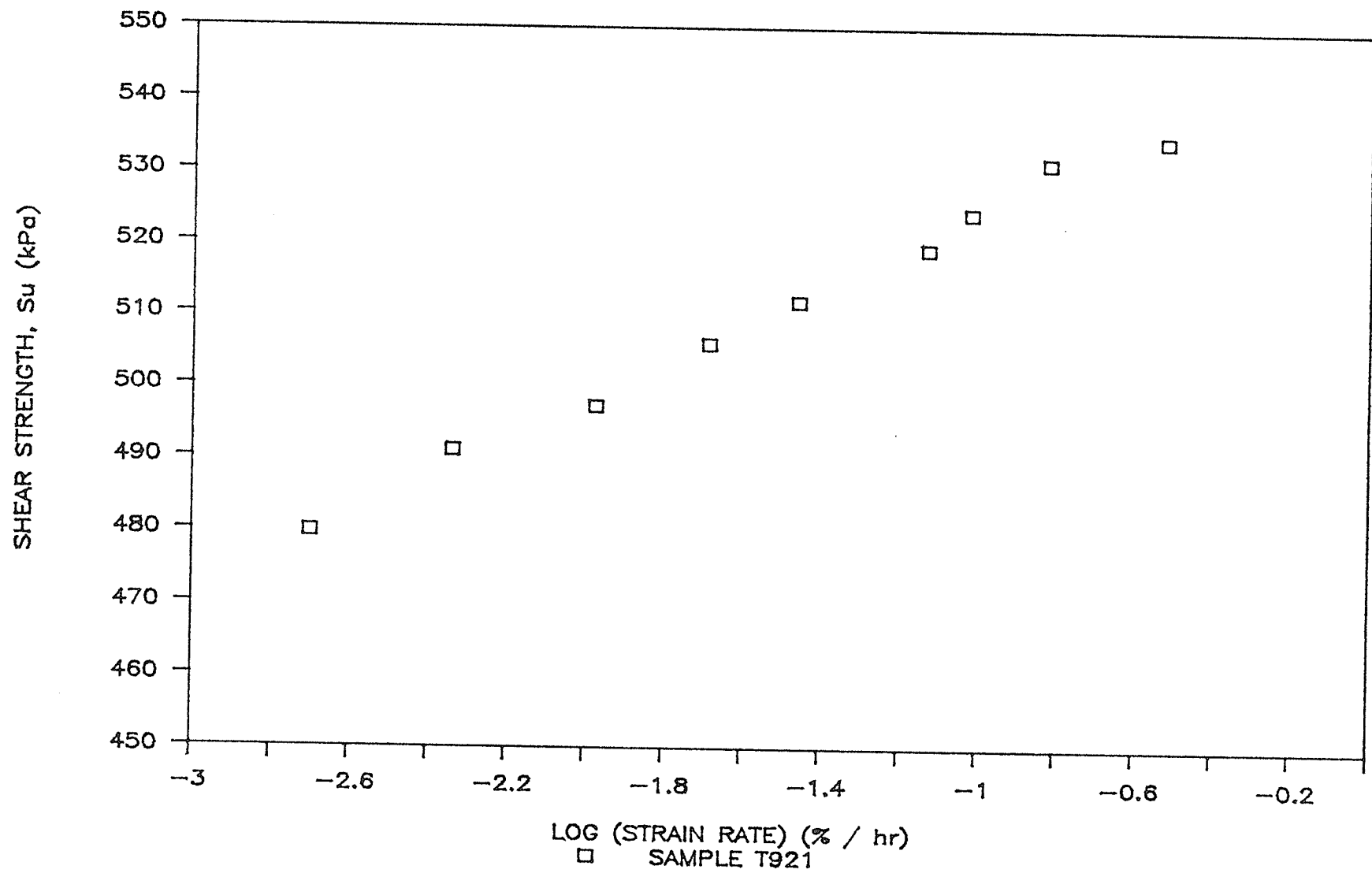


FIGURE 5.33 - INFLUENCE OF STRAIN RATE ON UNDRAINED SHEAR STRENGTH - T921

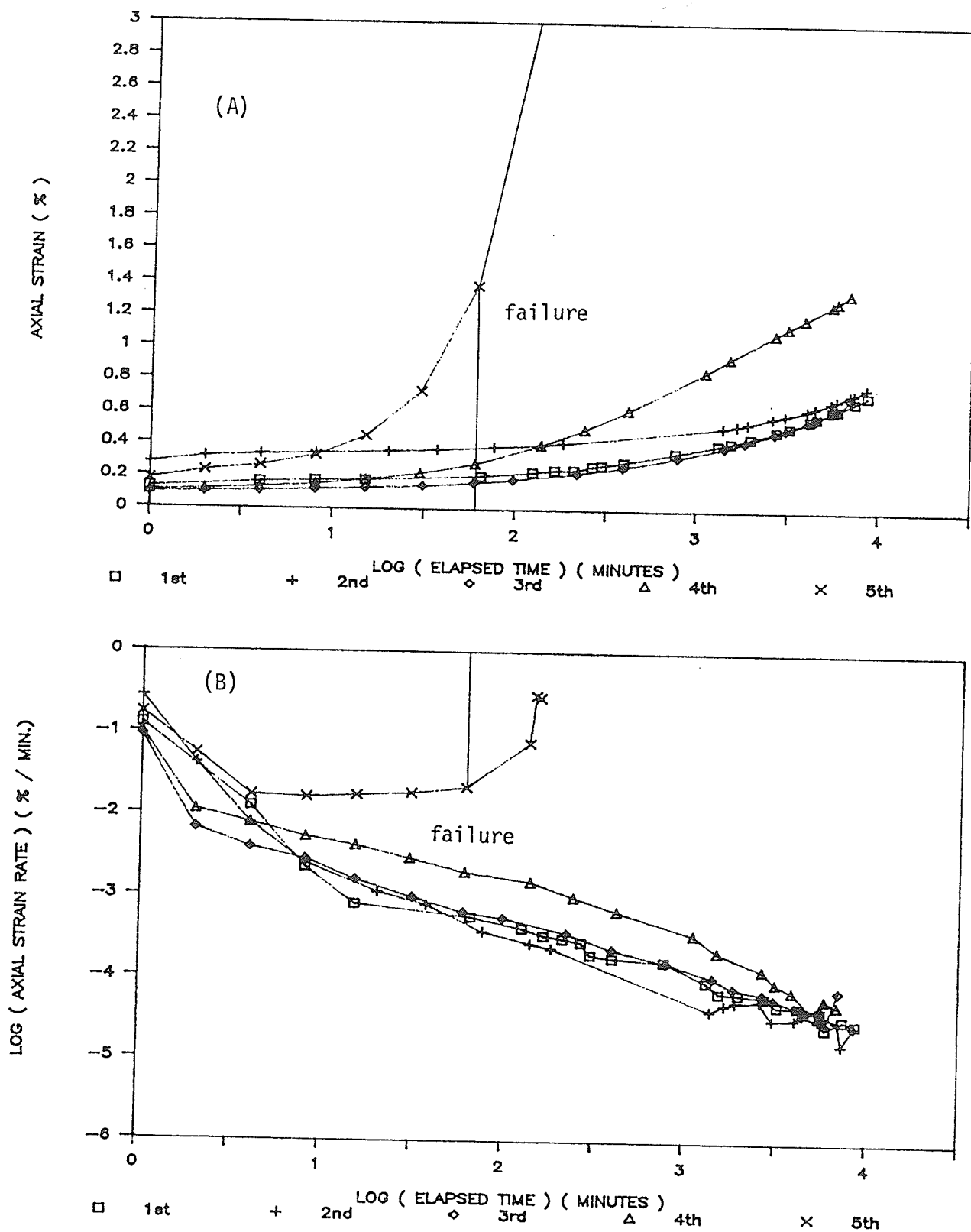
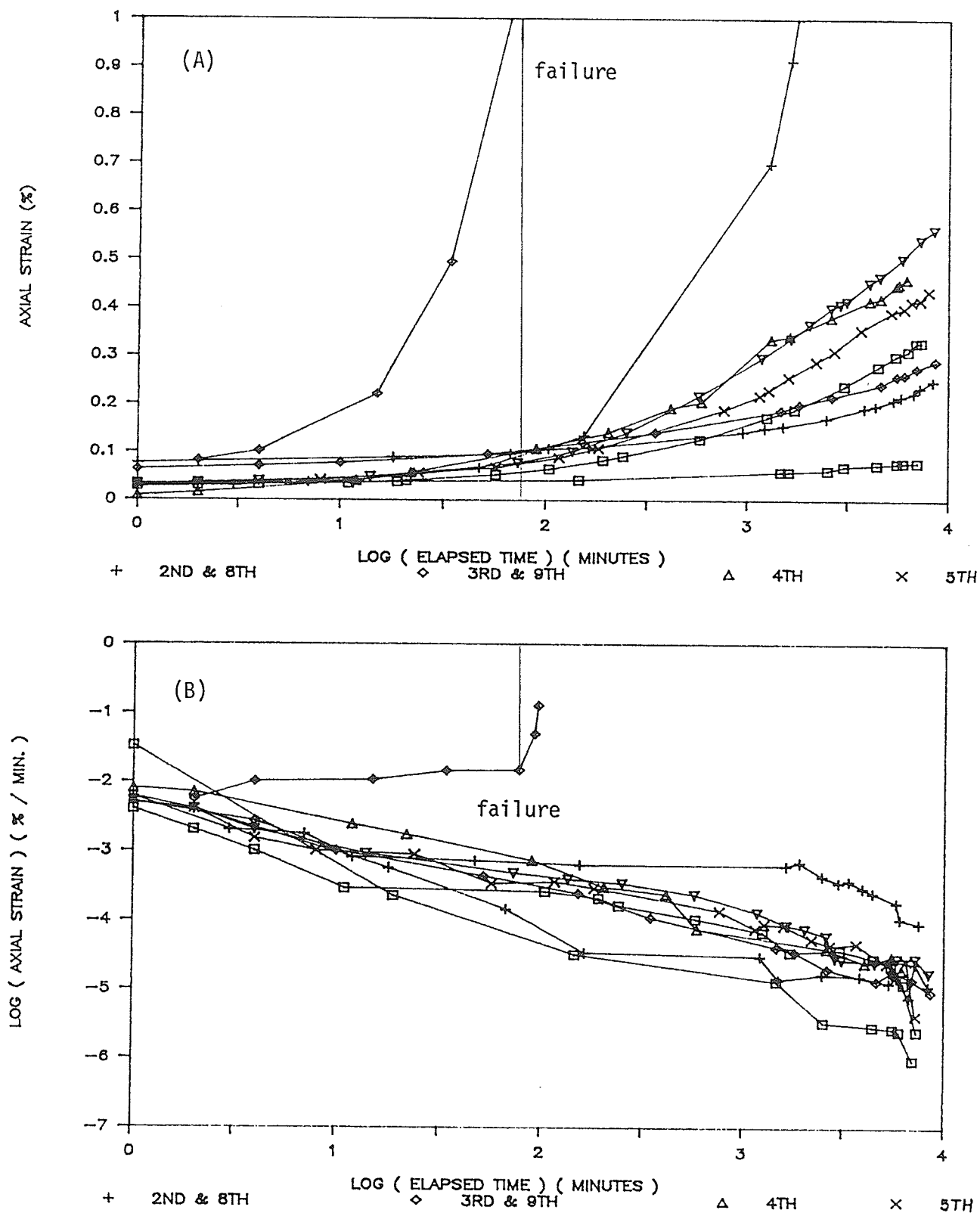


FIGURE 5.34 - INCREMENTAL DRAINED SHEAR CONSTANT- P' TEST- T910

(A) AXIAL STRAIN VERSUS LOG (ELAPSED TIME)

(B) LOG (AXIAL STRAIN RATE) VERSUS LOG (ELAPSED TIME)

FIGURE 5.35 - INCREMENTAL DRAINED SHEAR CONSTANT- p' TEST - T931

(A) AXIAL STRAIN VERSUS LOG (ELAPSED TIME)

(B) LOG (AXIAL STRAIN RATE) VERSUS LOG (ELAPSED TIME)

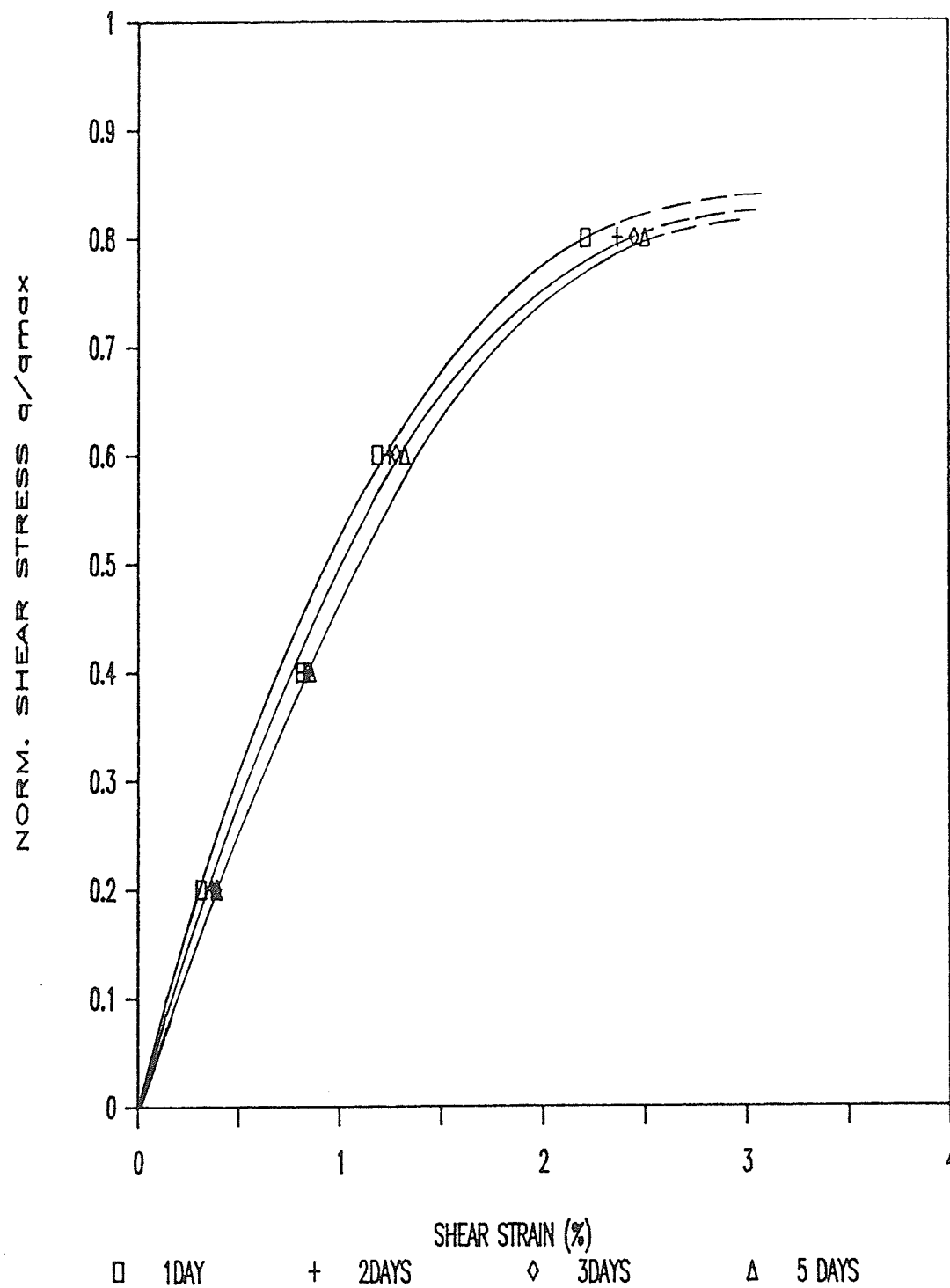


FIGURE 5.36 - NORMALIZED SHEAR STRESS q/q_{max} VERSUS SHEAR STRAIN - T910

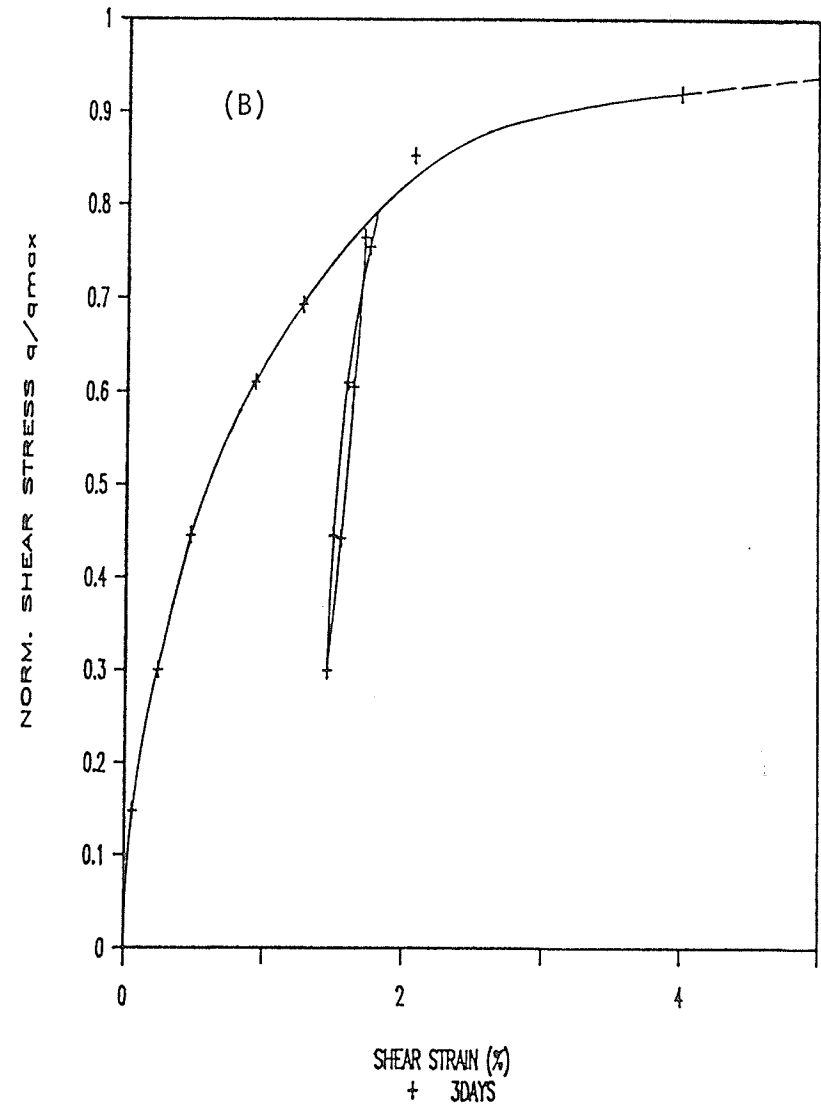
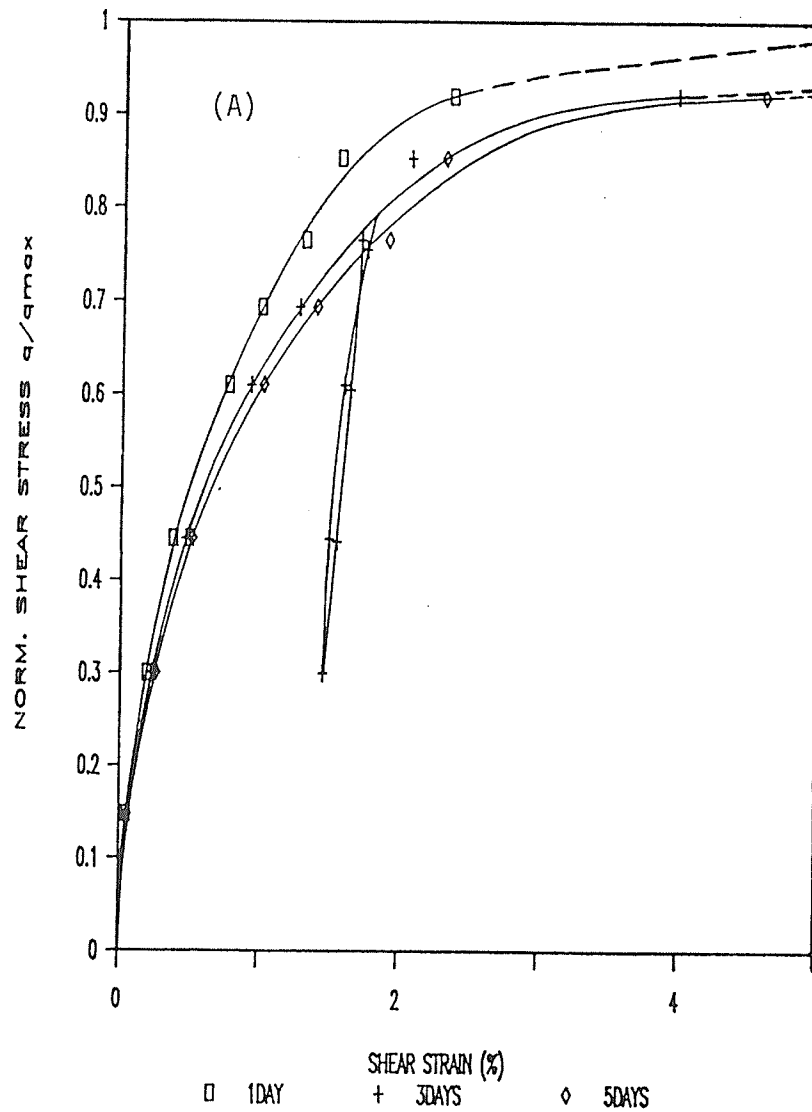


FIGURE 5.37 - NORMALIZED SHEAR STRESS q/q_{max} VERSUS SHEAR STRAIN INCLUDING UNLOAD-RELOAD CYCLE - T931

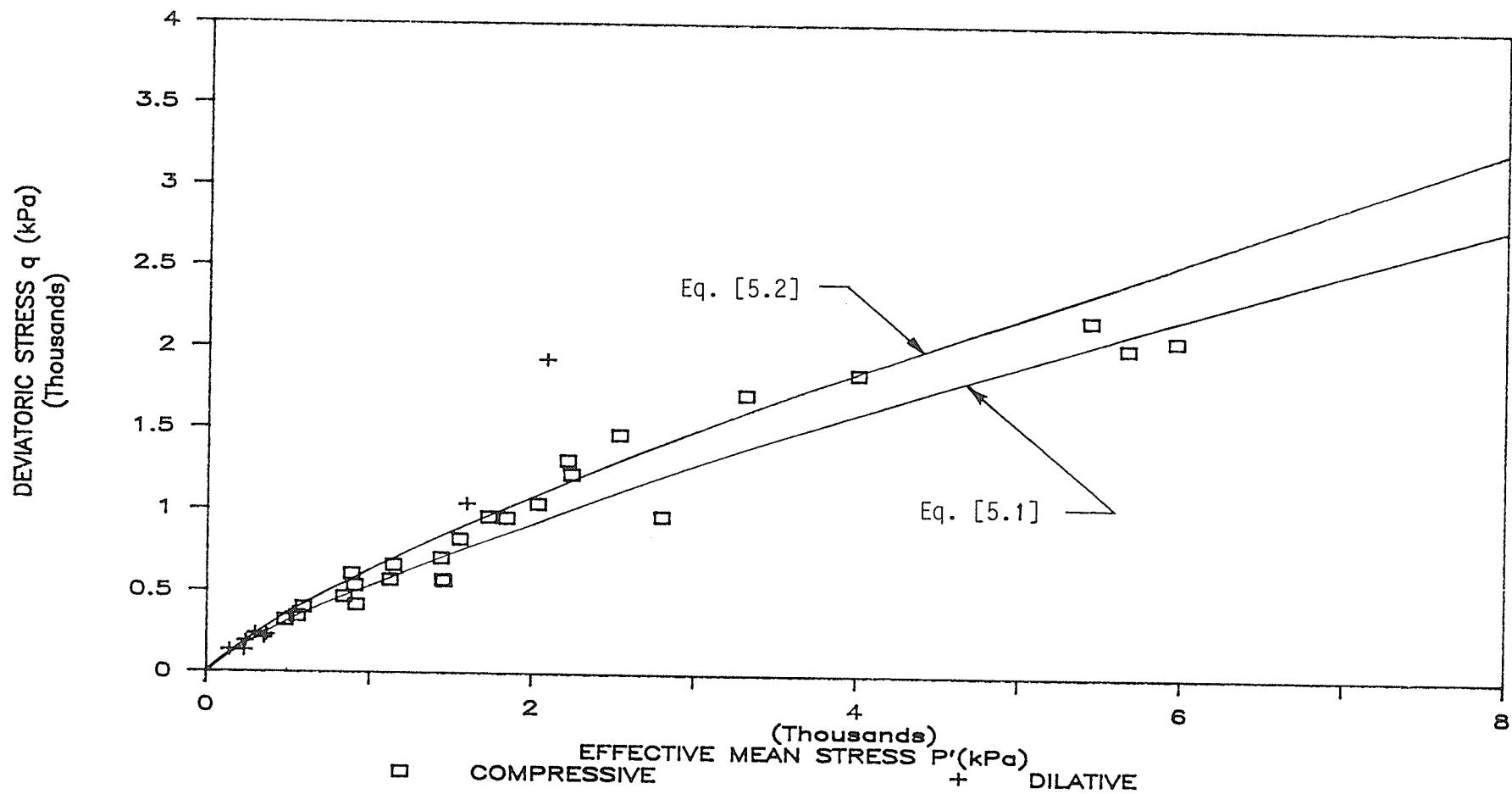


FIGURE 5.38 - SHEAR STRENGTH ENVELOPE BASED ON END-OF-TEST UNDRAINED TESTS

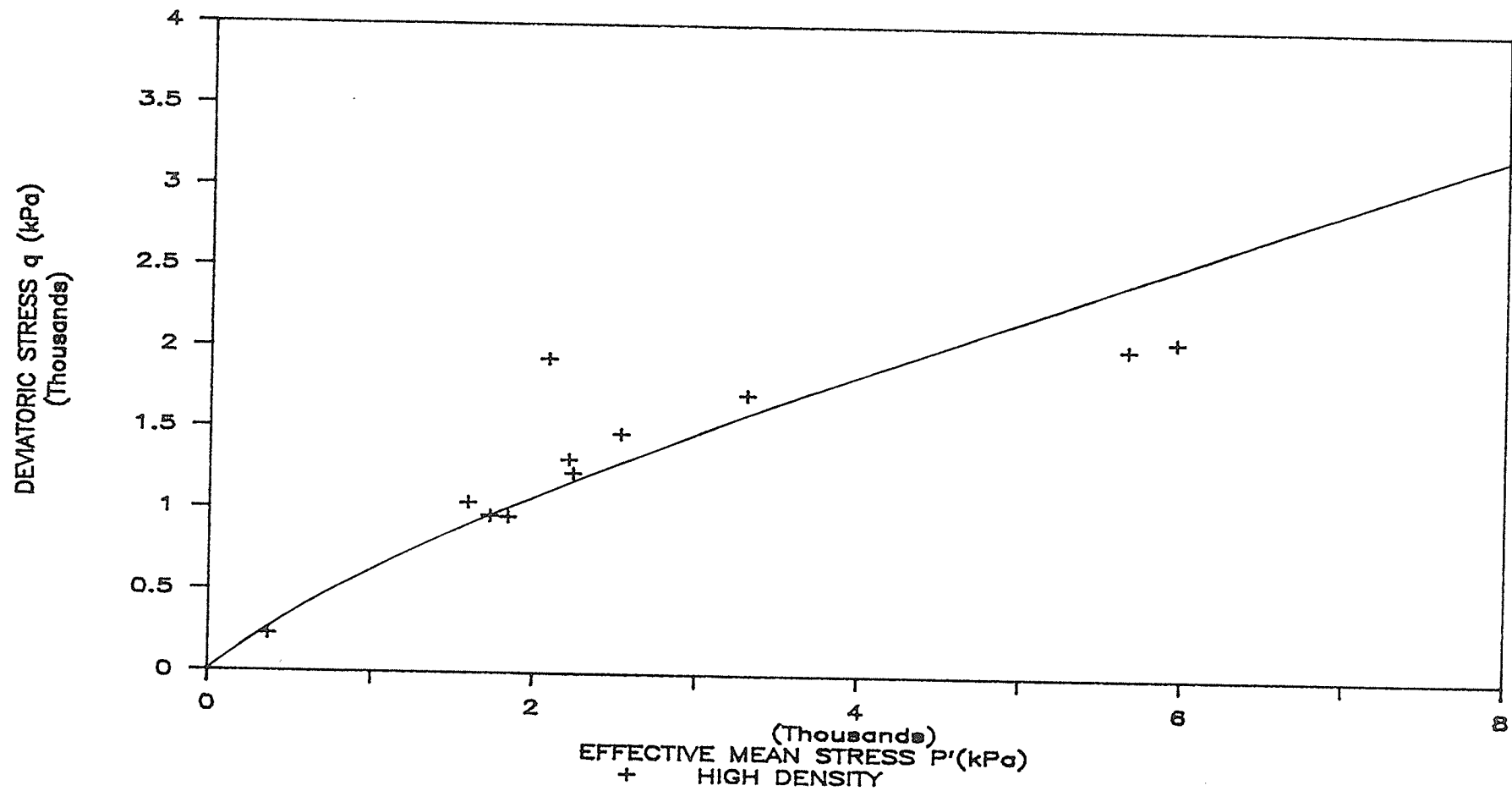


FIGURE 5.39 - SHEAR STRENGTH ENVELOPE BASED ON END-OF-TEST UNDRAINED TESTS - HIGH DENSITY SPECIMENS

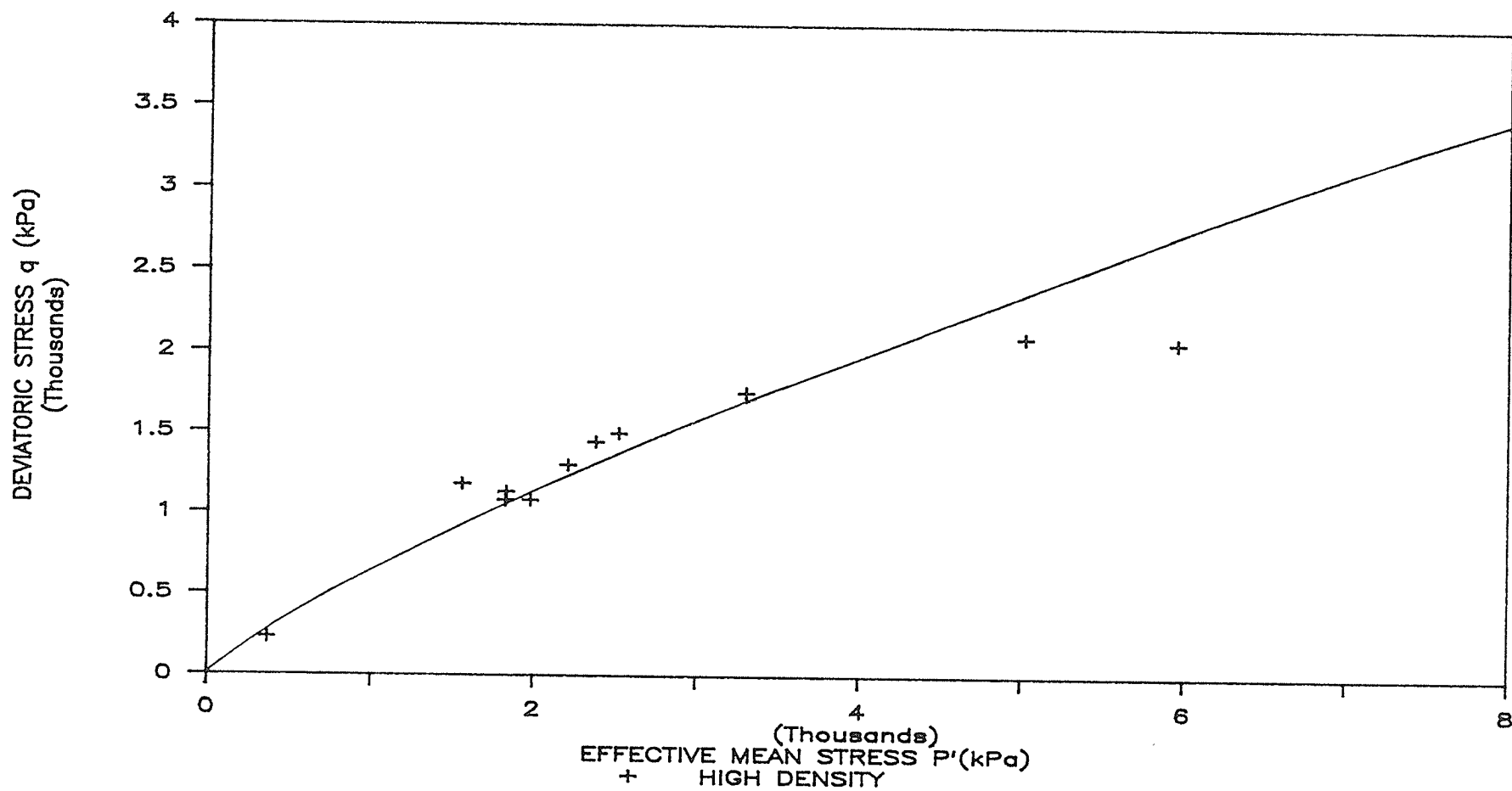


FIGURE 5.40 - SHEAR STRENGTH ENVELOPE BASED ON PEAK UNDRAINED RESULTS - HIGH DENSITY SPECIMENS

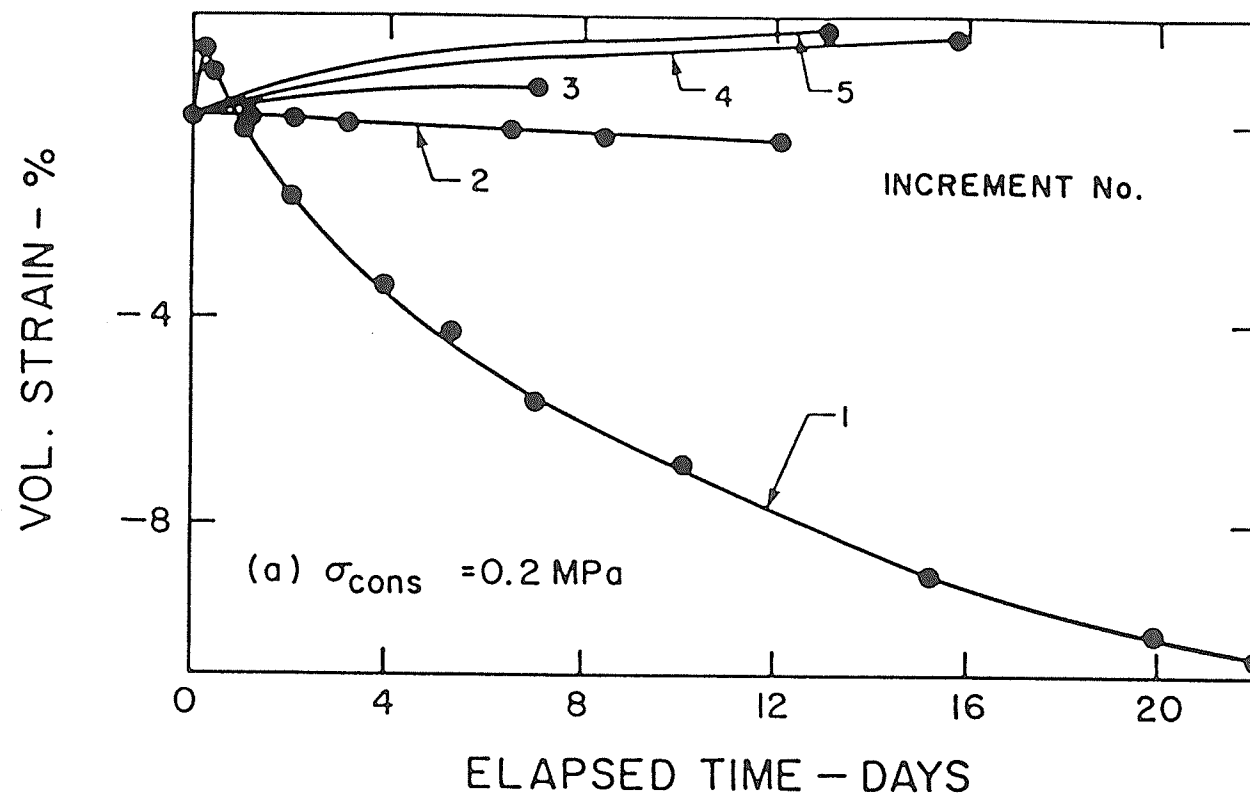


FIGURE 6.1 - VOLUMETRIC STRAIN VERSUS ELAPSED TIME - T806 (INCREMENTAL ISOTROPIC LOADING)
(AFTER SUN, 1986)

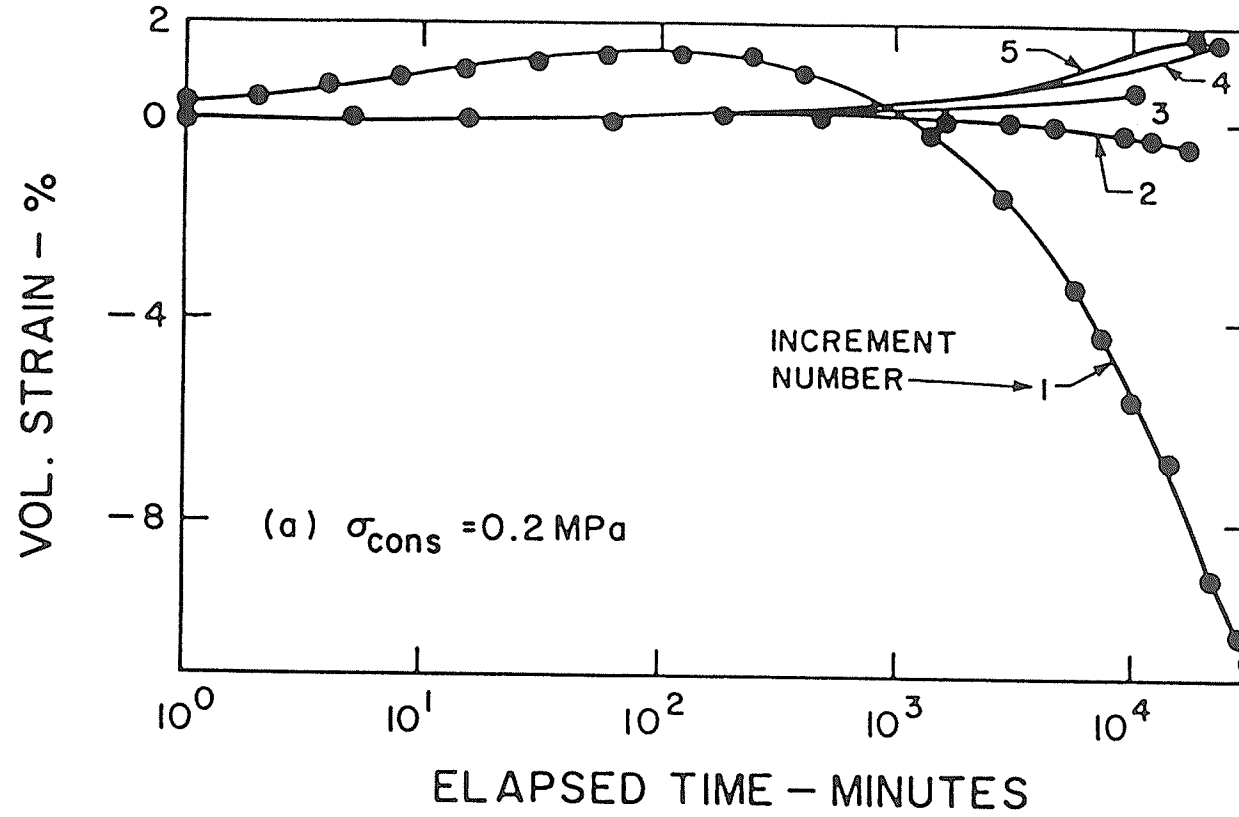


FIGURE 6.2 - VOLUMETRIC STRAIN VERSUS LOG (ELAPSED TIME) - T806 (INCREMENTAL ISOTROPIC LOADING)
(AFTER SUN, 1986)

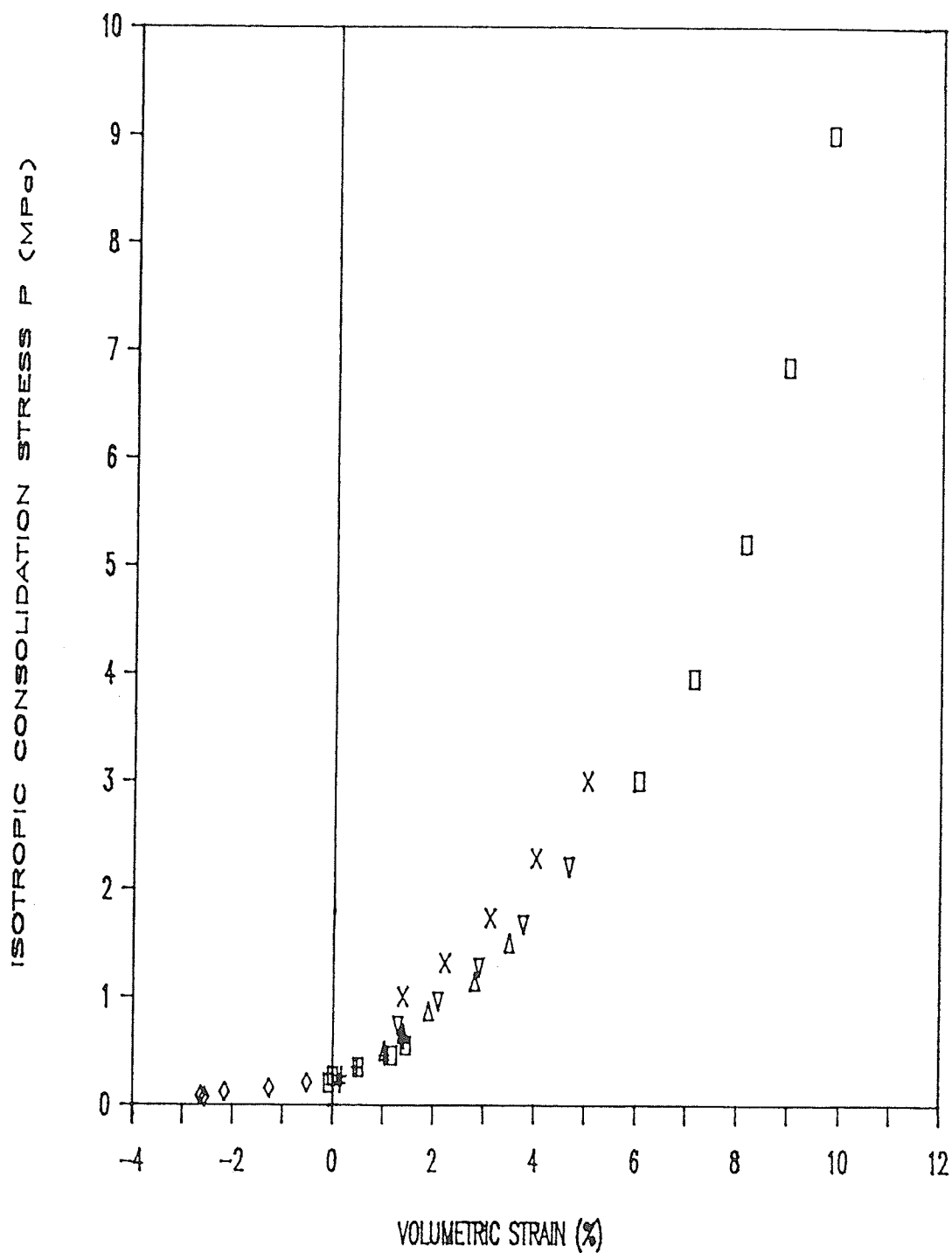


FIGURE 6.3 - RELATIONSHIP BETWEEN ISOTROPIC CONSOLIDATION STRESS P'
AND VOLUMETRIC STRAIN V - LOW DENSITY SPECIMENS

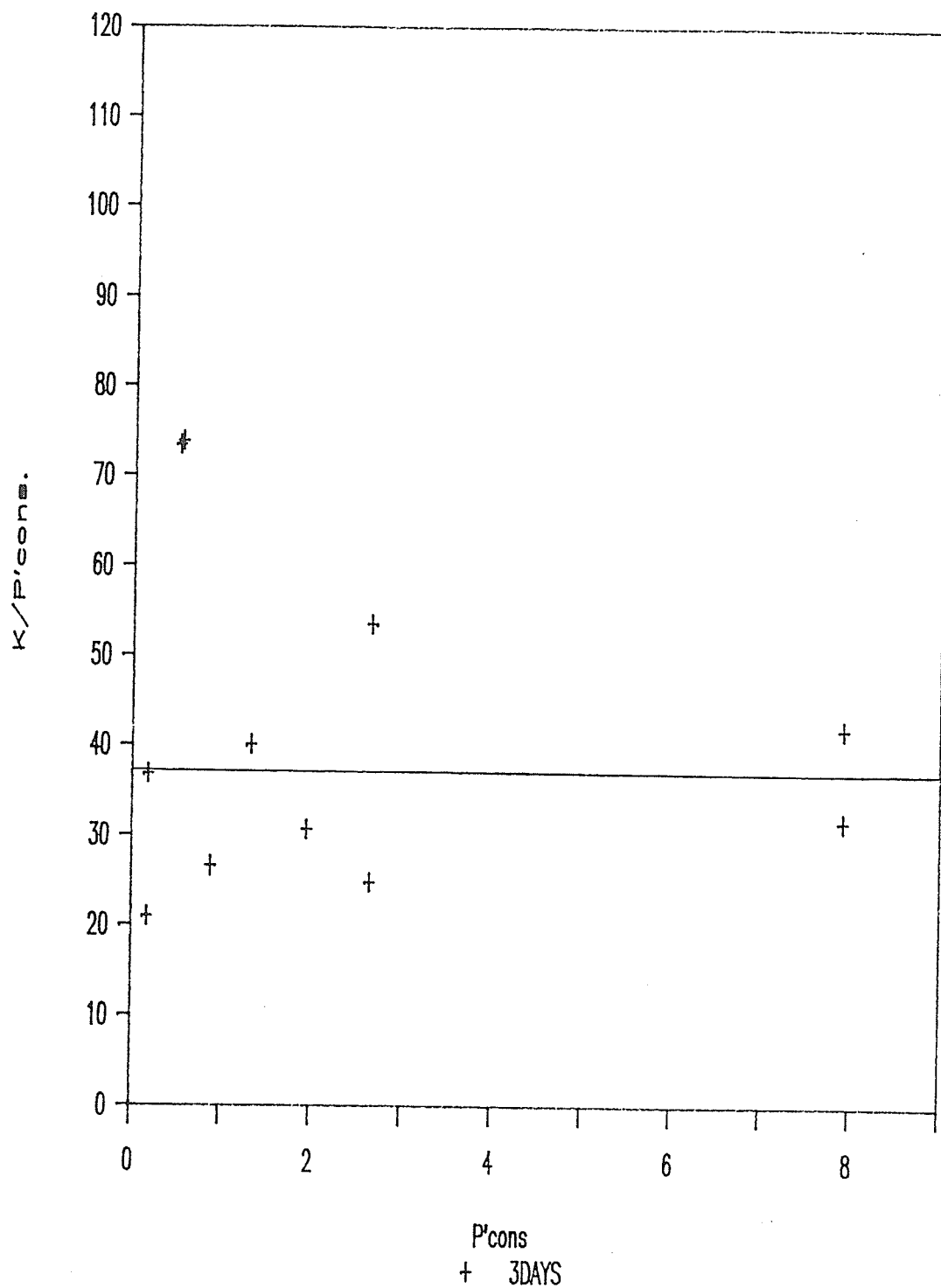


FIGURE 6.4 - RELATIONSHIP BETWEEN NORMALIZED BULK STIFFNESS AND CONSOLIDATION PRESSURE (3 DAYS DATA) - LOW DENSITY SPECIMENS

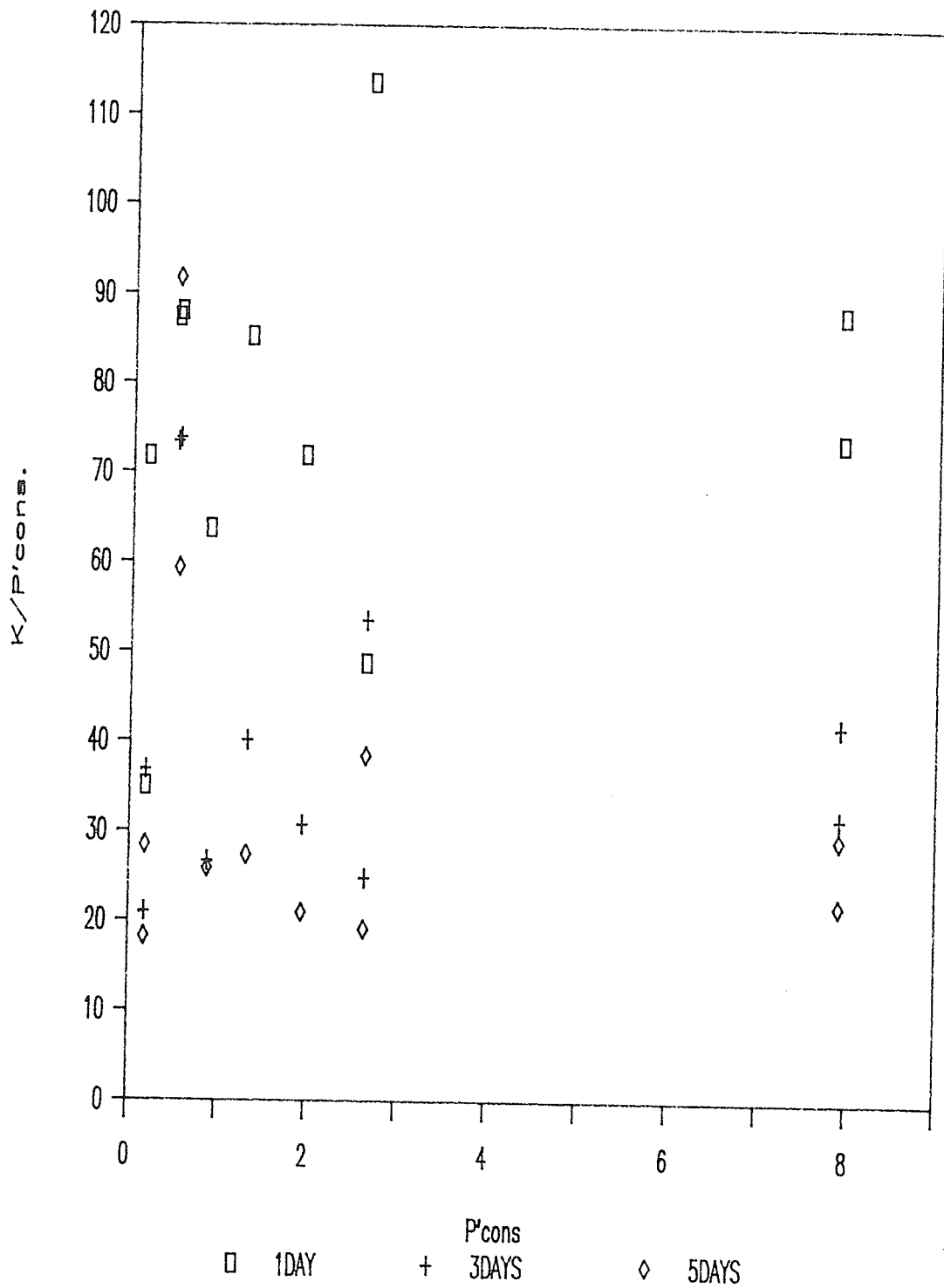


FIGURE 6.5 - RELATIONSHIP BETWEEN NORMALIZED BULK STIFFNESS AND CONSOLIDATION PRESSURE (1 DAY, 3 DAYS, 5 DAYS DATA) - LOW DENSITY SPECIMENS

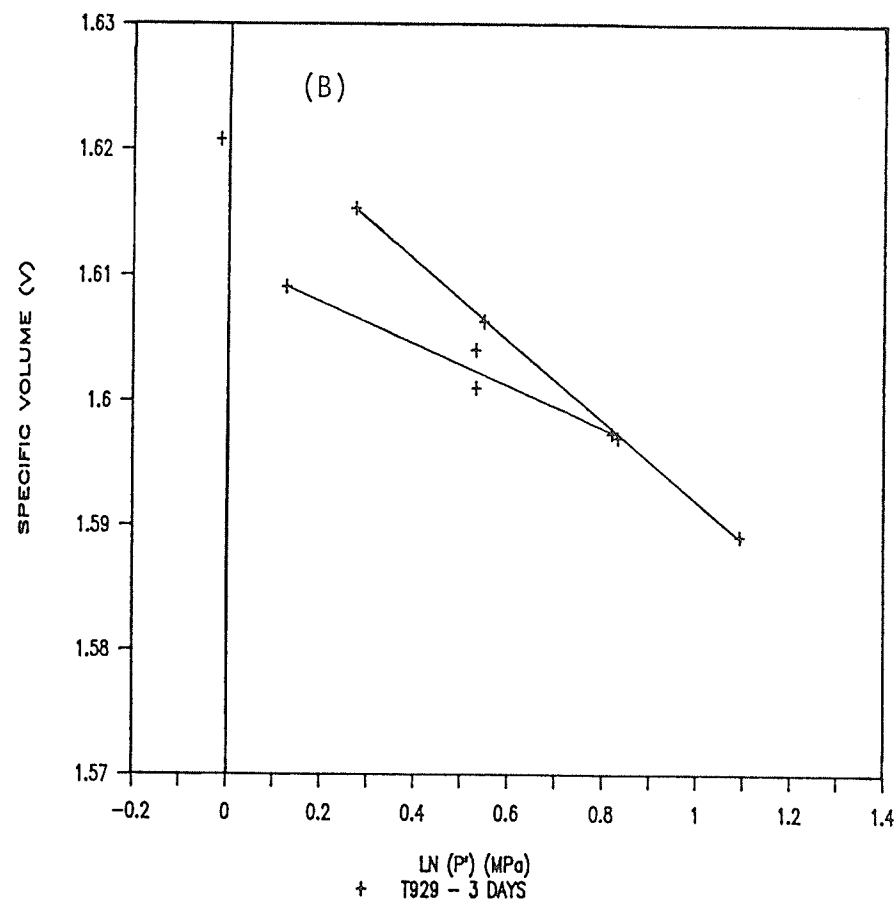
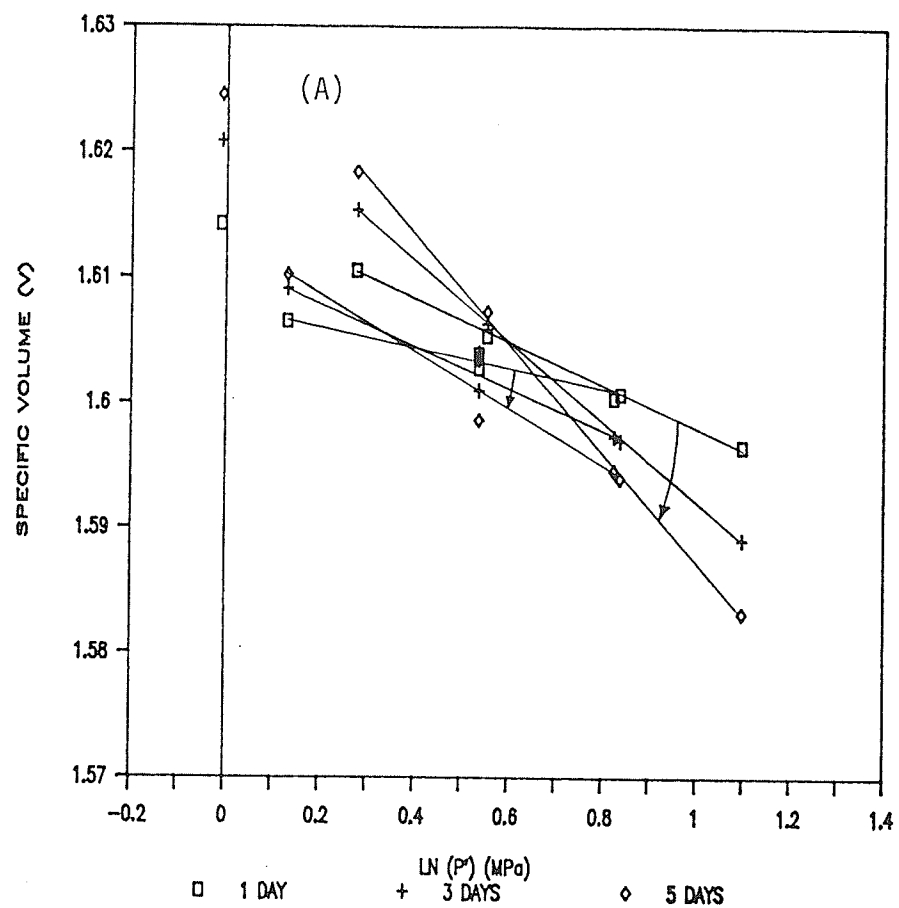


FIGURE 6.6 - SPECIFIC VOLUME V VERSUS LN (P') - T929 (INCREMENTAL ISOTROPIC LOADING-UNLOADING)

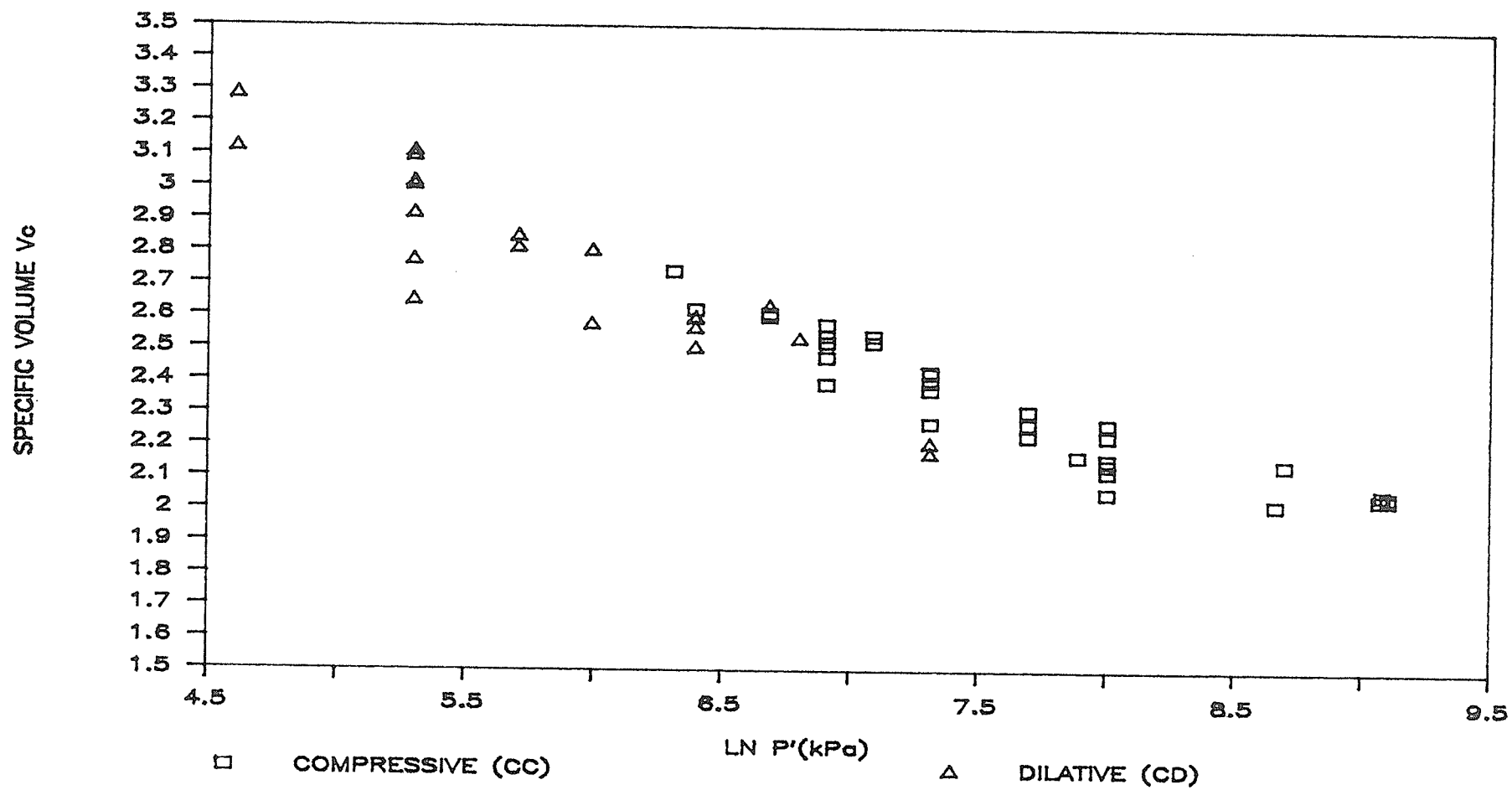


FIGURE 6.7 - SUMMARIZED END-OF-CONSOLIDATION DATA - V_c VERSUS $\ln(P')$

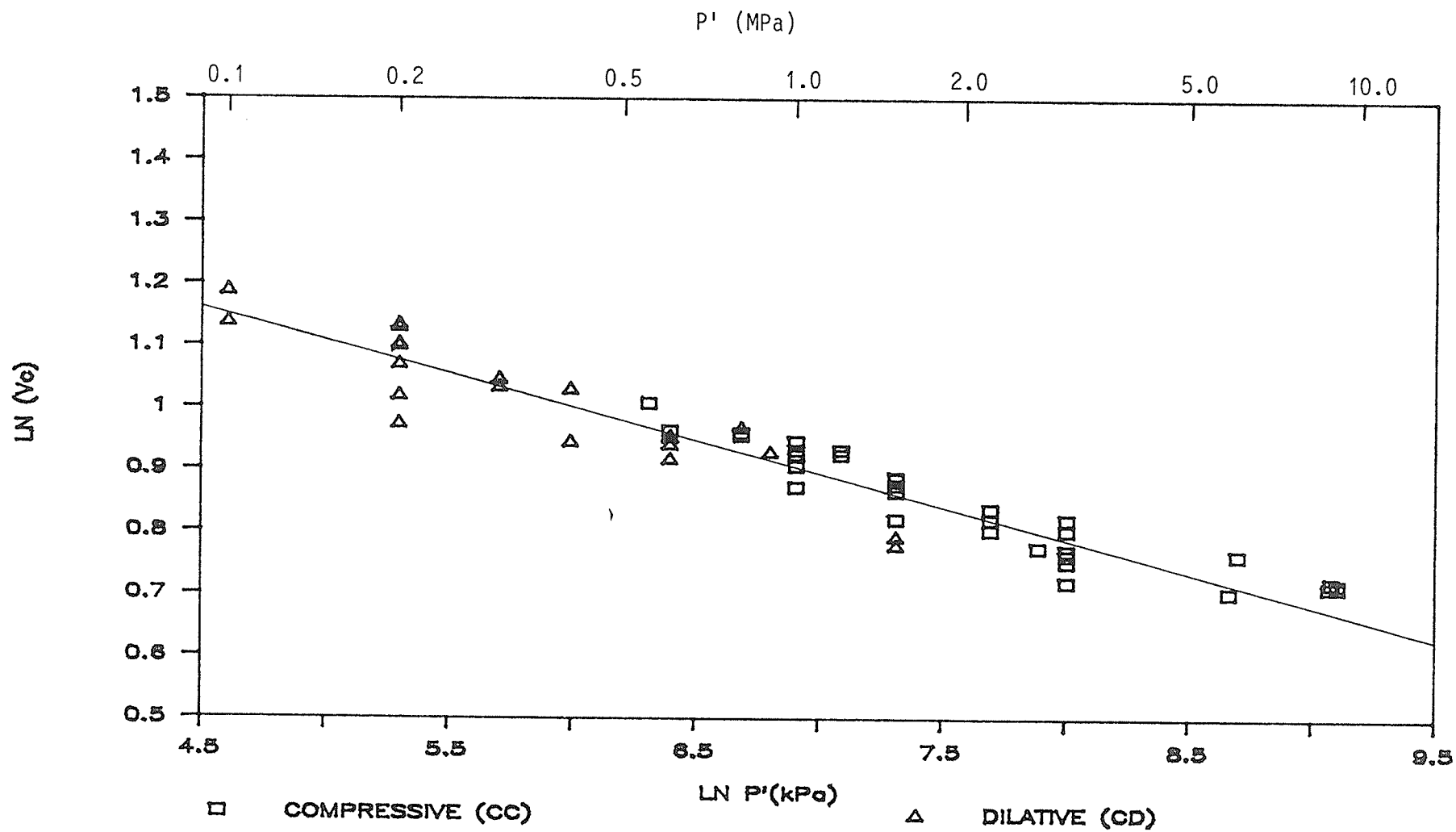


FIGURE 6.8 - SUMMARIZED END-OF-CONSOLIDATION DATA - $LN(V_c)$ VERSUS $LN(P')$

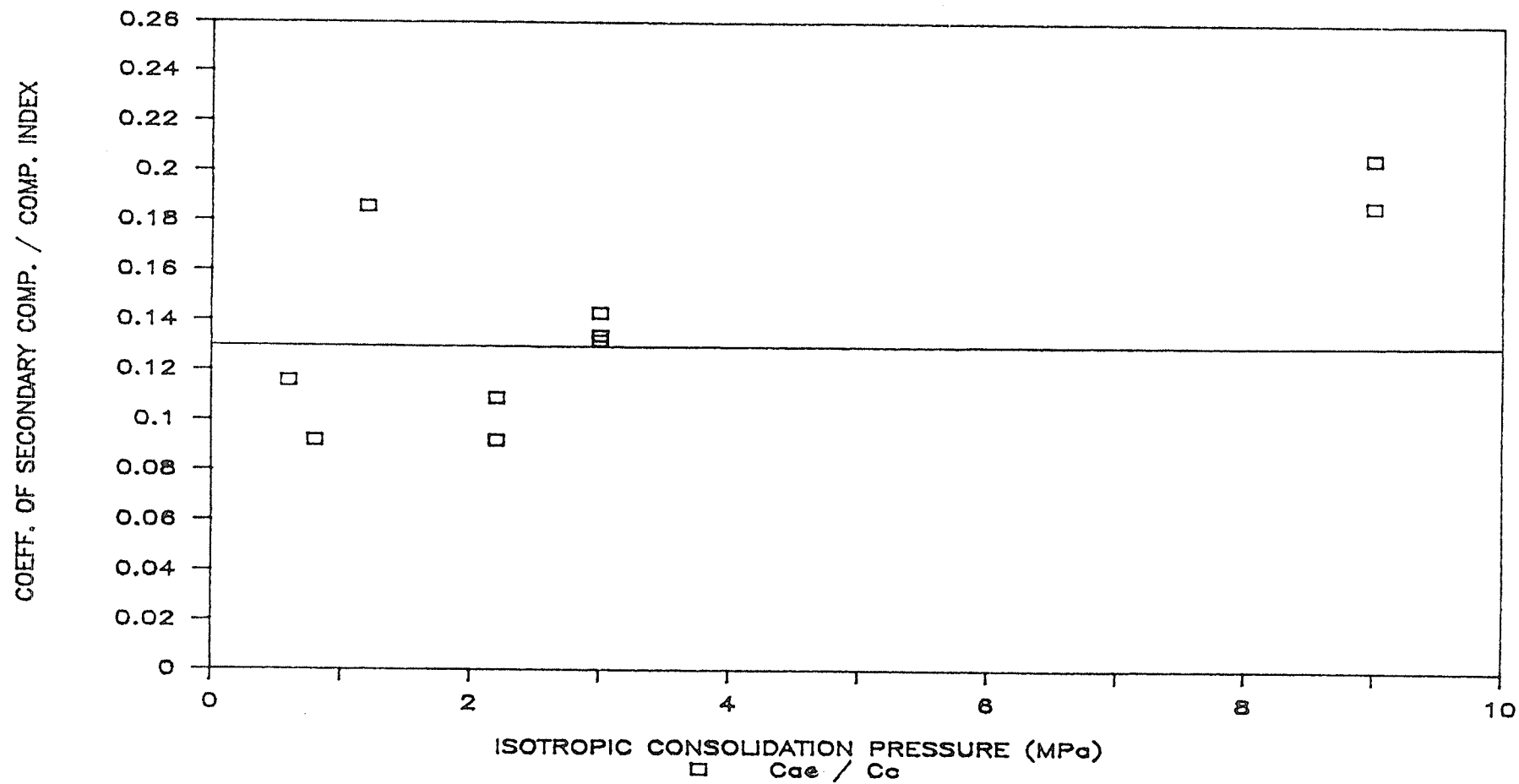


FIGURE 6.9 - RELATIONSHIP BETWEEN NORMALIZED SECONDARY CONSOLIDATION COEFFICIENT AND CONSOLIDATION PRESSURE

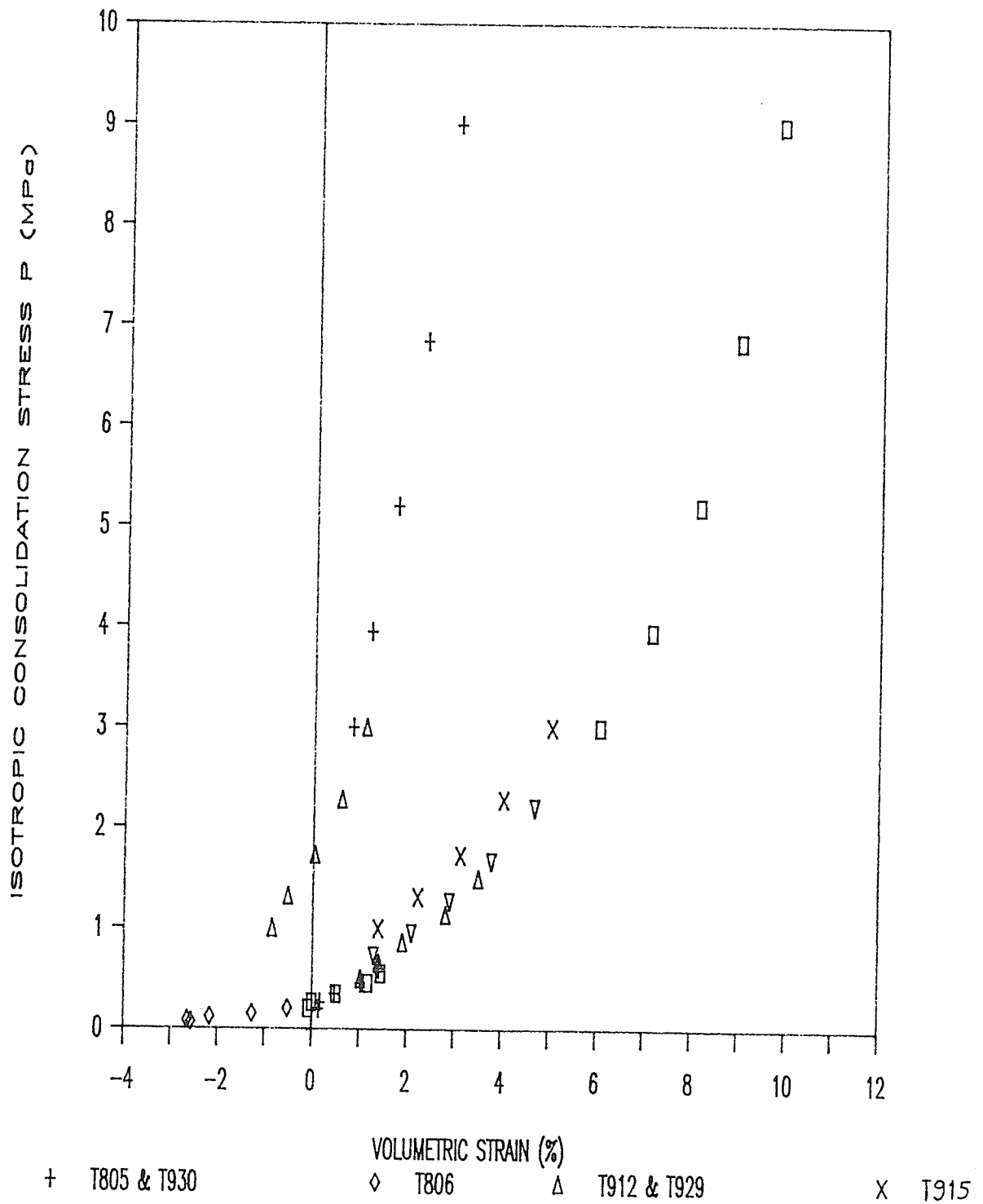


FIGURE 6.10- INFLUENCE OF INITIAL DENSITY ON THE RELATIONSHIP BETWEEN ISOTROPIC CONSOLIDATION PRESSURE P' VERSUS VOLUMETRIC STRAIN v

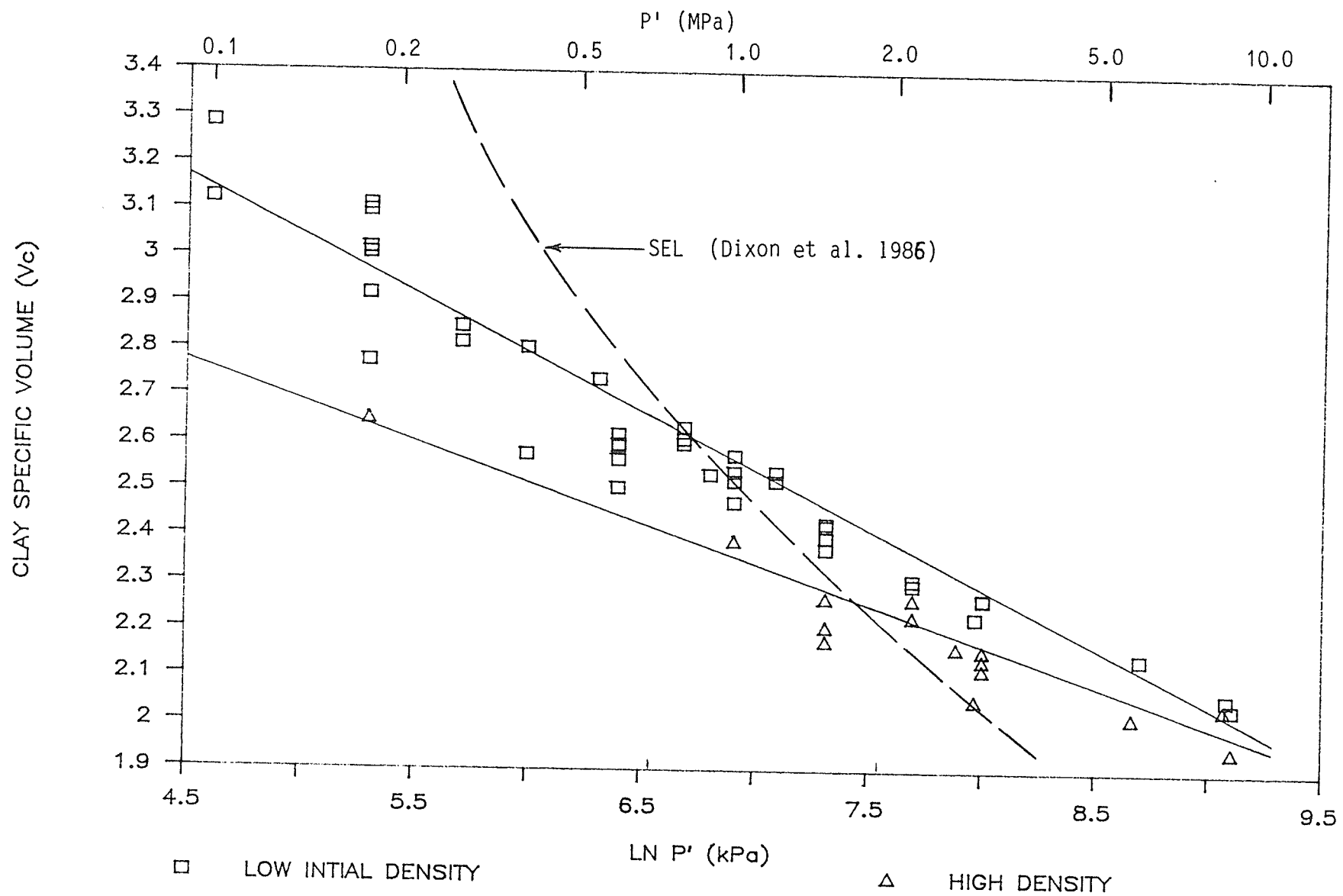


FIGURE 6.11- RELATIONSHIP BETWEEN END-OF-CONSOLIDATION V_c AND $\ln (P')$ FOR LOW DENSITY AND HIGH DENSITY BUFFER

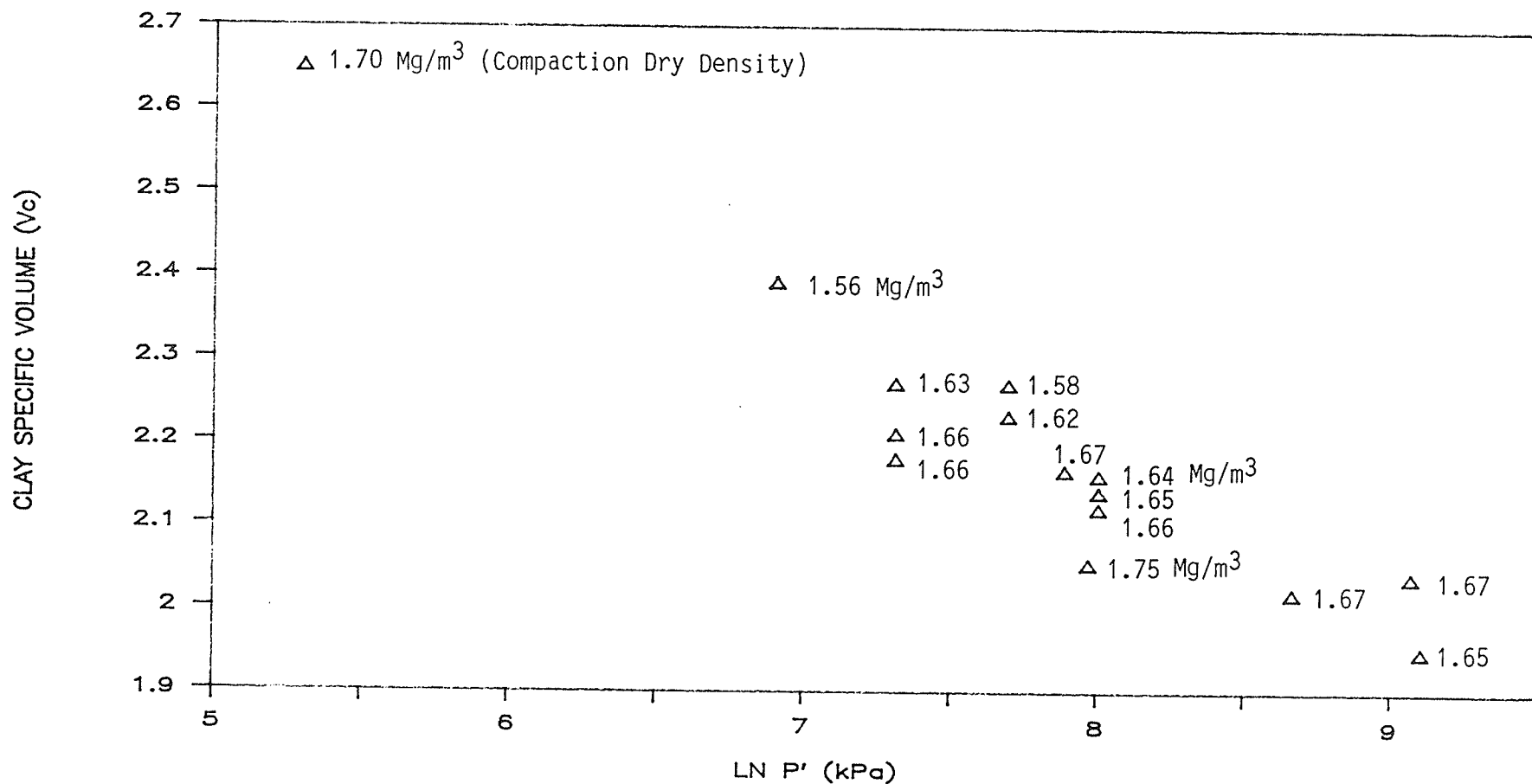


FIGURE 6.12 - RELATIONSHIP BETWEEN END-OF-CONSOLIDATION V_c AND LN (P') FOR HIGH DENSITY BUFFER

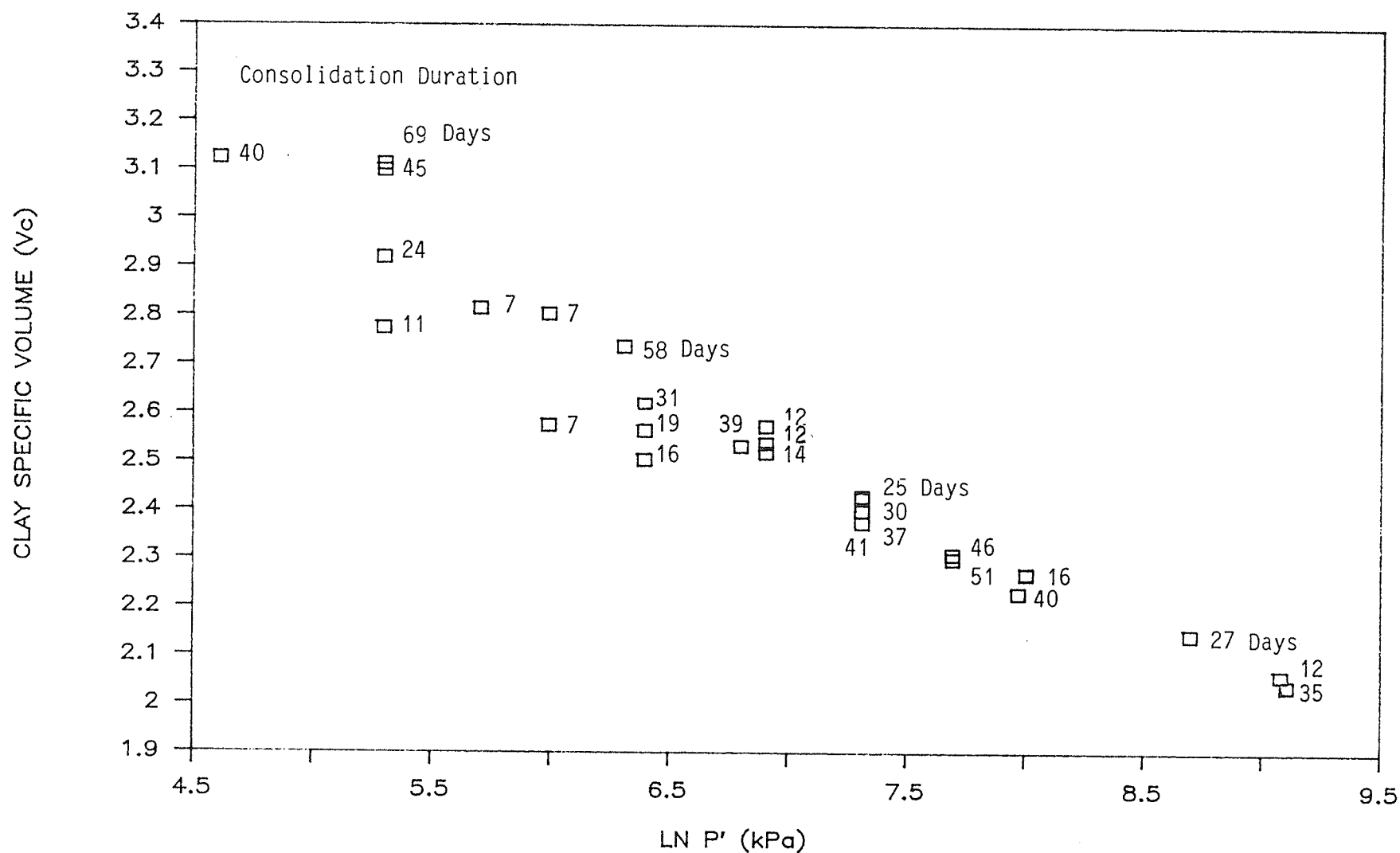


FIGURE 6.13 - EFFECT OF TIME ON END-OF-CONSOLIDATION V_c VERSUS $\ln(P')$ RELATIONSHIP FOR LOW DENSITY SPECIMENS COMPACTED AT 1.5 Mg/m^3

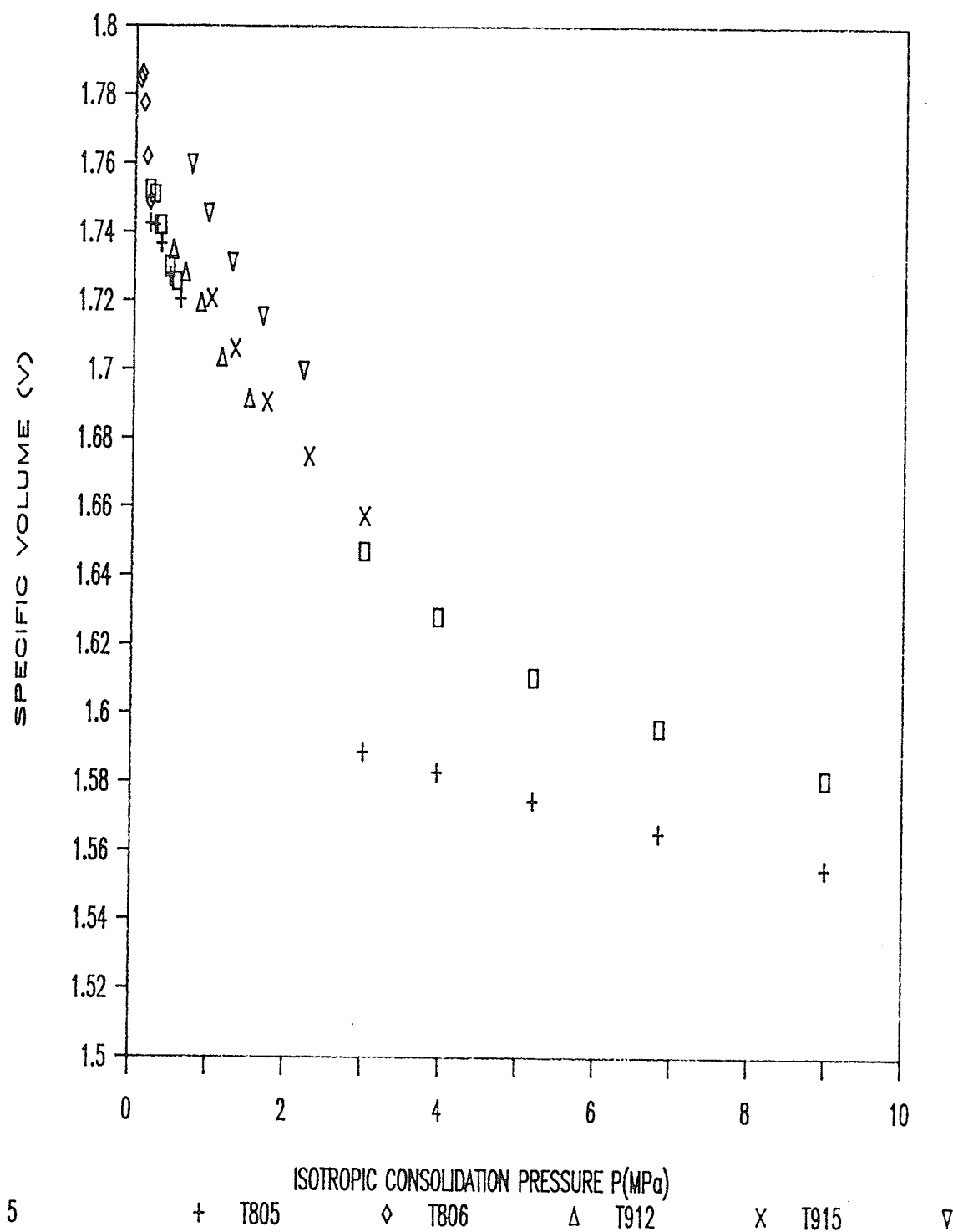


FIGURE 6.14 - RELATIONSHIP BETWEEN END-OF-CONSOLIDATION V AND LN (P')
FOR LOW DENSITY AND HIGH DENSITY BUFFER

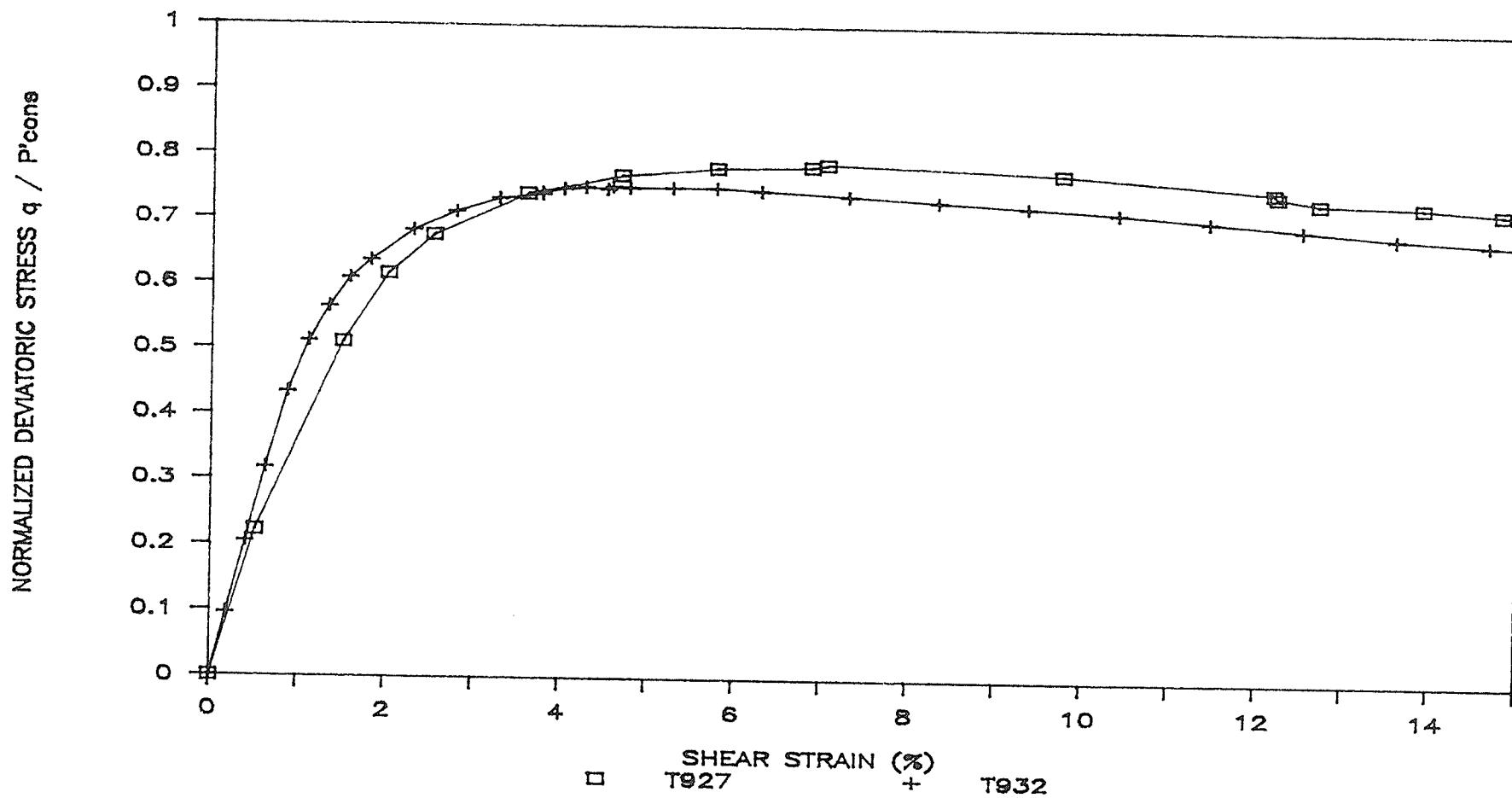


FIGURE 6.15 - NORMALIZED DEVIATORIC STRESS VERSUS SHEAR STRAIN RELATIONSHIP - HIGH DENSITY DILATIVE SPECIMENS CONSOLIDATED TO 1.5 MPa

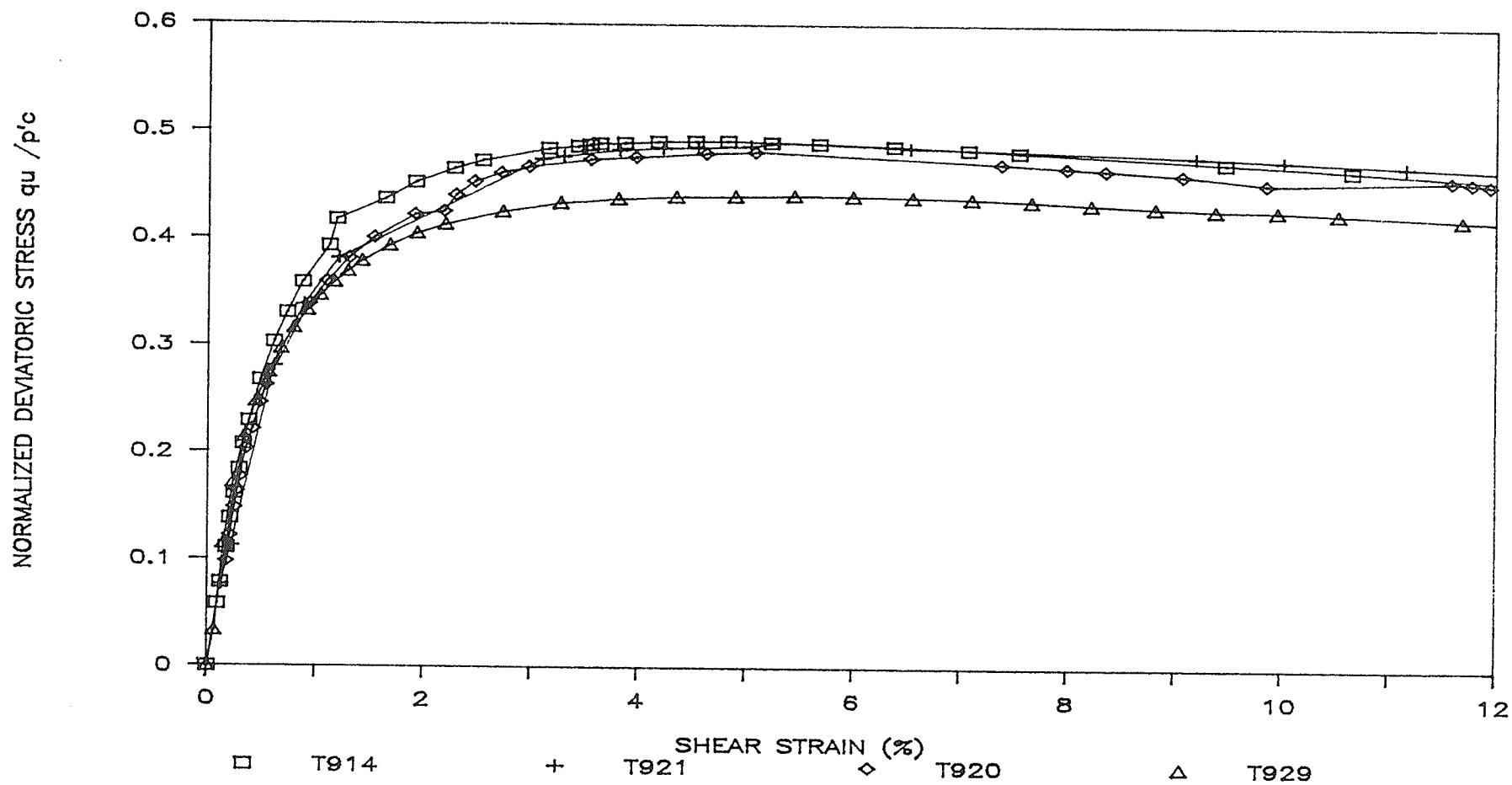


FIGURE 6.16 - NORMALIZED DEVIATORIC STRESS VERSUS SHEAR STRAIN RELATIONSHIP - HIGH DENSITY COMPRESSIVE SPECIMENS

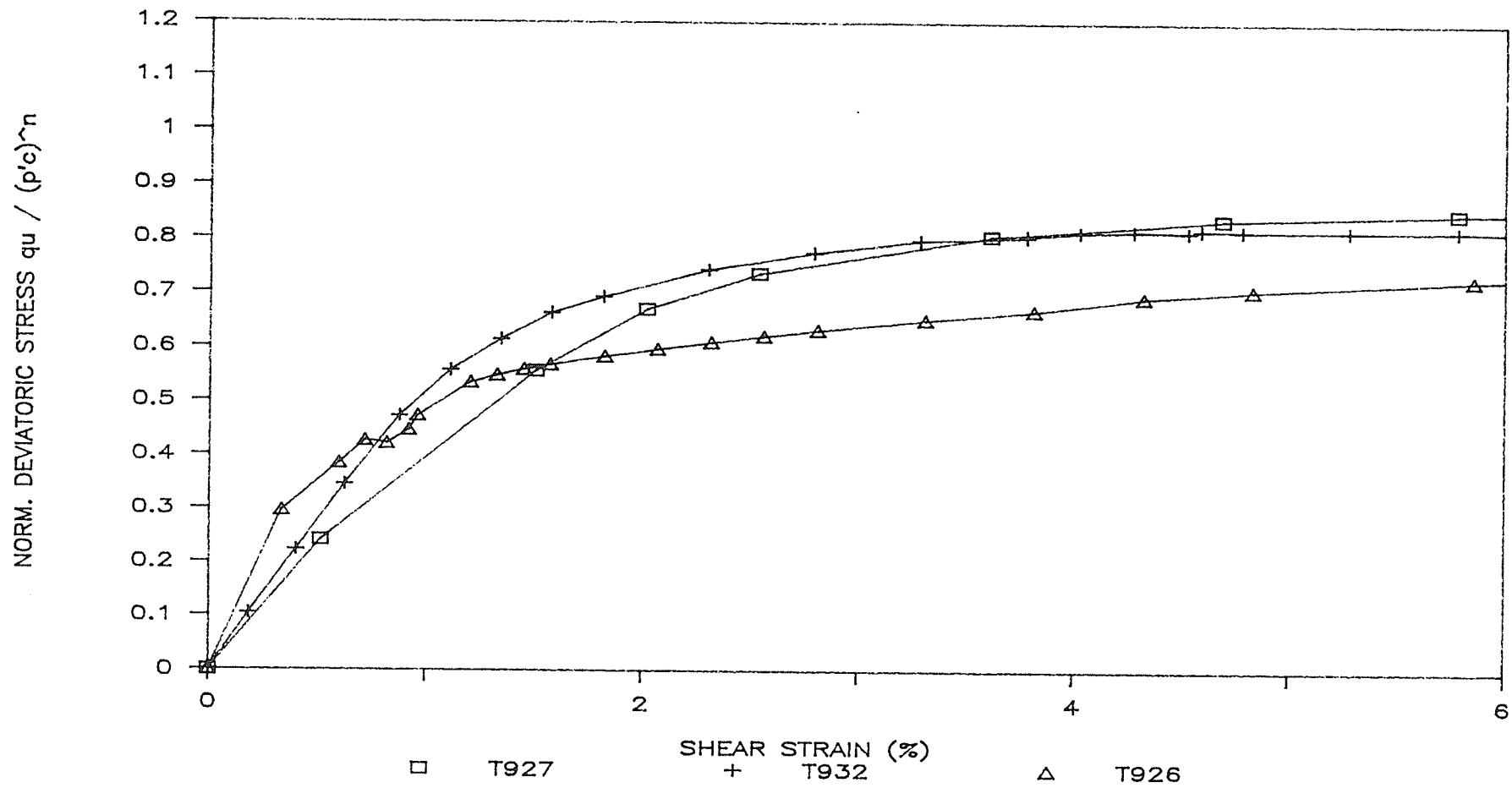


FIGURE 6.17 - NORMALIZED DEVIATORIC STRESS VERSUS SHEAR STRAIN RELATIONSHIP - HIGH DENSITY DILATIVE SPECIMENS

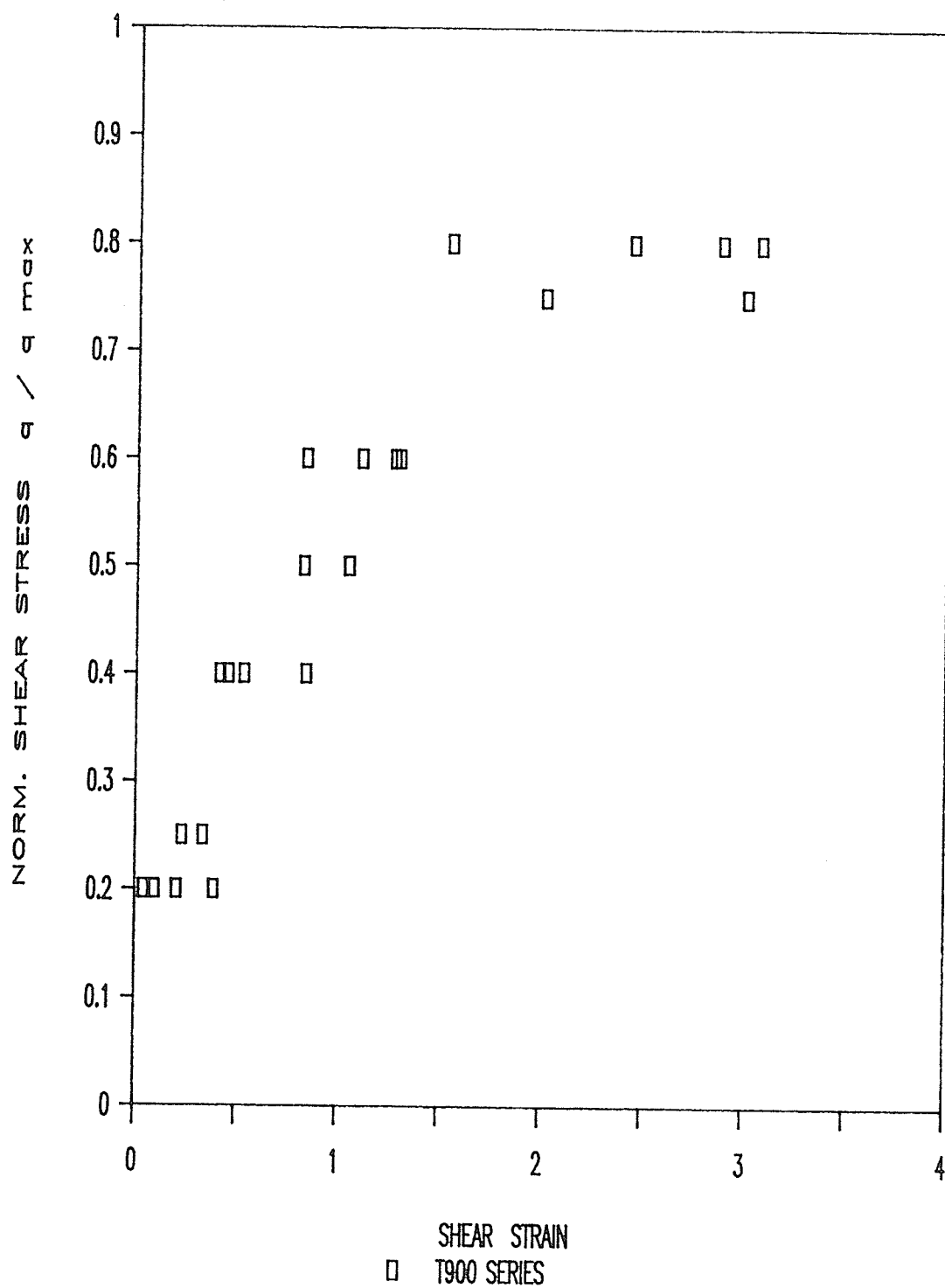


FIGURE 6.18 - NORMALIZED SHEAR STRESS q / q_{max} VERSUS SHEAR STRAIN
RELATIONSHIP FOR LOW DENSITY BUFFER

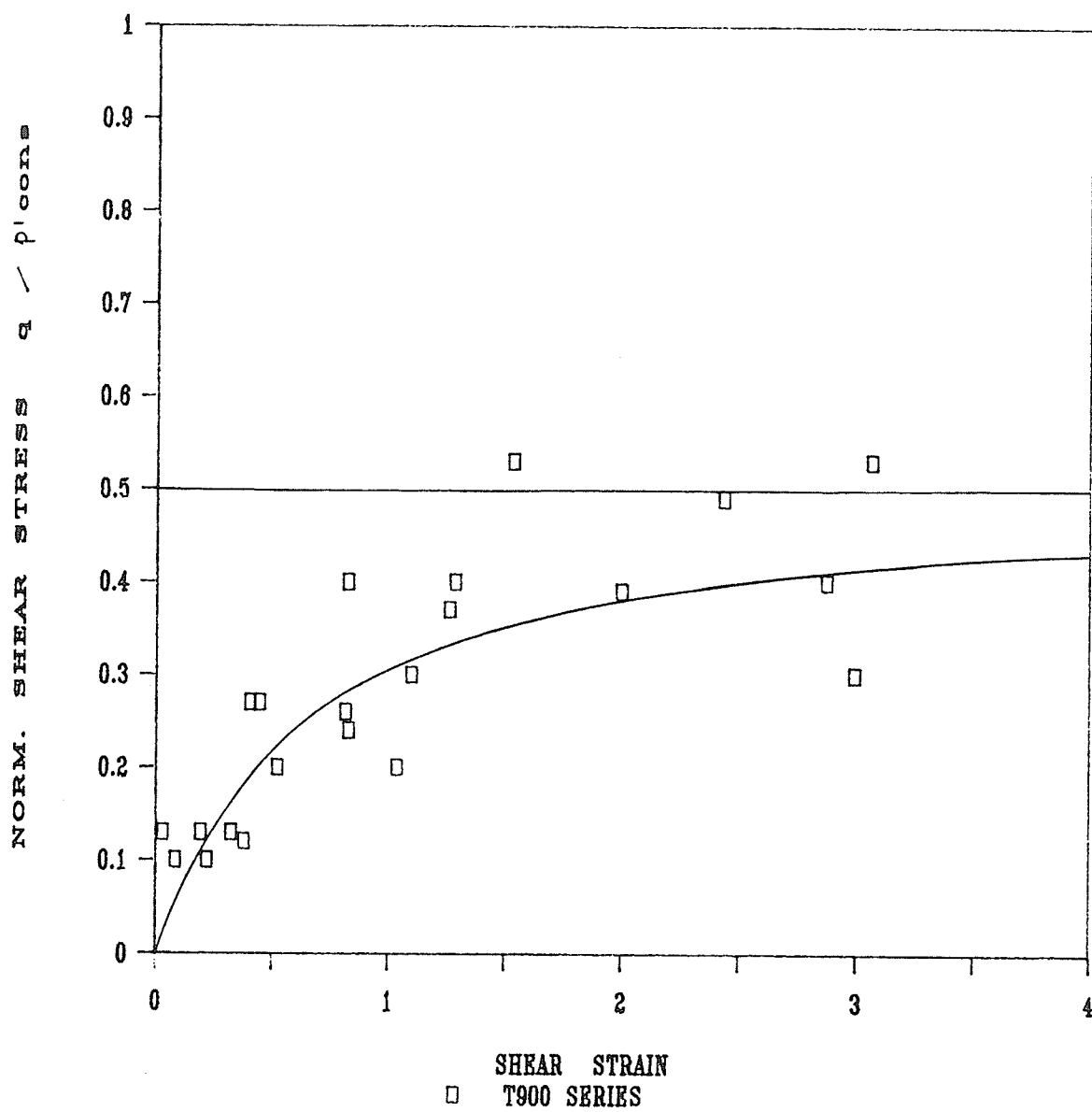


FIGURE 6.19 - NORMALIZED SHEAR STRESS q / p'_{cons} AND SHEAR STRAIN RELATIONSHIP

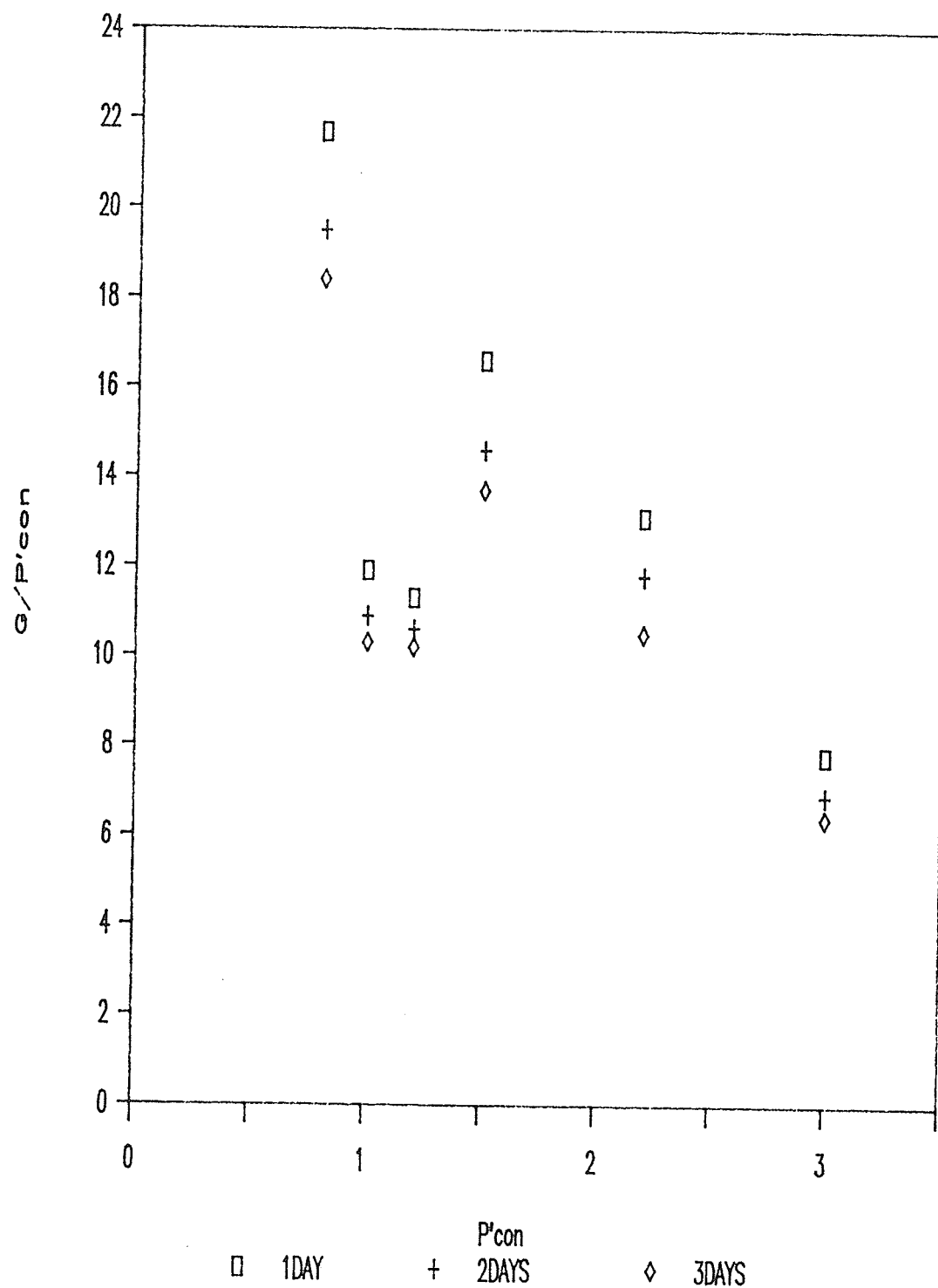


FIGURE 6.20 - RELATIONSHIP BETWEEN NORMALIZED SHEAR STIFFNESS G_{50} / P'_{cons}
(1 DAY, 2 DAYS, AND 3 DAYS DATA) AND CONSOLIDATION PRESSURE P'_{cons}

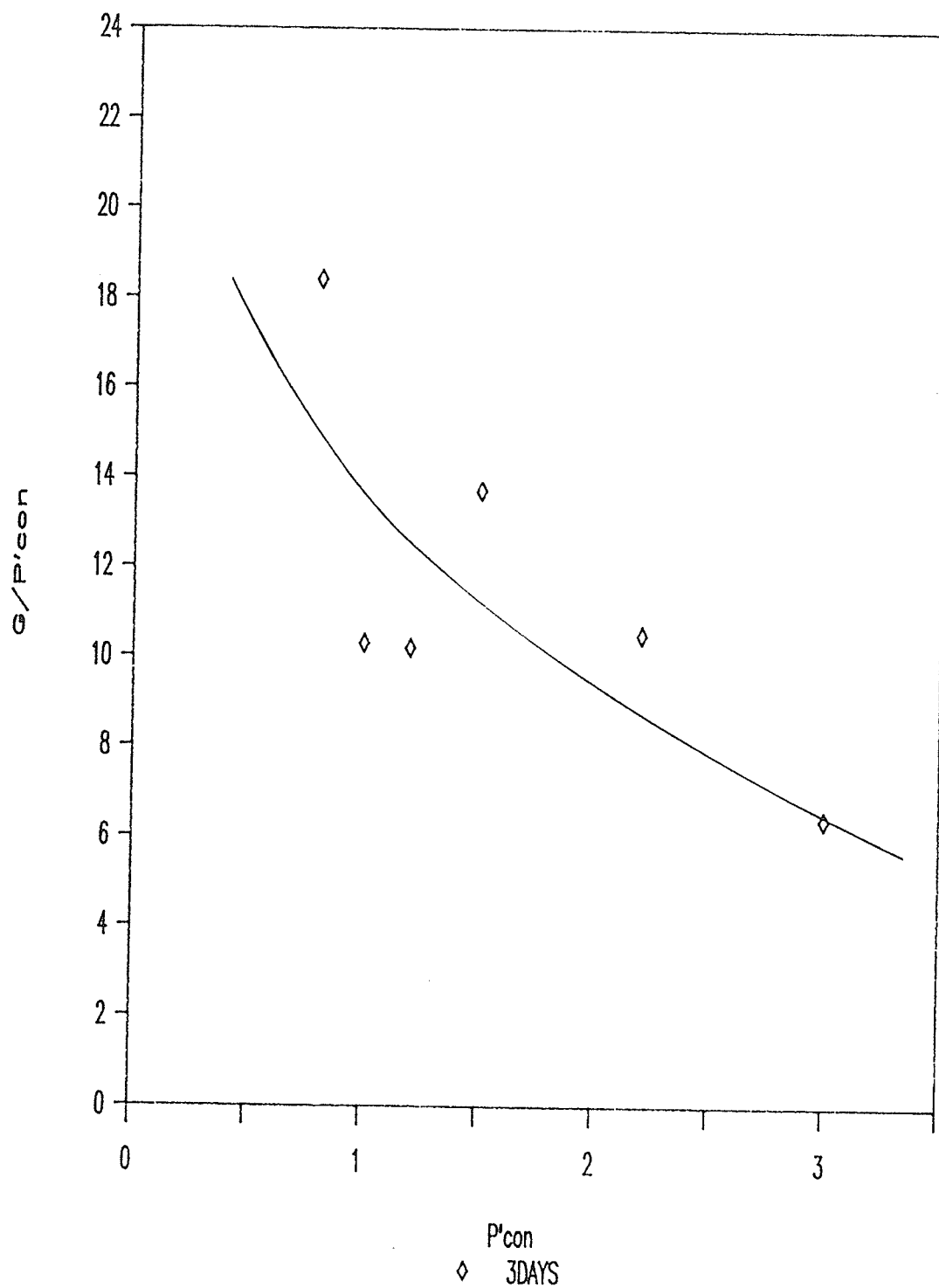


FIGURE 6.21 - RELATIONSHIP BETWEEN NORMALIZED SHEAR STIFFNESS G_{50} / P'_{cons} (3 DAYS DATA) AND CONSOLIDATION PRESSURE P'_{cons}

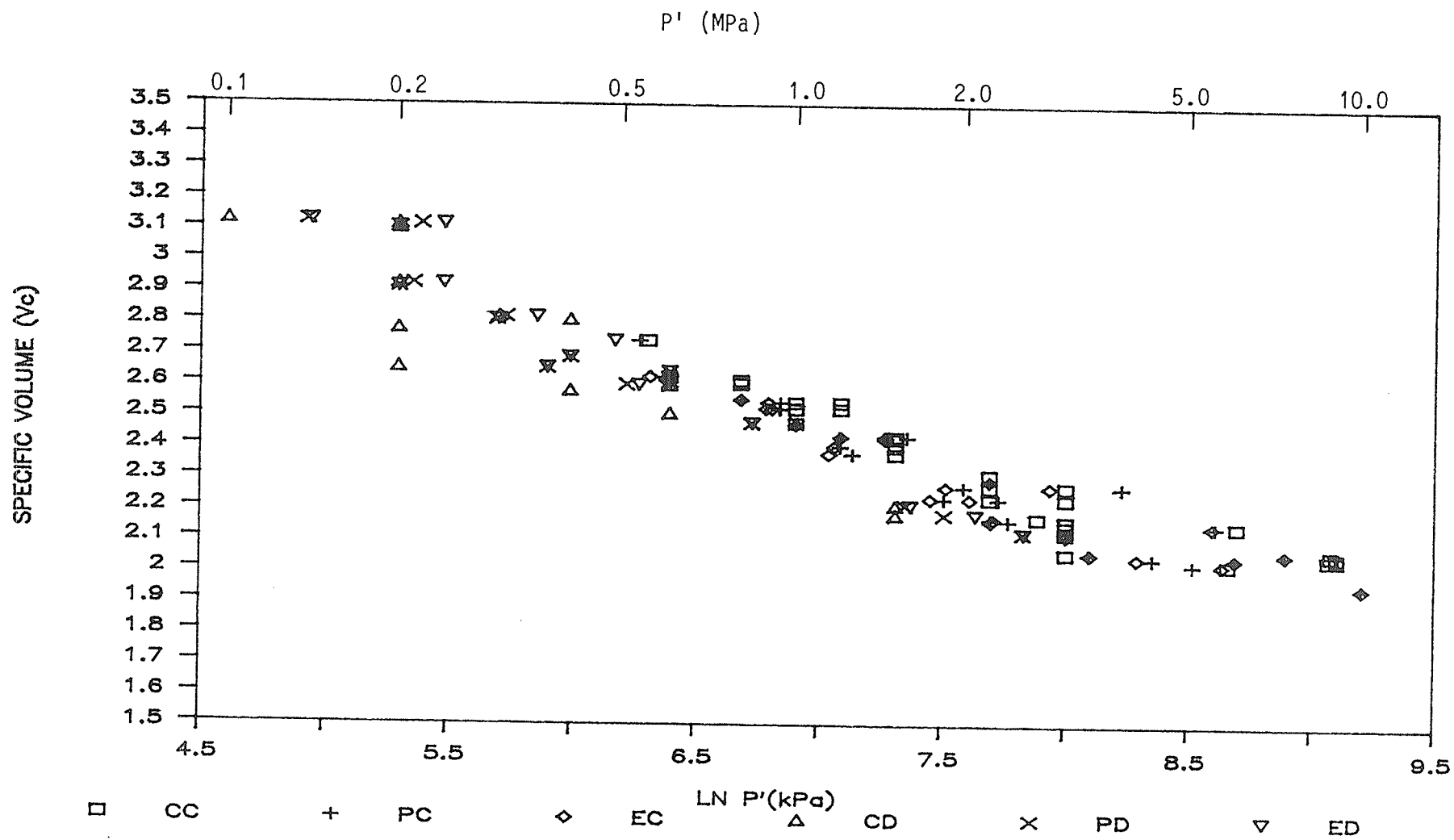


FIGURE 6.22 - V_c VERSUS $\ln (P')$ RELATIONSHIP AT THE END-OF-CONSOLIDATION, PEAK SHEAR, AND END-OF-TEST FOR COMPRESSIVE AND DILATIVE SPECIMENS

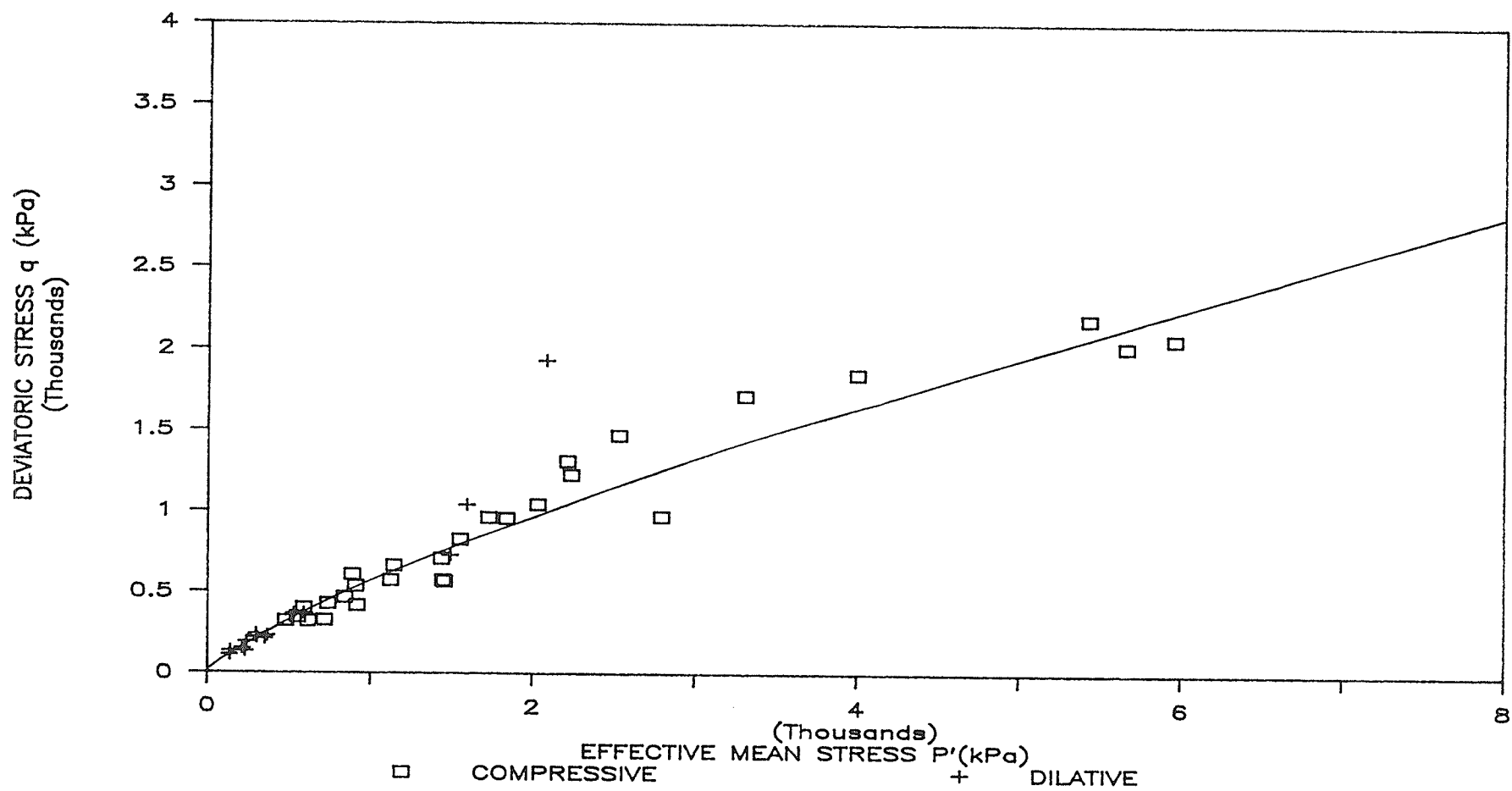


FIGURE 6.23 - SHEAR STRENGTH ENVELOPE BASED ON END-OF-TEST UNDRAINED TESTS

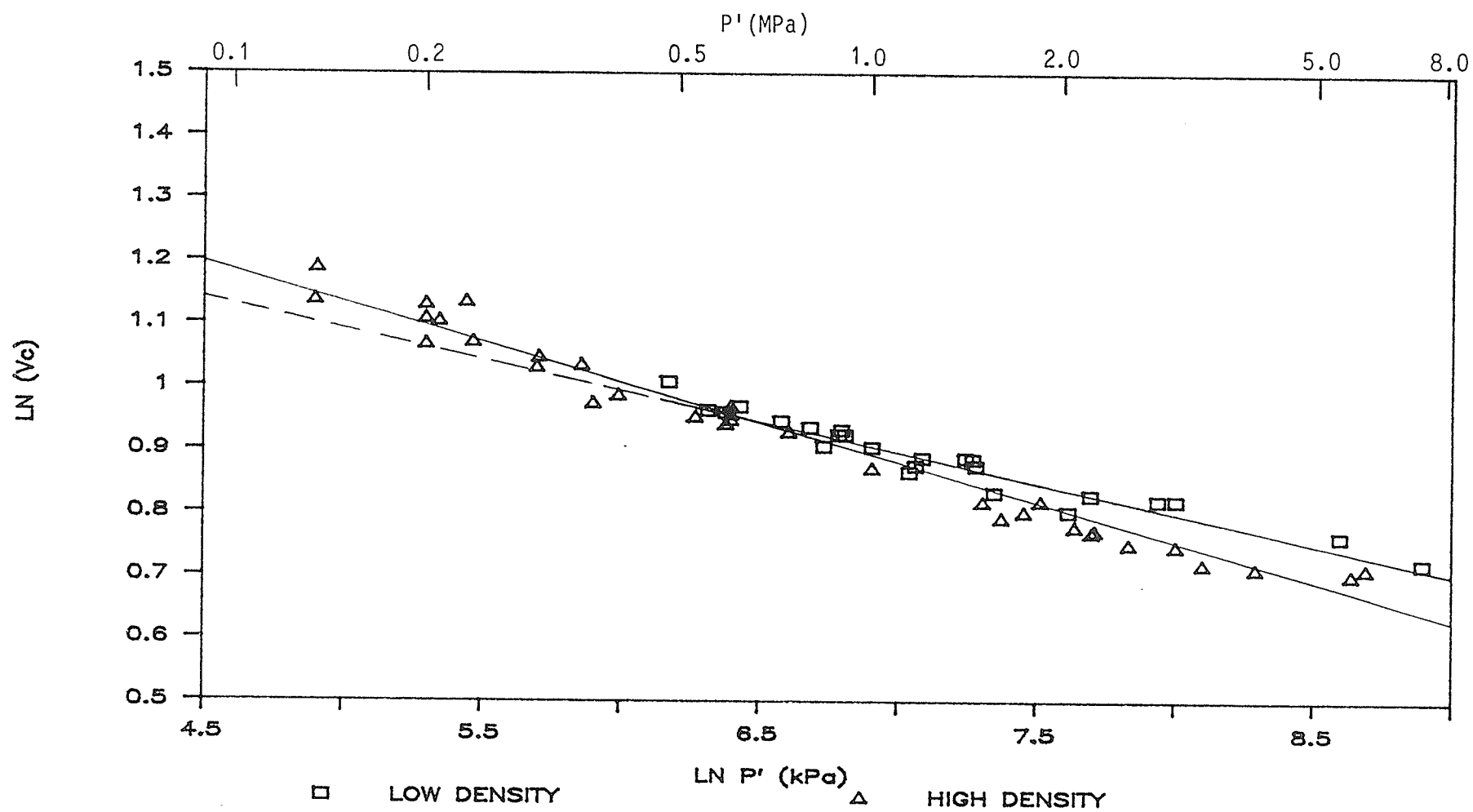


FIGURE 6.24 - LN (CLAY SPECIFIC VOLUME) VERSUS LN (EFFECT. MEAN. PRESSURE) RELATIONSHIP AT THE END-OF-TEST

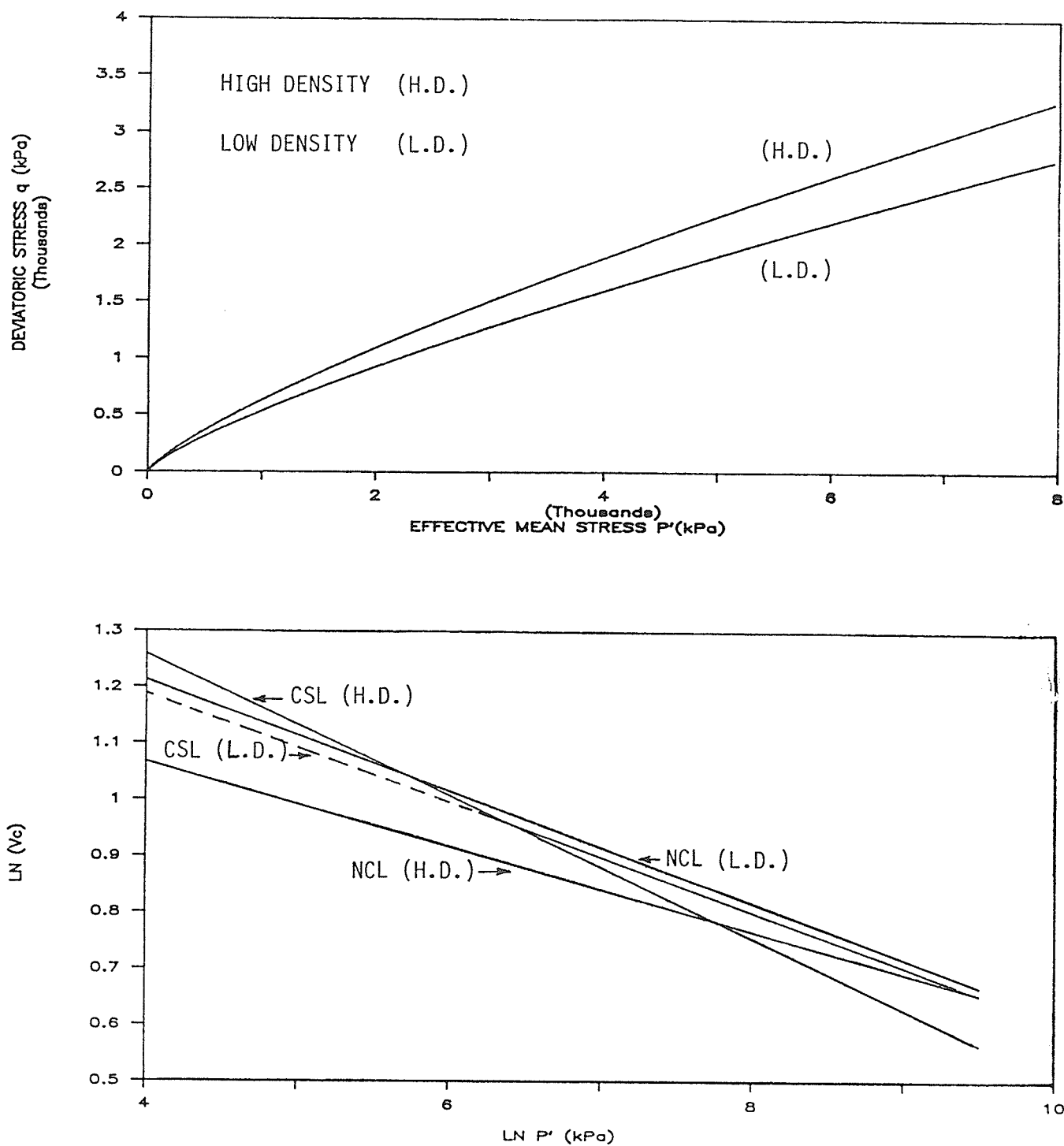
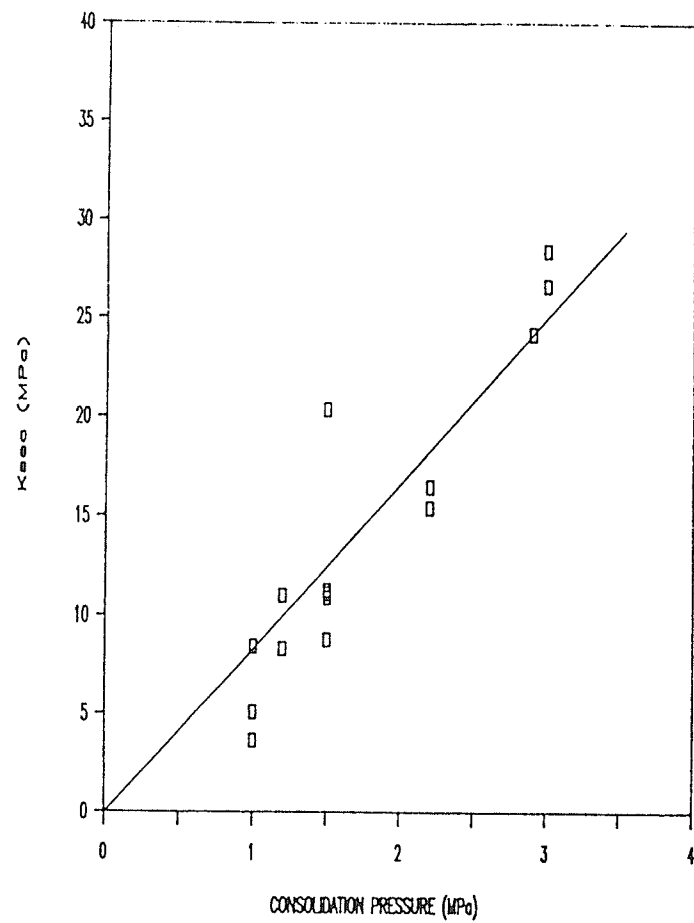
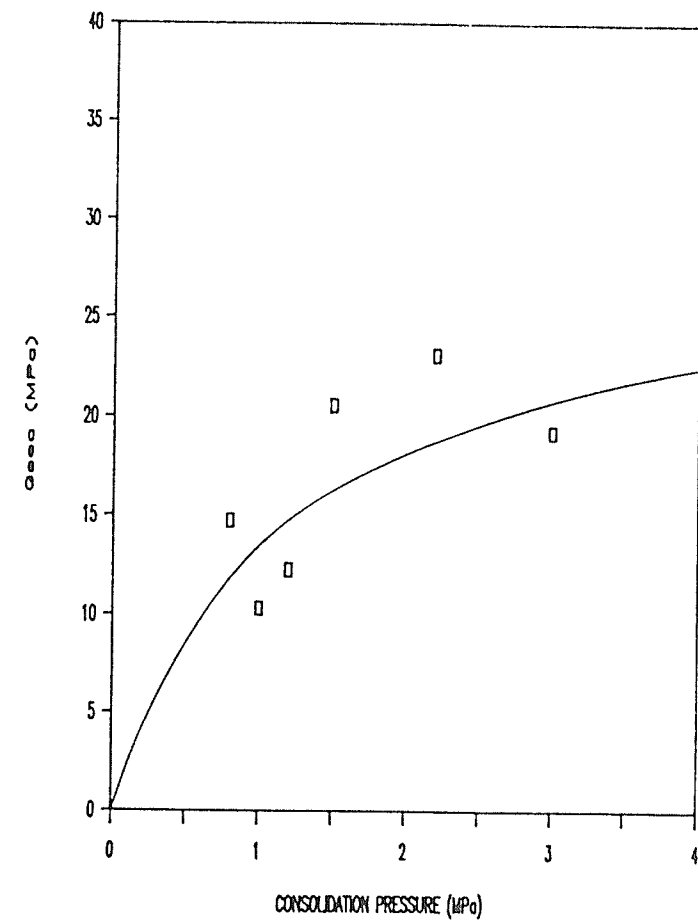


FIGURE 6.25 - FUNCTIONAL RELATIONSHIP FOR THE BEHAVIOUR OF THE BUFFER IN
 q , p' AND V_c SPACE



(A) SECANT BULK STIFFNESS



(B) SECANT SHEAR STIFFNESS

FIGURE 7.1 - PSUEDO-ELASTIC MODEL FOR LOW DENSITY BUFFER

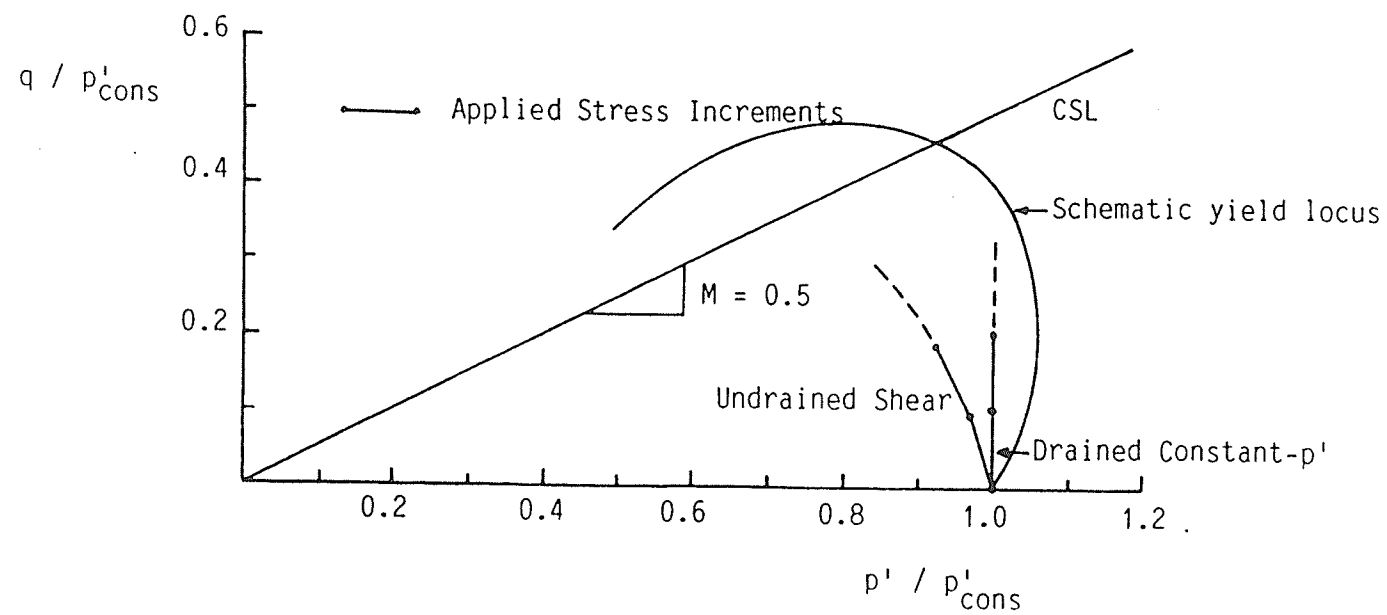


FIGURE 7.2 - NORMALIZED STRESS PATHS IN EARLY STAGES OF UNDRAINED AND DRAINED CONSTANT- p' SHEAR TESTS

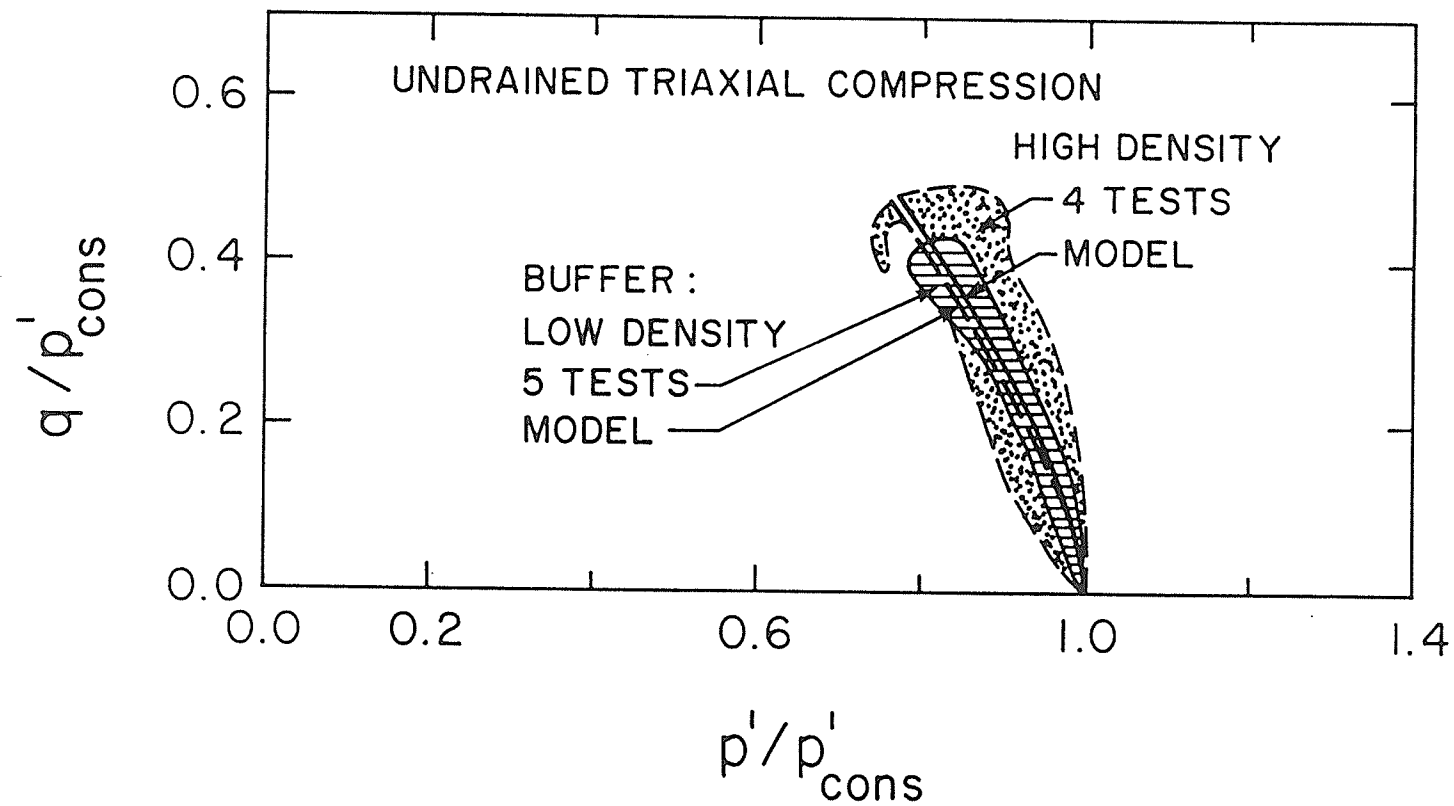


FIGURE 7.3 - NORMALIZED q/p'_{cons} , p'/p'_{cons} - RELATIONSHIP FOR LOW DENSITY AND HIGH DENSITY BUFFER

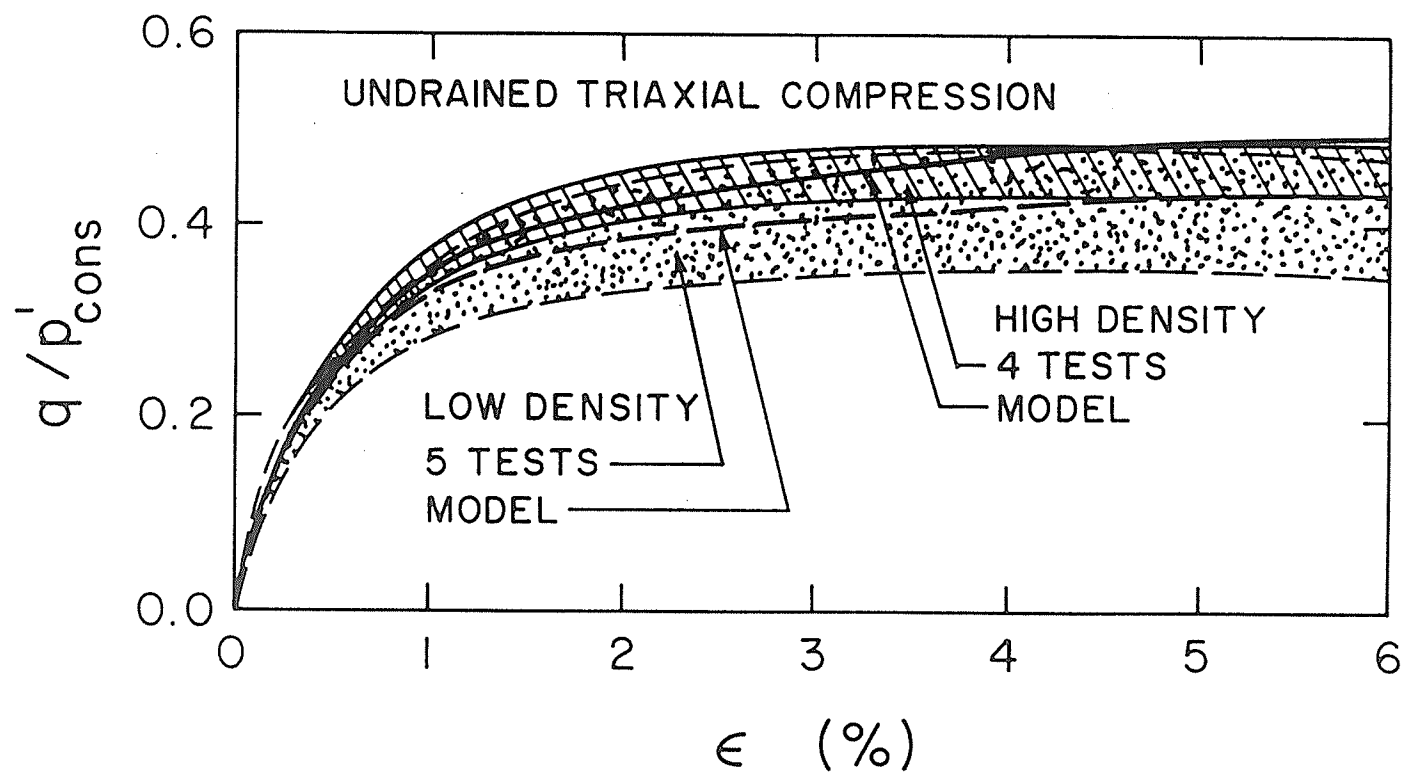


FIGURE 7.4 - NORMALIZED q/p'_{cons} , ϵ - RELATIONSHIP FOR LOW DENSITY AND HIGH DENSITY BUFFER

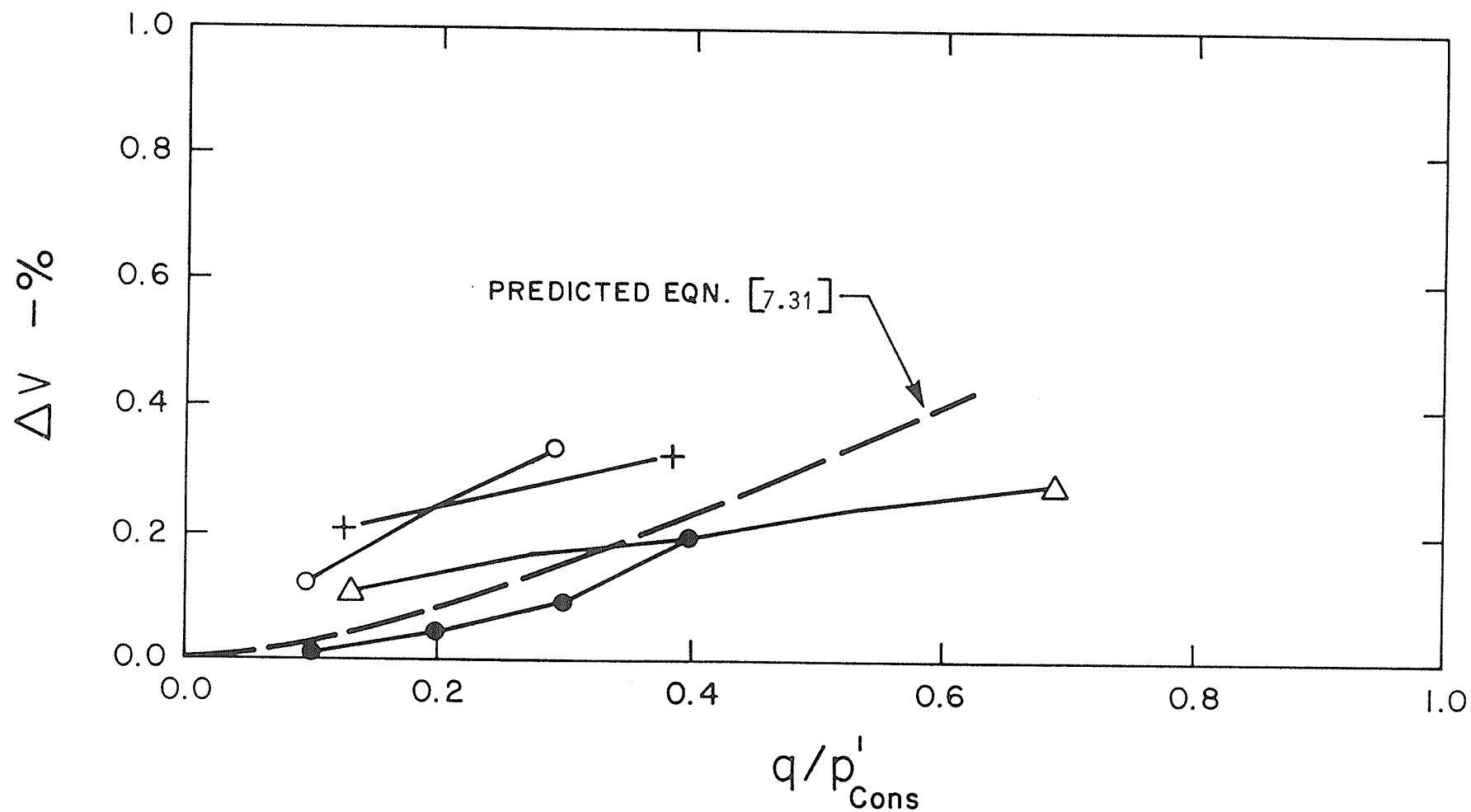


FIGURE 7.5 - COMPARISON OF PREDICTED AND MEASURED $\Delta V - q/p'_{Cons}$ CURVES IN INCREMENTAL CONSTANT- p' TESTS FOR LOW DENSITY BUFFER

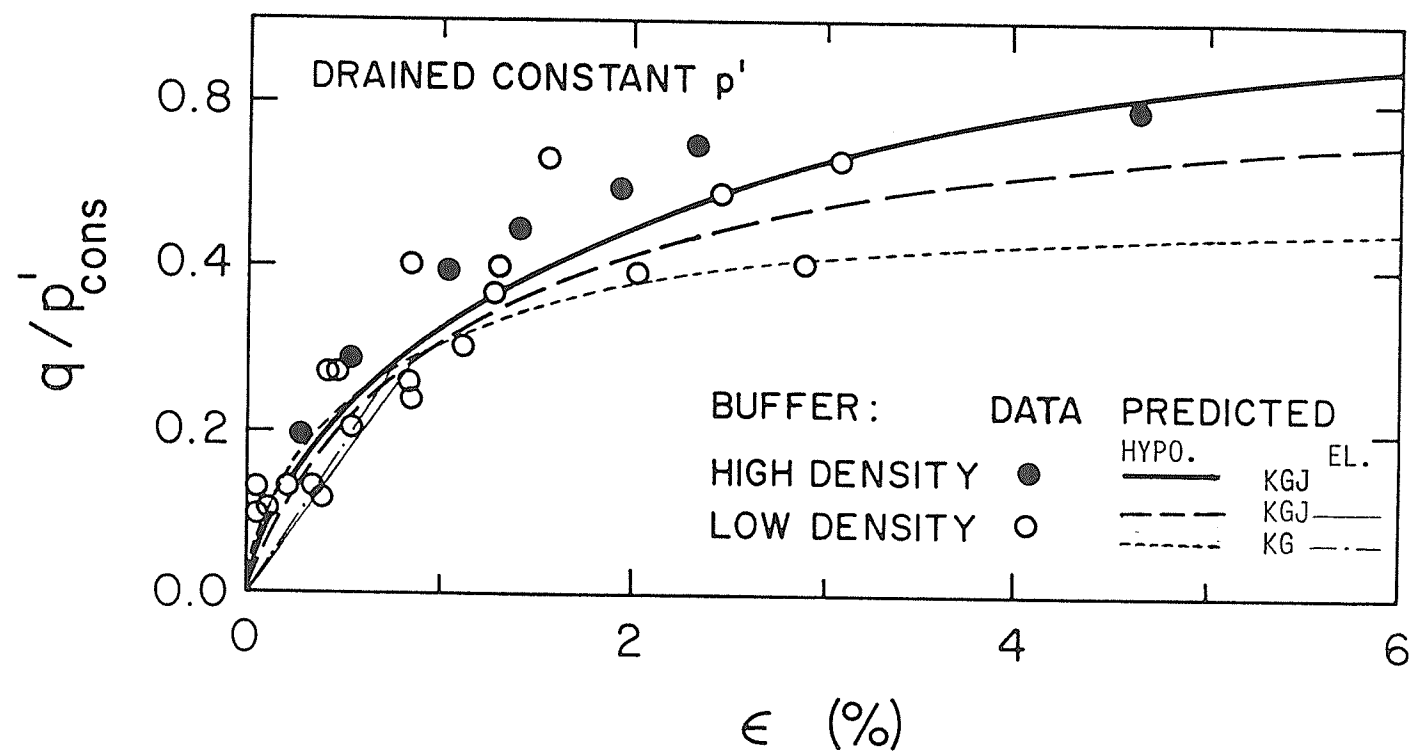


FIGURE 7.6 - MEASURED AND PREDICTED q/p'_{cons} VERSUS ϵ RESULTS FOR 'LOW DENSITY AND HIGH DENSITY BUFFER

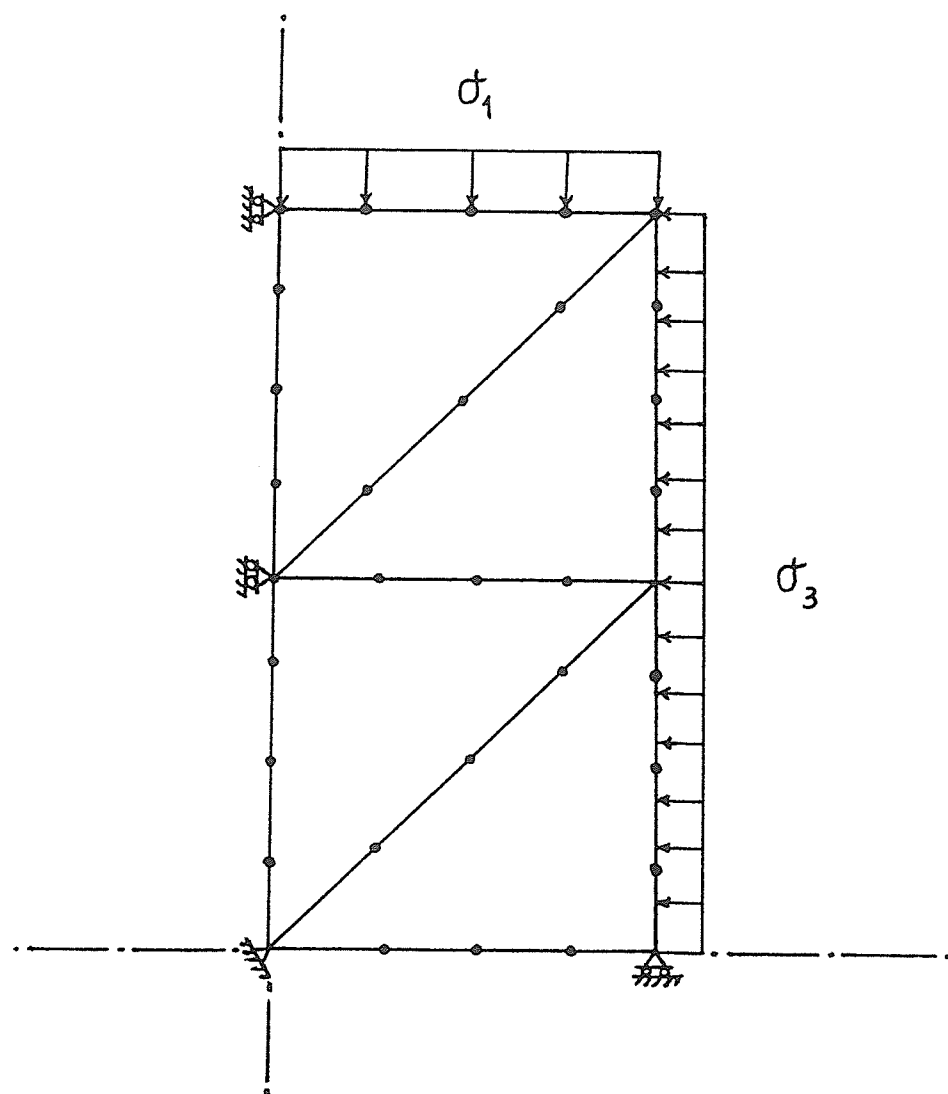


FIGURE 8.1 - FINITE ELEMENT MESH FOR ANALYSIS OF A LABORATORY TRIAXIAL SPECIMEN

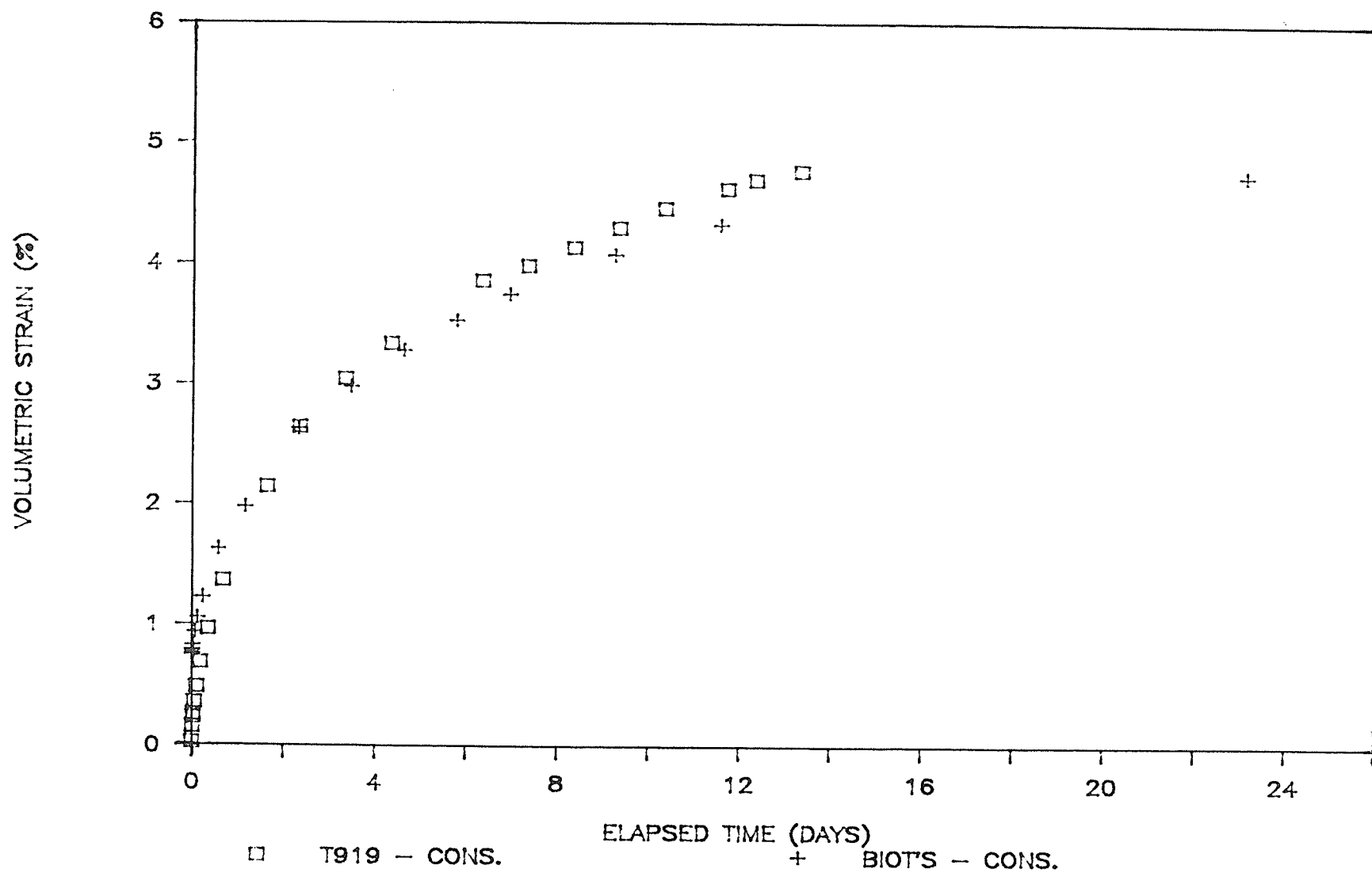


FIGURE 8.2 - COMPARISON BETWEEN CONSOLIDATION TEST RESULTS AT 1.5 MPa CONFINING PRESSURE AND PREDICTIONS OF BIOT'S THEORY USING CRISP

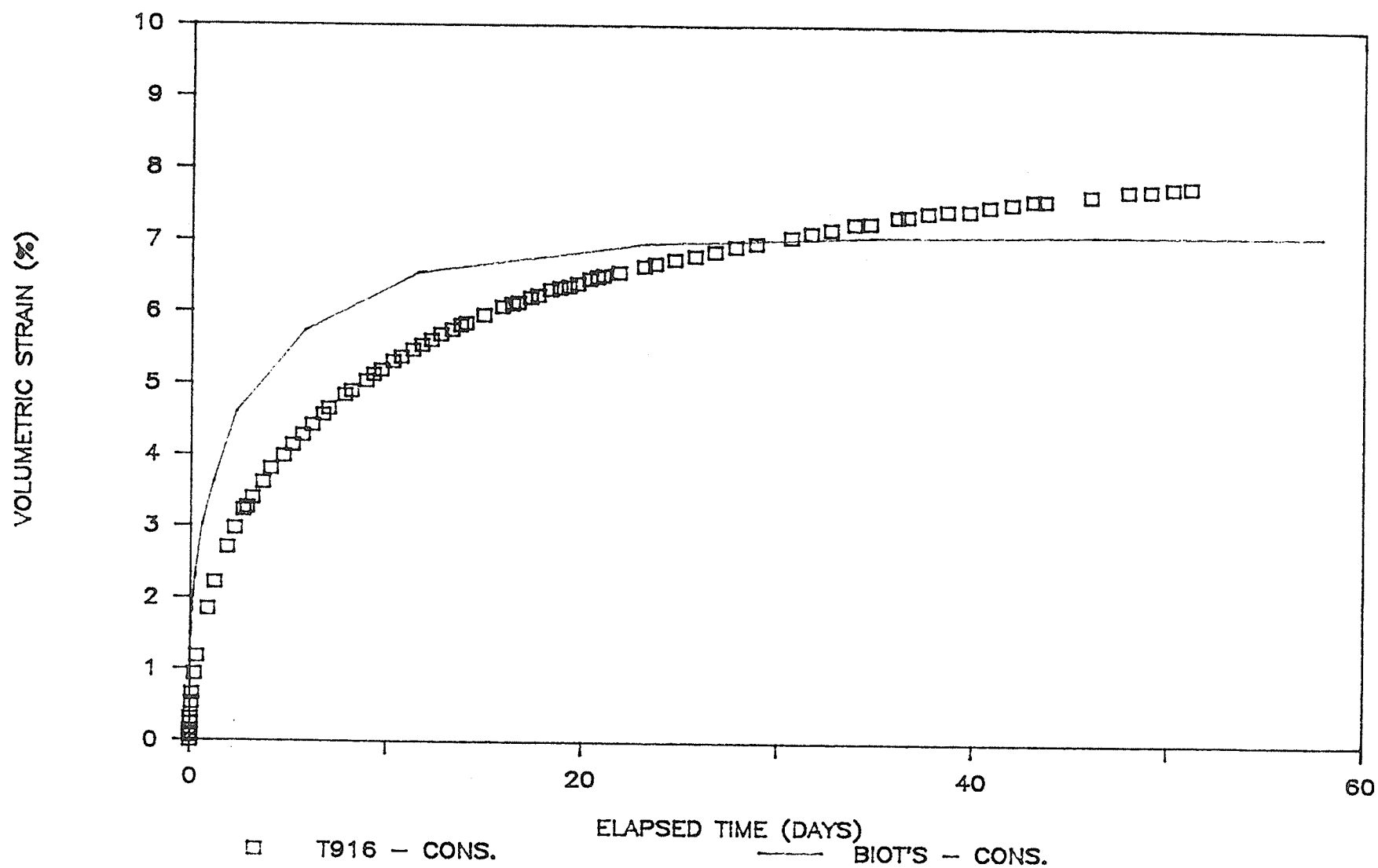


FIGURE 8.3 - COMPARISON BETWEEN MEASURED VOLUME STRAIN - TIME RELATIONSHIP AT 2.2 MPa CONFINING PRESSURE AND PREDICTED BEHAVIOUR BY BIOT'S CONSOLIDATION THEORY USING CRISP

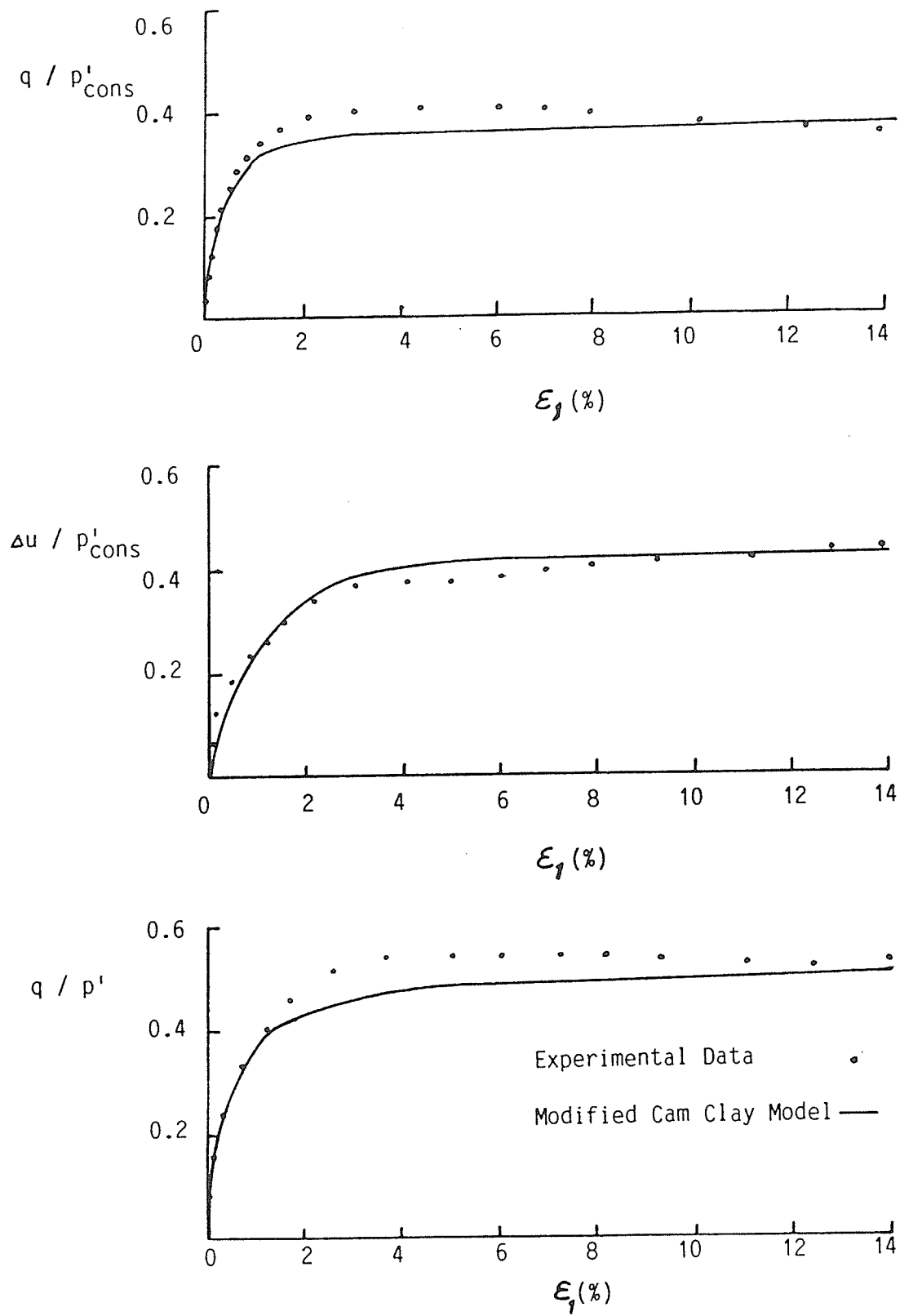


FIGURE 8.4 - COMPARISON BETWEEN MEASURED SHEAR BEHAVIOUR AT 3 MPa CONFINING PRESSURE AND PREDICTIONS OF MODIFIED CAM CLAY MODEL

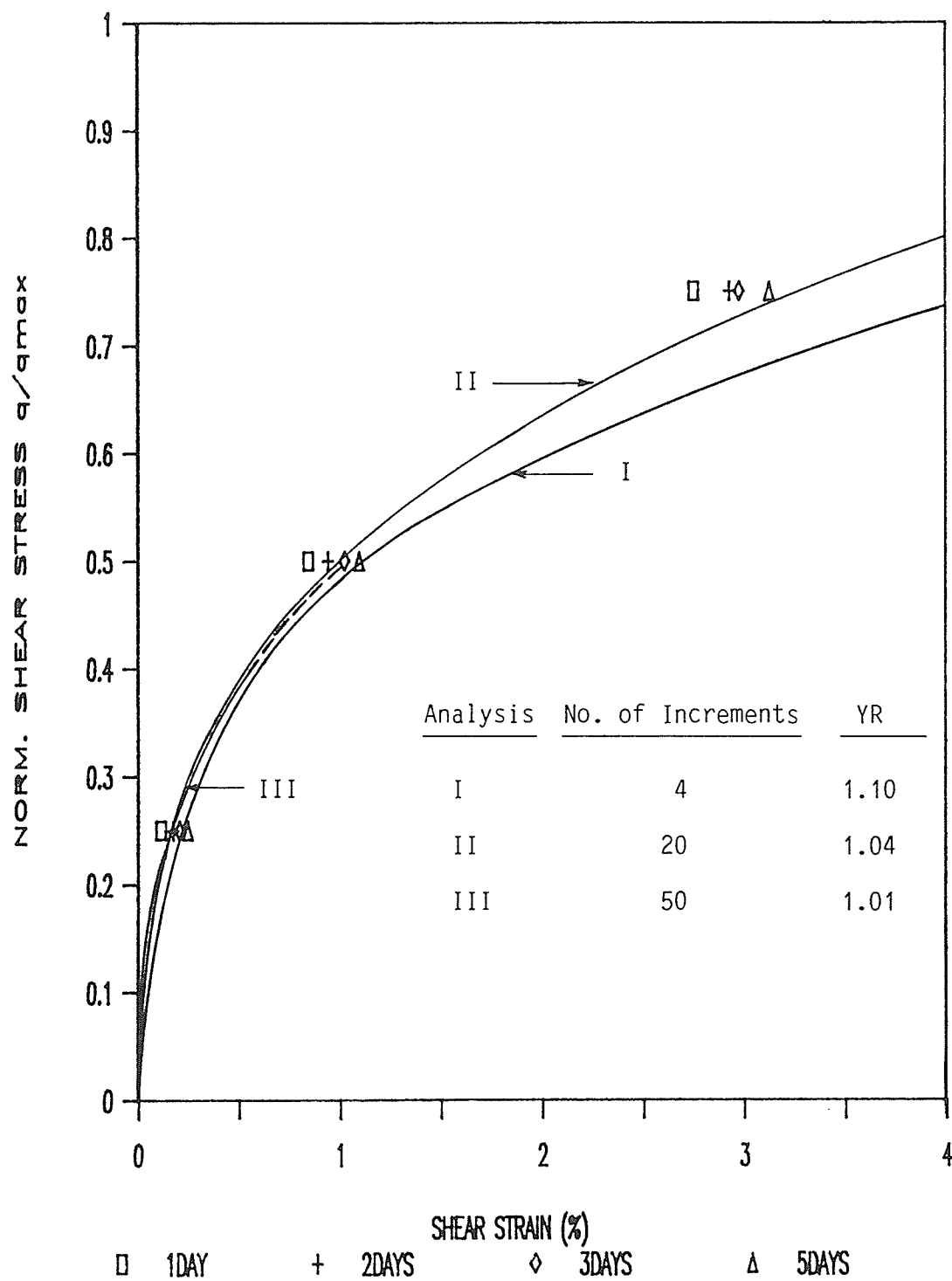


FIGURE 8.5 - COMPARISON BETWEEN MEASURED CONSTANT- P' DRAINED RESULTS FOR SPECIMEN T918 AND PREDICTIONS OF MODIFIED CAM CLAY MODEL

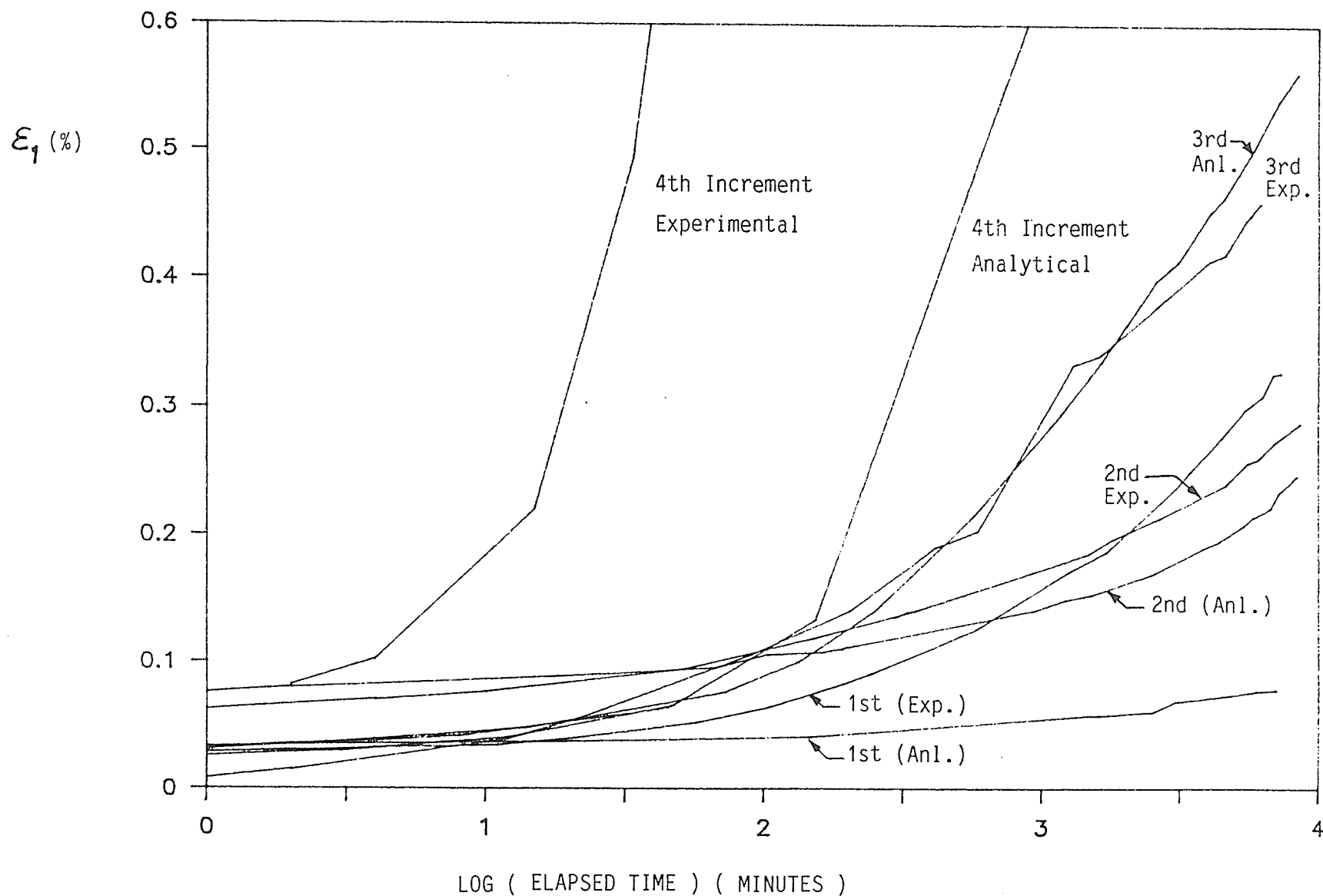


FIGURE 8.6 - COMPARISON BETWEEN MEASURED AXIAL STRAIN - TIME RELATIONSHIP IN CONSTANT- P^1 DRAINED TEST AND PREDICTIONS OF CRISP

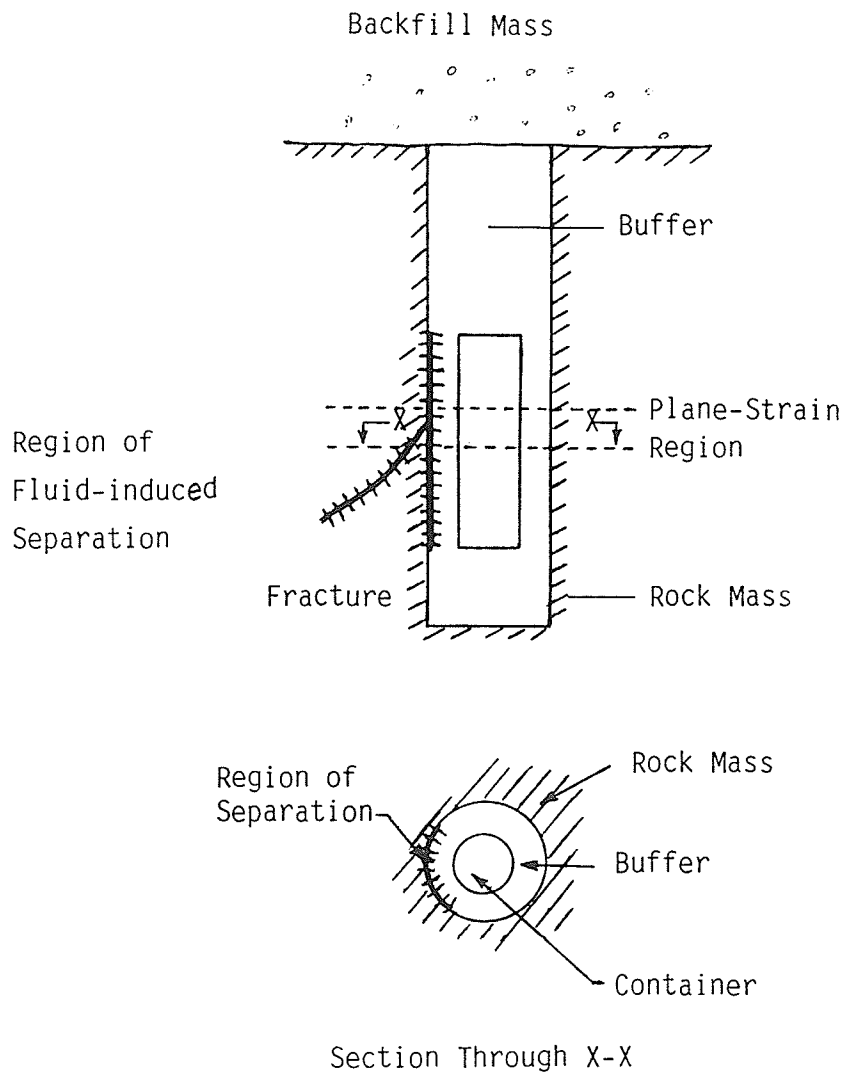


FIGURE 8.7 - POTENTIAL FLUID-INDUCED SEPARATION AT THE BUFFER-ROCK MASS INTERFACE ASSUMING PLANE-STRAIN CONDITION
(AFTER SELVADURAI ET AL. 1985)

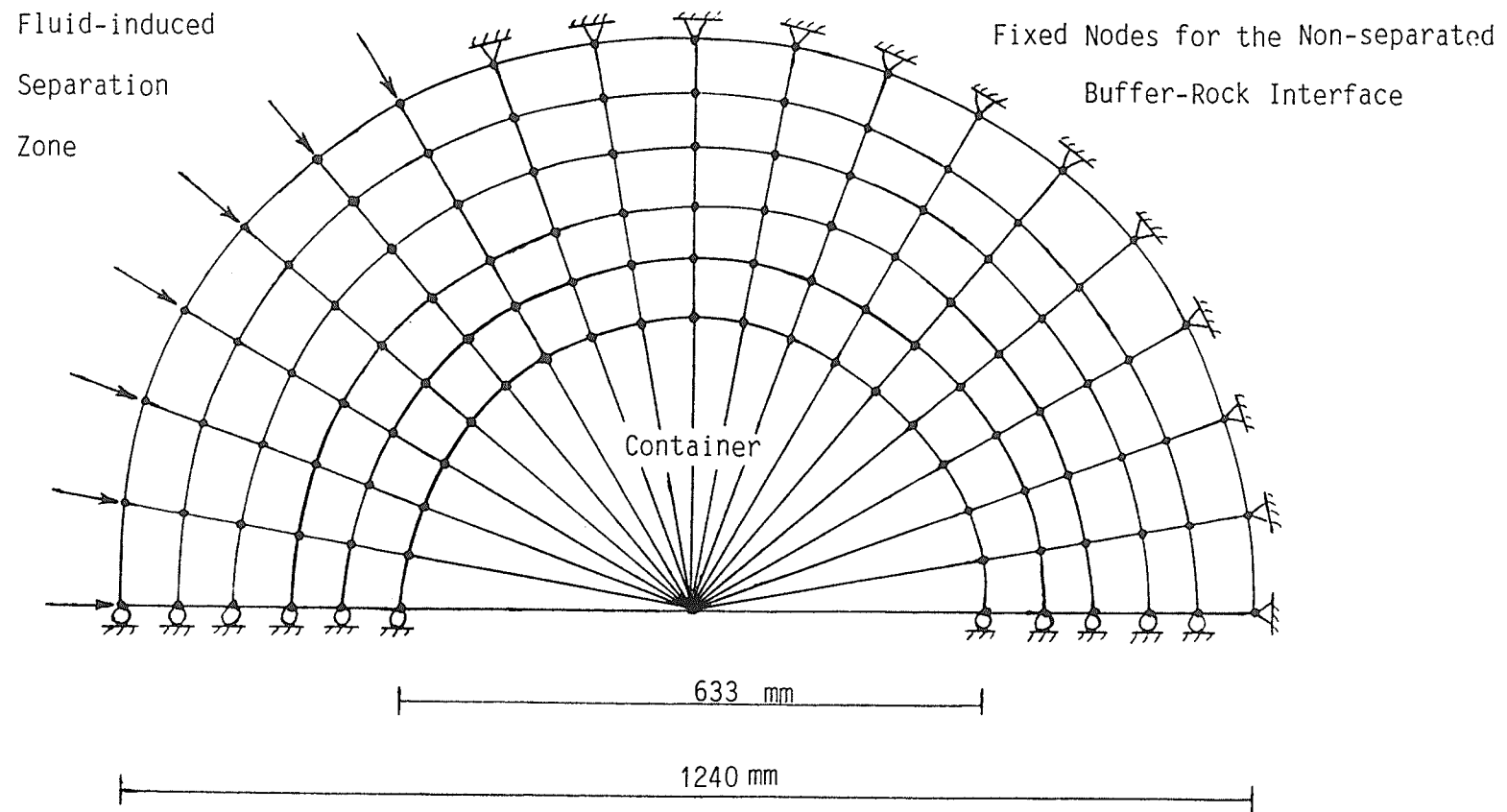


FIGURE 8.8 - FINITE ELEMENT MESH FOR THE CONTAINER-BUFFER-ROCK SYSTEM IN PLANE STRAIN CONDITION
(AFTER SELVADURAI ET AL. 1985)

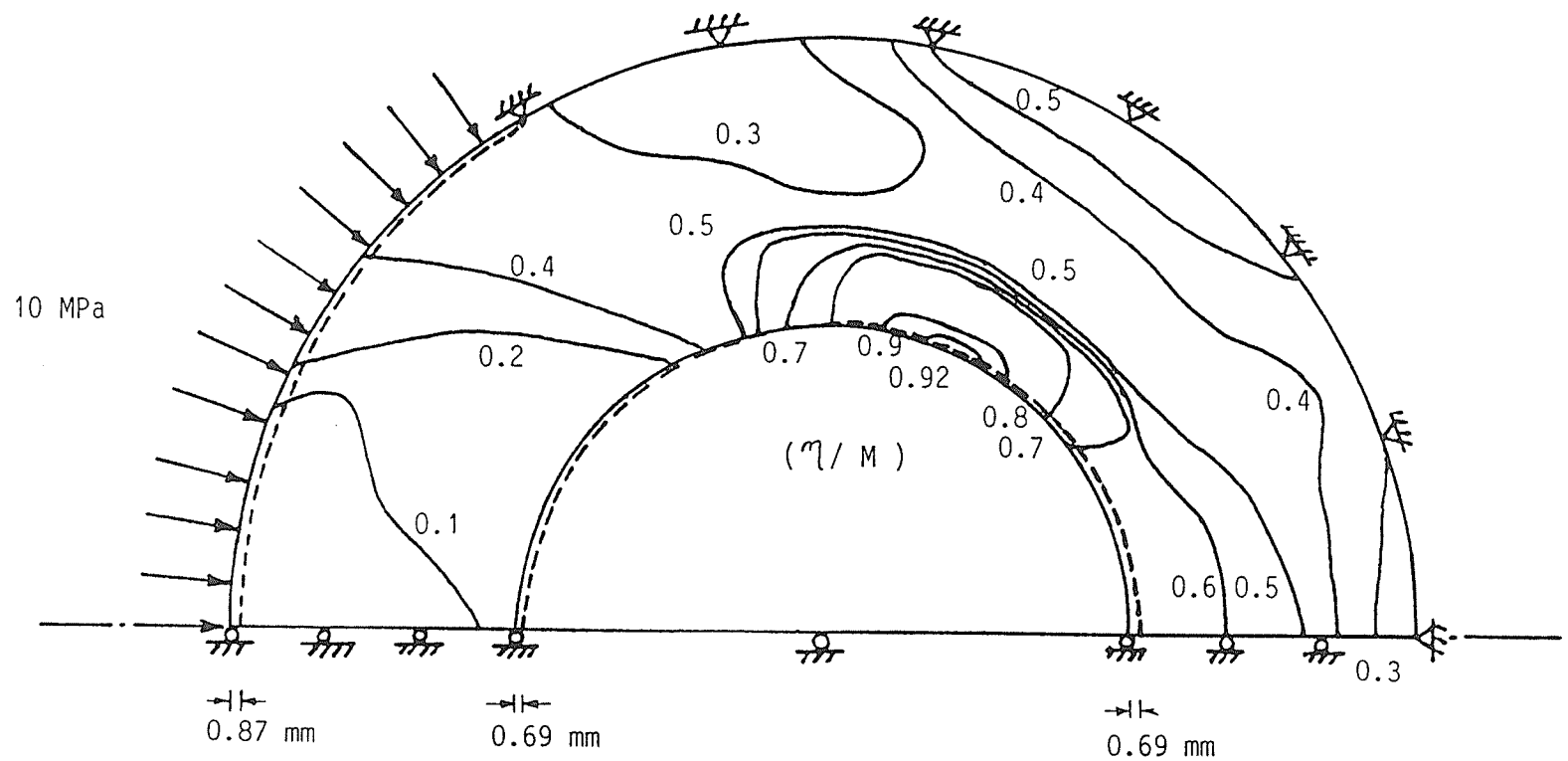


FIGURE 8.9 - STRESS RATIO DISTRIBUTION (σ / M) AND DEFORMATION OF THE BUFFER DUE TO A UNIFORM FLUID PRESSURE OF 10 MPa IN THE SEPARATION ZONE

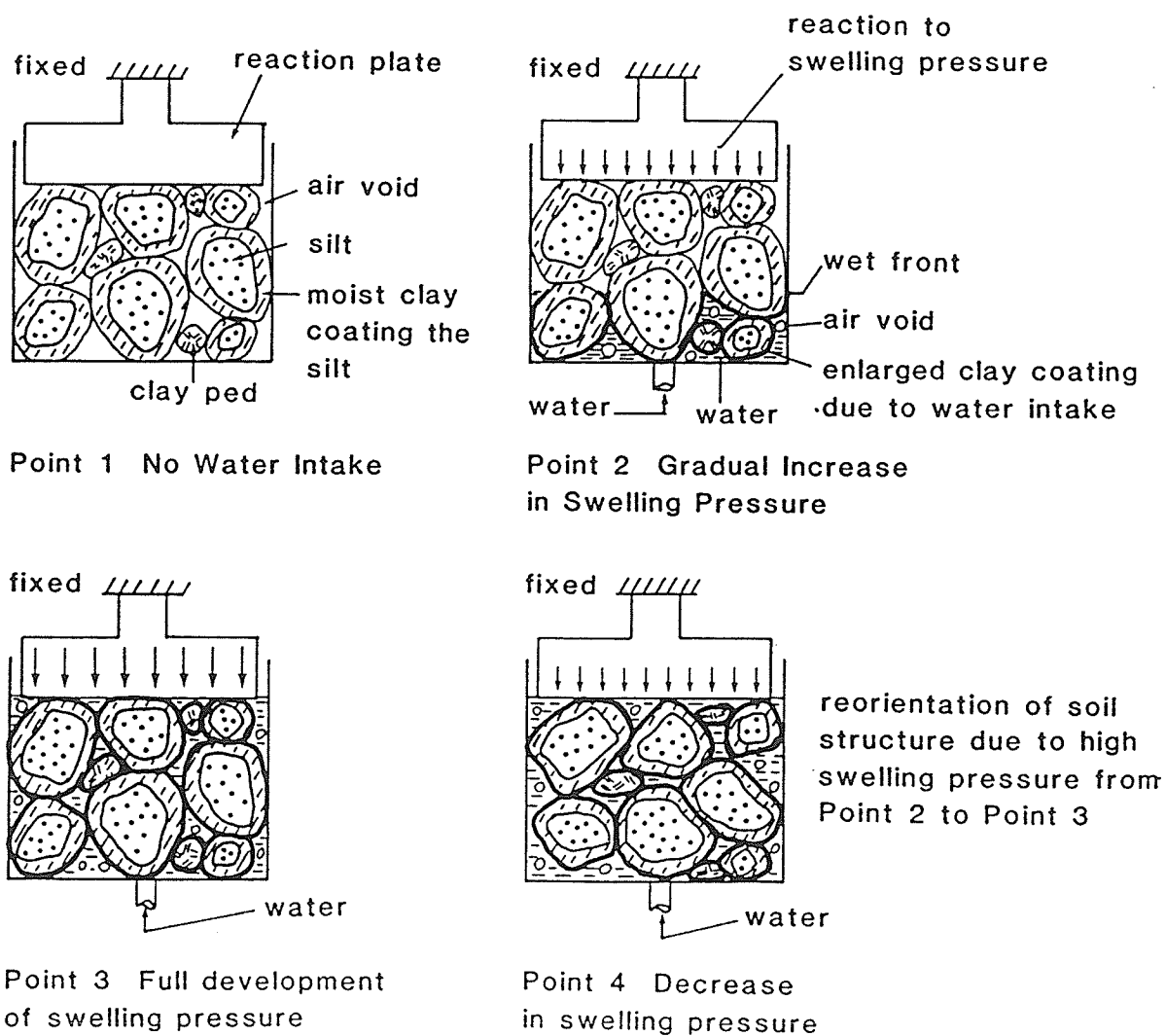
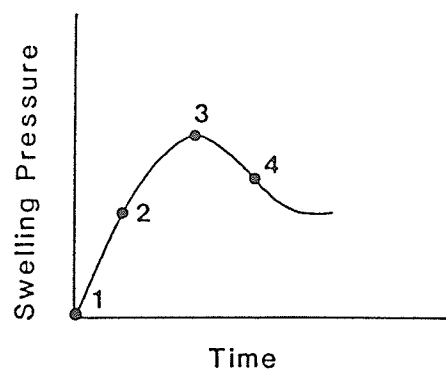


FIGURE 8.10 - SCHEMATIC OF SWELLING PRESSURE BUILD-UP AND RELIEF MECHANISM
IN CONFINED SWELL TESTS (AFTER YONG ET AL. 1988)

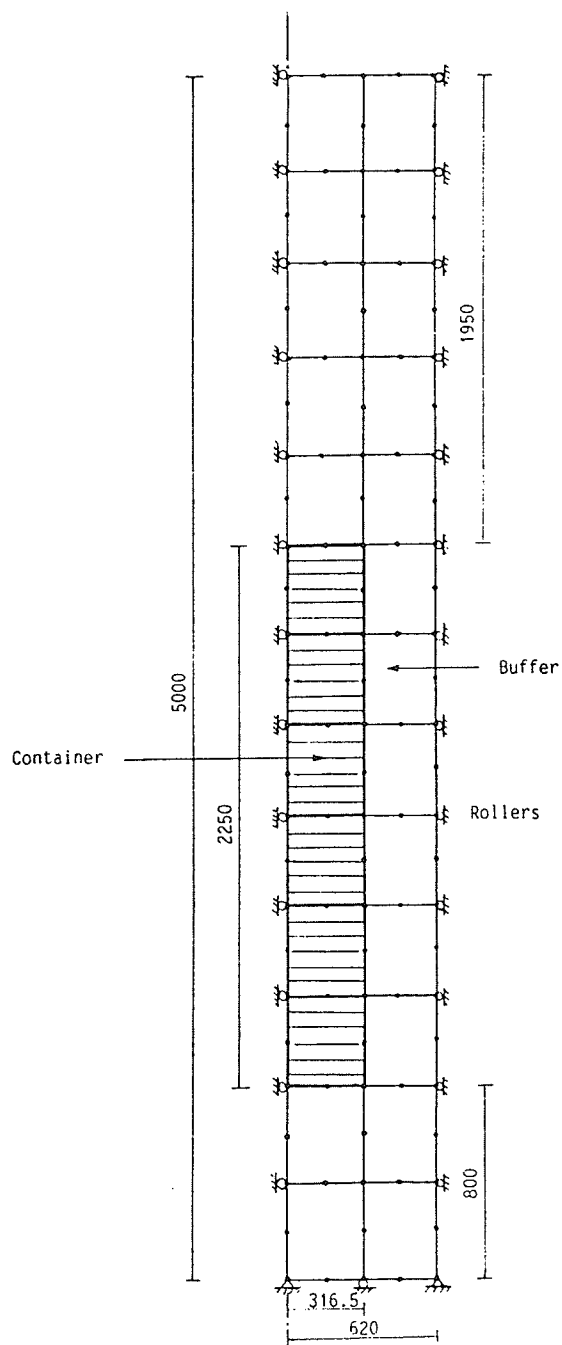
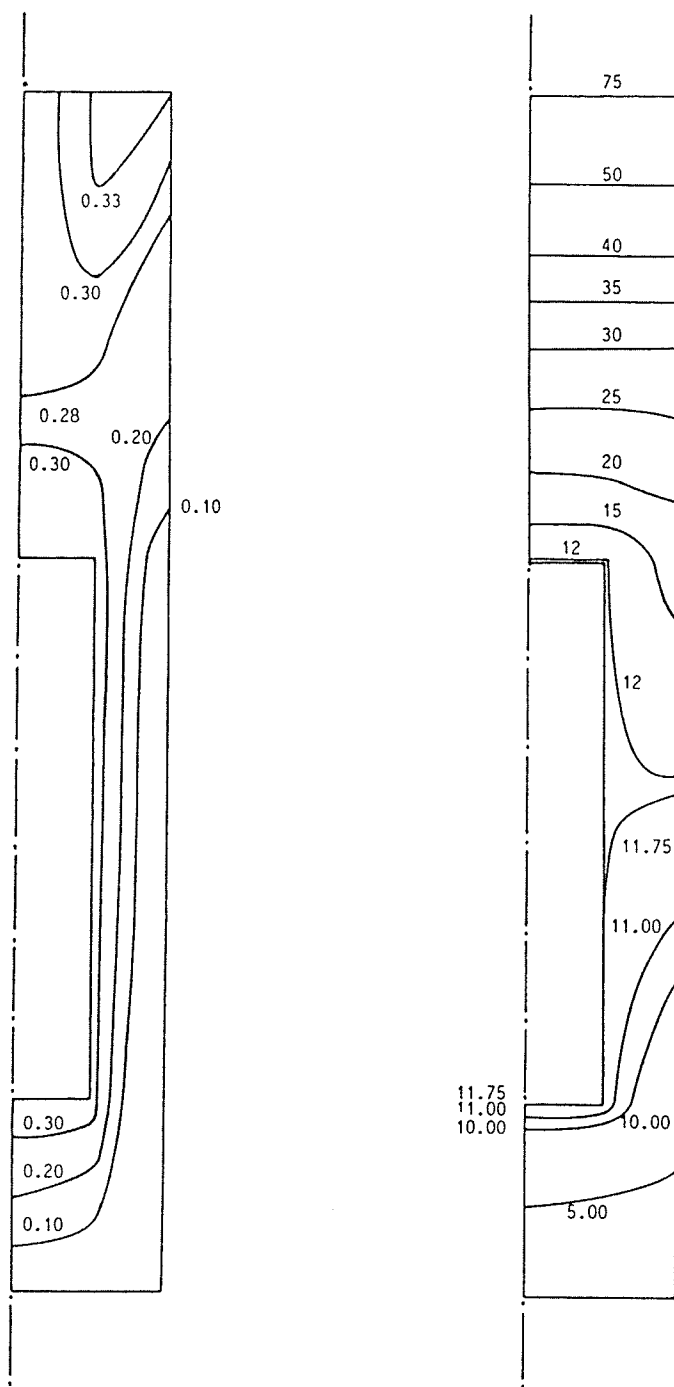


FIGURE 8.11 - FINITE ELEMENT MODEL OF THE CONTAINER-BUFFER-ROCK SYSTEM
IN AXI-SYMMETRICAL CASE



(A) UNDRAINED

(B) DRAINED

FIGURE 8.12 - VERTICAL DISPLACEMENT OF THE BUFFER DUE TO CONTAINER WEIGHT
AND BACKFILL SURCHARGE FROM THE FINITE ELEMENT ANALYSIS
(ALL DISPLACEMENTS ARE GIVEN IN MM)

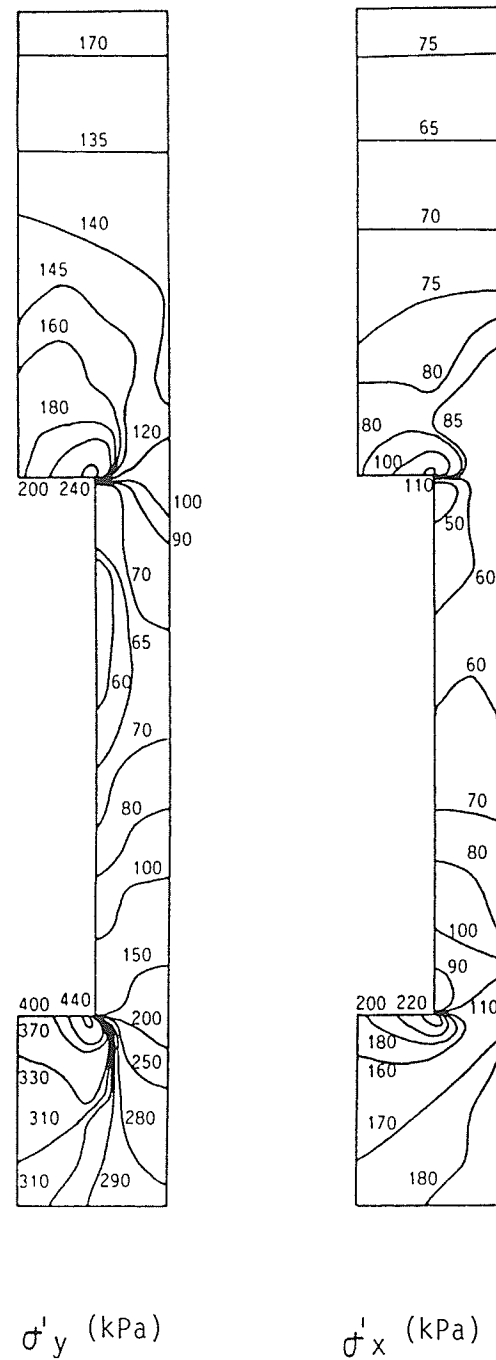
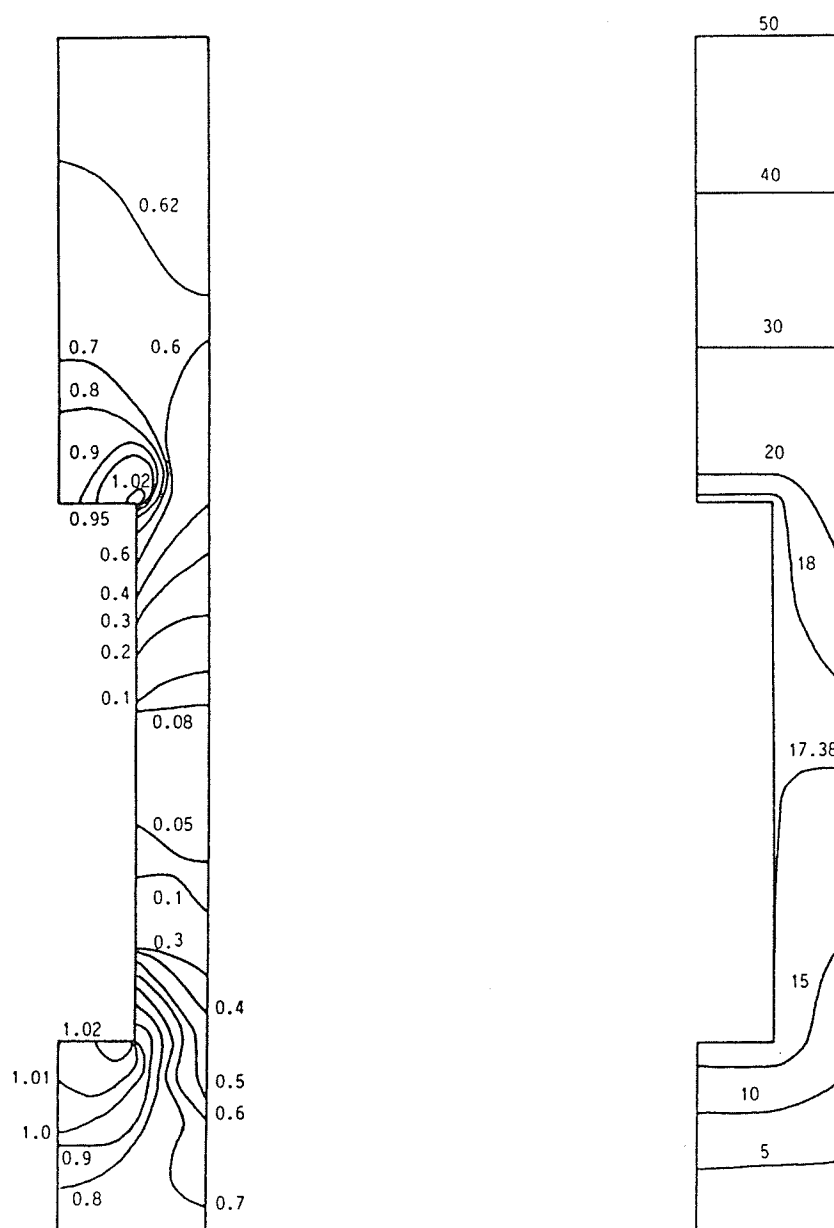


FIGURE 8.13 - VERTICAL AND HORIZONTAL STRESS DISTRIBUTIONS IN THE BUFFER
DUE TO CONTAINER WEIGHT AND BACKFILL SURCHARGE UNDER DRAINED
CONDITIONS FROM THE FINITE ELEMENT ANALYSIS
(ALL STRESSES ARE GIVEN IN KPA)

(A) NORMALIZED STRESS RATIO (γ/M)

(B) VERTICAL DISPLACEMENT (MM)

FIGURE 8.14 - YIELDING OF THE BUFFER DUE TO PARTIAL RELEASE OF SWELLING PRESSURES IN INTERACTION WITH BACKFILL

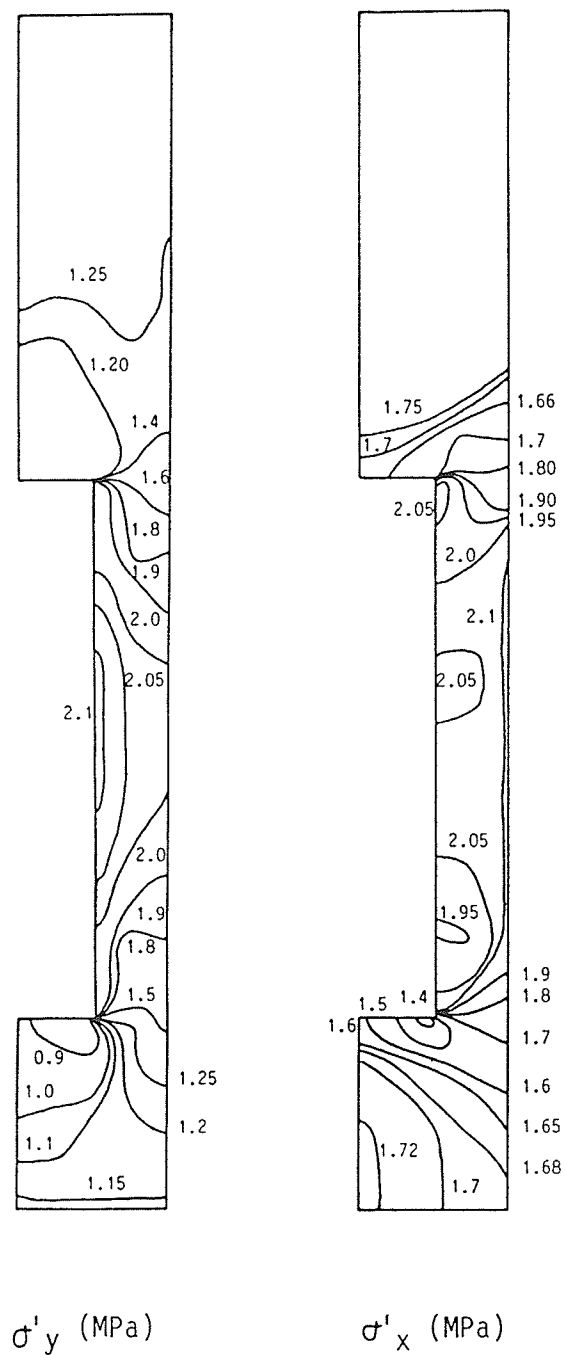


FIGURE 8.15 - VERTICAL AND HORIZONTAL STRESS RE-DISTRIBUTIONS IN THE BUFFER
UPON RELEASE OF SWELLING PRESSURES OF THE BUFFER DUE TO
DEFORMATION OF THE BACKFILL
(ALL STRESSES ARE GIVEN IN MPa)