BEHAVIOUR OF SENSITIVE LEDA CLAY UNDER SIMPLE SHEAR LOADING

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by

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Abstract

An experimental investigation was conducted on undisturbed sensitive Leda clay samples from the Ottawa area using a simple shear device. Both small strain and large strain tests were conducted under a range of consolidation stress levels. The Carleton University simple shear device was improved to measure shear properties at small strain amplitudes. Strength tests at large strain levels were performed in order to study the monotonic and cyclic behaviour of sensitive Leda clays, while quasi–cyclic tests were carried out to obtain material specific modulus and damping curves at small strain amplitudes.

Strength tests indicate that Leda clay might reach a steady state during monotonic loading. Under cyclic loading, the clay deformed with progressively increasing cyclic strain, and liquefaction becomes a concern when cyclic stress amplitudes exceed at least 60% of their monotonic undrained shear strength. Few small amplitude stress or strain cycles do not produce significant strength loss or do not change the subsequent shear behaviour.

Successful application of small strain quasi cyclic tests proved to be an optimum means of obtaining modulus reduction and damping data in the laboratory. Material specific modulus reduction and damping ratio curves are presented for Ottawa area Leda clay based on this experimental data. These results show that the behaviour of sensitive clays cannot be simply equated to that of typical non sensitive clays with high plasticity index, if accurate site response analysis results are required.

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CSR	Cyclic Stress Ratio (= τ_{cyc}/σ'_{vc})
CRR	Cyclic resistance ratio
C_u	Undrained cohesion
D_r	Relative density
G_{max}	Maximum shear modulus
G_s or G_{sec}	Secant shear modulus
G_{tan}	Tangent shear modulus
K_0	Coefficient of earth pressure at rest
LI	Liquidity index
LL	Liquid limit
LVDT	Linear variable differential transducer
N	Number of loading cycles
NBCC	National building Code of Canada
NC	Normally consolidated
OC	Over consolidated
OCR	Over consolidation ratio
PI	Plasticity index
PL	Plastic limit
PWP	Pore water pressure
S_p	Peak shear stress
S_r	Residual shear stress
S_u	Undrained shear strength
S_{uh}	Undrained shear strength in horizontal orientation
S_{uv}	Undrained shear strength in vertical orientation
$S_{u\alpha}$	Undrained shear strength at an angle α from bedding plane
T	Period
c'	Effective cohesion
e	Void ratio
p'	Effective stress parameter
q	Shear stress parameter

v	Specific volume
v_s	Shear wave velocity
z	Depth from ground surface
Δu	Excess pore water pressure
α	Angle measured from horizontal bedding plane
α_σ	Inclination of principal stress
γ	Shear strain
γ_b	Bulk unit weight / density
γ_{cyc} or γ_c	Cyclic shear strain
γ_d	Dry unit weight / density
γ_{max}	Maximum shear strain
γ_{tl}	Threshold shear strain for linear behaviour
γ_{tv}	Threshold shear strain for volume change / pore pressure
δ	Modulus degradation index
ε_v	Volumetric strain
ξ	Damping ratio
ρ	Density
σ_1 & σ'_1	Major principal & Effective major principal stress
σ_2 & σ'_2	Intermediate principal & Effective intermediate principal stress
σ_3 & σ'_3	Minor principal & Effective minor principal stress
σ'_m	Effective mean confining stress
σ'_v	Vertical effective stress
σ'_{vc}	Vertical effective consolidation stress
σ'_{pc}	Pre consolidation stress
ϕ'	Effective internal angle of friction
ϕ'_p	Effective internal angle of friction at peak stress state
ϕ'_r	Effective internal angle of friction at residual stress state
ϕ_u	Undrained friction angle

1 INTRODUCTION

1.1 Background

Sensitive clay is considered a problematic soil in geotechnical engineering practice. It is vulnerable to severe shear strength loss when its natural structure is disturbed due to remoulding. These clays are prevalent in various parts of the world especially in the Northern regions as these parts had been covered by glaciers during the last glacial era (Brand & Brenner, 1981). Countries such as Canada, Sweden, Norway and US states of Alaska have sensitive clays in vast, densely populated areas.

In Canada, sensitive clays are prevalent in Eastern Ontario and Quebec along the St. Lawrence and Ottawa River valleys. The salinity in these soils is due to the Champlain Sea which covered these regions about 10,000 years ago due to retreating glaciers (Canadian Mortgage and Housing Corporation, 2001). Hence, this sensitive clay is generally called 'Champlain Sea clay' or sometimes 'Leda clay'. The Leda clay poses a potential hazard to the densely populated regions in Eastern Canada. This clay has been responsible for a large number of landslides that occurred in this region. The Saint-Jean-Vianney landslide, which occurred in 1971, claimed the lives of 31 people, damaged properties and created a crater of approximately 324 000 square meters (Tavenas et al., 1971). In addition, a similar disaster was averted in Lemieux, Ontario since the town was relocated two years before a landslide in 1993. However, it created a large crater and an estimated 2.8 to 3.5 million cubic meters of sand, silt and liquefied clay collapsed into a river valley as seen in Fig 1.1 (Evans & Brooks, 1994).

Fig 1.1 The Lemieux landslide (after Evans & Brooks, 1994)

Furthermore, Eastern Canadian clays are vulnerable to cyclic loading events such as earthquakes. The magnitude six earthquake occurred in Saguenay, Quebec in 1988 is considered the largest earthquake in Eastern Canada in recent times. In addition, various smaller magnitude events such as the Val des Bois earthquake in 2010 (which was just over 50 km from Ottawa) also pose significant threat to these sensitive clays.

It is necessary for engineers to design safe structures to avoid disasters in this challenging material. A required input for seismic ground response analysis is the modulus and damping characteristics of the soil. This is generally provided in the form of modulus reduction and damping curves. However, there is a serious lack of information of material specific curves for Leda clay, and design practice is often based on perceived equalities between the behaviour of normal and sensitive clays. Ground response analysis are becoming routine given the seismic regulations prescribed in the 2005 (and 2010)

building codes, and as a result an urgent need exists to better understand the modulus reduction and damping characteristics of Leda clay.

A number of researchers have investigated various aspects of Leda clay (e.g. Mitchell & King, 1976; Lefebvre et al, 1989; Silvestri et al, 1989; Tala, 2010; Rasmuseen, 2012). But, no attempts have been made until recently to establish material specific modulus reduction and damping data in Leda clay. In addition, given the recent focus on seismicity in this region, better understanding of the liquefaction potential in this material would also be of value. Hence, a comprehensive laboratory study that can contribute some data to address these concerns would be valuable to the practicing engineers.

In addition, soil behaviour is known to be loading path dependent. But, experimental investigations of Leda clay under simple shear conditions, which represent the loading conditions during an earthquake very closely, have been very limited. Moreover, Sivathayalan et al. (2014) indicate that simple shear investigation is most appropriate for design purposes as it yields a conservative measure of the cyclic resistance, even when considering the effects of principal stress rotation and multi-dimensional loading.

1.2 Objectives

The primary objective of this research programme is to evaluate the validity of using high plasticity index (PI) modulus reduction and damping curves from the literature for designs in Leda clay. If such an approach is established to be inappropriate then this research will propose material specific modulus reduction and damping curves for ground response analysis in Leda clay. In addition, the monotonic and cyclic behaviour of Ottawa area Leda clay under simple shear conditions will also be studied. Even though, various studies have investigated shear strength behaviour of clays at large strains using

simple shear, typical simple shear devices are not suitable for small strain measurements. An attempt has been made in this research study to enhance the simple shear device at Carleton University to enable small strain measures that can yield modulus reduction and damping data under simple shear loading. If successful, this will avoid the need for the highly specialized resonant column tests that are employed to investigate small strain behaviour. The required modifications for the data acquisition and instrumentation will be implemented and their suitability for small strain measurements will be evaluated.

This study will also investigate the general monotonic and cyclic strength characteristics, and assess the strength loss characteristics in the material with loading history to evaluate the behaviour of partially-remoulded material. The material specific modulus and damping curves will be used to conduct a cyclic site response analysis and the results will be compared to those obtained using generic data from the literature. While investigating these behaviours and properties, this investigation intends to look at possible means of maximizing the efficiency and effectiveness of testing undisturbed specimens, since obtaining good quality undisturbed samples is an expensive undertaking. Even if financial resources are not a concern, it is difficult to find essentially identical soils samples that are required for fundamental studies in nature. The successful use of these techniques would curtail the large expenses and time required to obtain higher quantities of undisturbed samples, and the outcome of this research would enable geotechnical engineers to conduct reliable ground response analysis.

1.3 Organization of Thesis

A critical review of relevant literature is presented in chapter 2 following this introductory chapter. The monotonic and cyclic strength characteristics of clays and

modulus and damping curves that are necessary to carry out cyclic site response analysis are discussed in detail. Furthermore, this chapter illustrates specific aspects of the behaviour and characteristics of sensitive clays.

Chapter 3 presents details of the experimental programme. It documents the existing simple shear device and the modifications that have been made to facilitate testing at small strains to yield the data required for ground response analysis. Also, this chapter describes the details and geotechnical properties of the material tested.

Chapter 4 discusses monotonic and cyclic strength behaviour of Leda clay, strength reduction characteristics and factors that influences the gradual strength loss in this material. Chapter 5 discusses the modulus and damping characteristics of Leda clay, and proposes material specific modulus reduction and damping curves. Ground response analyses carried out using this data are presented, and compared to that using generic input data at the end of this chapter.

Chapter 6 presents the conclusions obtained from this research programme and proposes recommendations for future work. The summary results obtained from tests conducted in this research programme are presented in tables in the Appendices.

2 LITERATURE REVIEW

2.1 Introduction

Extensive amount of research has been carried out to better understand the shear strength characteristics of clays and sensitive clays during past few decades. Such soils are routinely encountered in geotechnical engineering practice. Sensitive clays are relatively more challenging to deal with due to problems associated with their sensitivity. The first part of this chapter attempts to provide a brief understanding of sensitive clays.

In the second part, the undrained monotonic shear characteristics of clays are discussed. Pore pressure dissipation in clays is slower due to their low permeability; as a result, undrained shear strength of clay is considered a very critical parameter in geotechnical practice and design.

Furthermore, clays are also vulnerable to undrained cyclic loading conditions such as earthquakes, wave loads and machine loads. Therefore, cyclic behaviour of clays and their post cyclic response are of interest for engineers. The response of clays to cyclic loads is reviewed in the third section of this chapter together with liquefaction potential in clays, and specifications available in literature to characterize vulnerable cohesive soils to liquefaction are discussed.

Fourth section elaborates the understanding in literature regarding the modulus and damping characteristics of clays which are important to carry out site response analysis. It is followed by a review of the current understanding of the behaviour of Eastern Canadian sensitive clays that have been the focus of this research.

2.2 Sensitive clays

Sensitive clays may change their behaviour from that of a particulate to that of a fluid due to de-structuration of the natural in-situ fabric of that clay. Such destruction of the fabric may be stress induced or due to physical remoulding of the soil. Sensitive clays generally have very high water content and weak bonds due to absence of balancing ions. When this weak structure is broken by a shock, it turns into a fluid state (Rankka et al, 2004). As a result, sensitive clays have a remoulded strength much smaller than their undisturbed strength. The term "sensitivity", which is the ratio between in situ shear strength and remoulded shear strength is used as the measure to represent the strength reduction.

$$Sensitivity = S_{u\,(undisturbed)} \,/\, S_{u\,(remoulded)}$$

Sensitive clays are commonly found in the northern countries such as Russia, Canada, Norway, Sweden and US state of Alaska, which were once glaciated during the Pleistocene epoch (Brand & Brenner, 1981). Various sources (e.g. Rankka et al., 2004) state the following reason for the sensitivity found in these clays. Sensitive clays are formed by deposition of sediments in a marine environment for a long time when it was covered by sea. In this environment the positive charge of cations (such as sodium) were able to bind clay particles with negative surface charge by balancing charges in layers. When the glaciers retreated, post-glacial rebound made the clay to expose and form the soil mass. They were no longer subjected to salt water conditions; rainwater infiltrated these clays and washed away the salts that allowed these clay particles to remain in a stable structure. Upon shear, the lack of counterbalancing charge from salts in the

sensitive clay results in clay particle repulsion and realignment of clay particles to a structure that is extremely weak and unstable.

In geotechnical engineering perspective, sensitive clays are categorized based on their sensitivity. A first sensitivity scale was proposed by Skempton et al. (1952), who classified clays as Insensitive, Low sensitive, Medium sensitive, Sensitive, Extra sensitive and Quick clays with increasing sensitivity index. This scale has been modified and/ or new scales have been proposed by various researchers (e.g. Rosenqvist, 1953; Rankka et al, 2004). For Canadian practice Canadian Foundation Engineering Manual CFEM (2006) proposed the following guidelines:

Table 2.1 Sensitivity Classifications in the CFEM (2006)

Sensitivity	**Definition**
< 2	Low sensitivity
2 - 4	Medium sensitivity
4 - 8	Extra (high) sensitivity
> 16	Quick

2.3 Monotonic shear behaviour of clay

Shear strength of clays are of paramount importance to engineers as they often turn out to be the critical design parameter. Due to their extremely low permeability, dissipation of excess pore water pressure is very slow in clays, and most loading conditions impose undrained deformation, at least in the short term. Shear strength (in short term) is considered most critical for stability in clays (e.g. Bishop & Bjerrum, 1960; Ladd & Foott, 1974; Craig, 2004), and when represented using total stress parameters, it is generally called the undrained cohesion, C_u. Skempton (1948) proposed that saturated

cohesive soils behave as frictionless materials with $\phi_u = 0$, and thus undrained shear strength $S_u = \frac{1}{2}(\sigma_1 - \sigma_3)$.

The monotonic undrained stress strain behaviour of clays has been studied extensively in the literature (e.g. Roscoe et al, 1958; Bishop, 1966; Ladd & Foott, 1974; Silvestri et al, 1989; Ladd, 1991). Behaviour of clays is elasto-plastic, even though the true elastic region is very small. When subjected to undrained shear, the shear stress in clays increases with strain and reaches an almost constant value. However, pore pressure increases steadily in NC clays showing contractive behaviour. But OC clays show a dilative response; pore pressure decreases following a small initial increase in undrained state. During drained conditions they show a peak in stress strain relationship and a dilative volume change. Fig 2.1 shows typical monotonic behaviour of clays.

2.3.1 Effect of effective vertical / confining stress

The true behaviour of clays, like any soil, is dependent on the effective stress state. The undrained strength of clay, when discussed within the effective stress framework is generally denoted by S_u. Undrained shear strength in clays is usually expressed as a function of effective vertical stress (σ'_{vc}) and for normally consolidated clays undrained strength generally increases linearly with σ'_v as shown in Fig. 2.2 (e.g. Ladd & Foott, 1974; Craig, 2004). Variation of S_u with σ'_v can be determined from Field Vane shear test conducted at different depths or from Consolidated undrained tests conducted at different consolidation pressures in clays (Bishop & Bjerrum, 1960; Ladd, 1991).

However, the relationship of S_u with σ'_v is generally non linear for clays in over consolidated state, as shown in Fig 2.2. Ladd and Foot (1974) proposed "Stress History

and Normalized Soil Engineering Properties (SHANSEP)" method. They consolidated clay well above the pre–consolidation pressure (σ'_{pc}) and reduced consolidation stress to different values in order to determine undrained strength at different OCR values. Ladd (1991) examined values of S_u/σ'_{vc} against different OCR values and proposed the following relationship in terms of OCR to determine undrained strength of clay.

$$S_u/\sigma'_{vc} = S(OCR)^m$$

where S is the normally consolidated value of S_u/σ'_{vc} and converges as a linear relationship for normally consolidated soils.

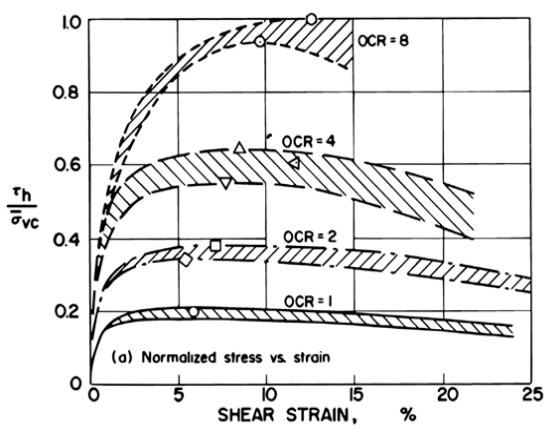

Fig 2.1 Typical monotonic response of clay (after Ladd & Foot, 1974)

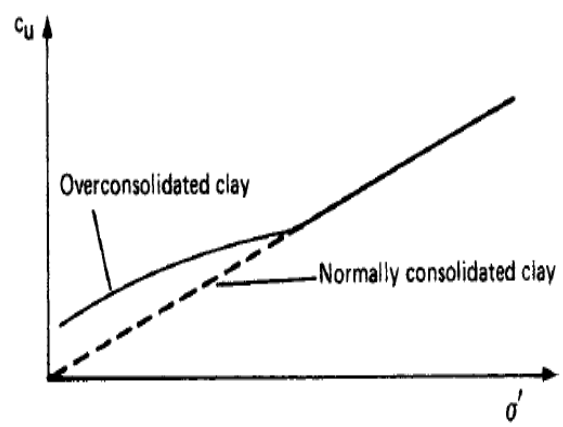

Fig 2.2 Variation of undrained strength with consolidation pressure (after Craig, 2004)

2.3.2 Effect of OCR and stress path

Shear characteristics of saturated clays is primarily a function of consolidation history or OCR and stress path. Initially, Roscoe et al (1958) found that there is a unique relationship between void ratio (e) and effective stress for normally consolidated (NC) clays and proposed "Critical State concept" for clayey soils which relates effective stress

parameter (p') with corresponding specific volume ($v = 1 + e$) of a normally consolidated clay during drained and undrained conditions. They also demonstrated the existence of a "characteristic surface" that limits all possible states of NC clays. All effective stress paths eventually reaches this surface; once they reach this surface clays yield continuously at a constant volume under constant effective stress. Based on the concept of Roscoe et al. (1958), a 'critical state line' as shown ($S - S$) in Fig 2.3 (a) can be presented in a chart with effective stress parameter (p'), shear stress parameter (q) and specific volume (v) or void ratio (e) as axis. The points on this line define combinations of p', q and v at which failure and subsequent yielding at constant volume take place in NC clays. Slightly over consolidated clays behave more or less similarly as NC clays. However, many researchers (e.g. Roscoe et al, 1958; Craig, 2004) indicate that behaviour and boundary states of heavily over consolidated clays can be demonstrated by 'Hvorslev surface' (Hvorslev, 1930) as in Fig 2.3 (b).

2.3.3 Strength anisotropy

The nature of deposition and natural consolidation process of cohesive soils make particles oriented horizontally and perpendicular to the direction of major principal stress. Resulting cross-anisotropic structure causes their strength to be loading direction dependent (Bishop, 1966; Soydemir, 1976; Silvestri et al, 1989; Alshawmar, 2014).

Initially, strength anisotropy in materials has been investigated by Casagrande & Carillo (1944), who proposed a relationship to determine directional variation of strength in anisotropic materials as $S_{u\alpha} = S_{uh} + (S_{uv} - S_{uh}) \sin^2 \alpha$. Here $S_{u\alpha}, S_{uh}$ and S_{uv} are undrained shear strength measured at an angle of α, in horizontal orientation ($\alpha = 0^0$) and in vertical orientation ($\alpha = 90^0$) respectively. Here α is the angle measured from bedding

plane (horizontal orientation). Similar correlations were proposed based on lab and field observations on clays by various researchers (e.g. Richardson et al, 1975).

Furthermore, strength anisotropy is the main reason for variations in strength measurements observed from different experimental devices, or different loading modes such as triaxial compression and extension (Bishop, 1966). Hansen and Gibson (1949) claimed that active and passive earth pressure conditions produces the highest and lowest undrained shear strengths respectively while all the other modes falls in between of them. Observations of Ladd (1991) in triaxial mode agree with that of Hansen and Gibson (1949). Ladd (1991) found that shear strength during triaxial compression mode, which represents active state, is the largest, triaxial extension mode which represents passive state shows lowest strength and simple shear or direct shear reported strength in between these values.

Soydemir (1976) and Silvestri et al (1989) showed that simple shear test can be used to investigate the compression and extension modes by cutting specimens at appropriate angles or orientations. Tests on Norwegian soft clays by Soydemir gave S_{uh} / S_{uv} values greater than one (i.e. $S_{uh} > S_{uv}$). However, tests on Canadian sensitive clays by Silvestri et al (1989) showed that shear strength at horizontal orientation (S_{uh}) is the weakest while shear strength at vertical orientation (S_{uv}) is the strongest when the soils are over consolidated/intact. On the other hand, they observed no strength anisotropy when the clay become normally consolidated and obtained a unique relationship between undrained strength and effective consolidation pressure disregarding orientation. Strength anisotropy was also observed under simple shear condition by Alshawmar (2014).

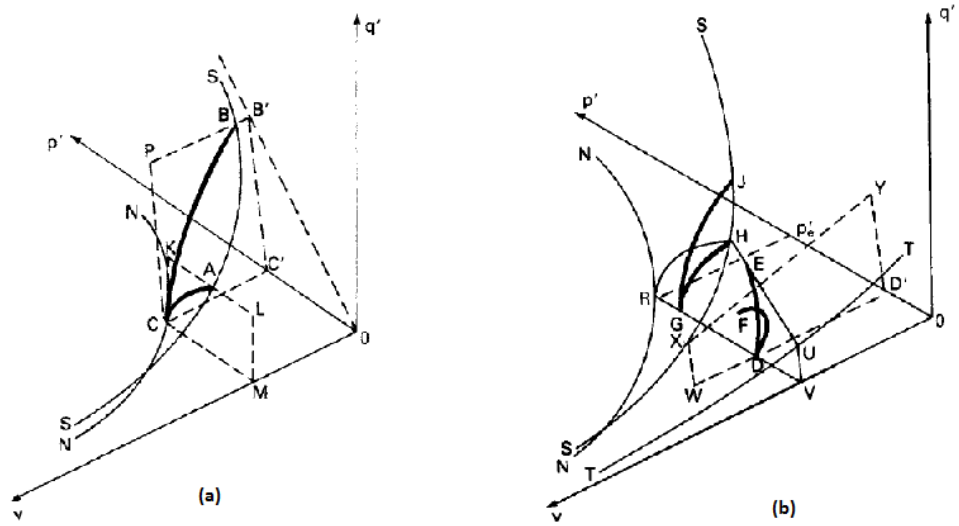

Fig 2.3 Critical state concepts for (a) NC clays (b) OC clays (after Craig, 2004)

2.3.4 Strain rate effects

Rate of strain influences undrained strength of clays considerably; undrained strength commonly increases with strain rate (e.g. Casagrande & Wilson, 1951; Ladd, 1991). Vaid (1979) and Lefebvre and LeBoeuf (1987) investigated various Eastern Canadian sensitive clays and reported an undrained strength increase of about 10% when the strain rate increased by a log cycle (i.e. 10 times). Lefebvre & LeBoeuf (1987) reported that strain rate effects on NC clays are mainly related to pore water pressure generation while failure envelope of them remain unchanged. On the other hand, lowering the strain rate lowers the strength envelope in OC clays without affecting the pore pressure generation significantly.

Ladd (1991) suggested an axial strain rate of 0.5% to 1% per hour for triaxial tests and a shear strain rate of 5% per hour for direct simple shear tests to measure undrained strength properties of clays in laboratory.

2.4 Cyclic shear behaviour of clay

2.4.1 Cyclic shear strength Clay

Soils are often vulnerable to undrained cyclic loadings such as earthquakes and wave loading. These cyclic loadings have a wide range of amplitude, frequency, and duration. Similarly, response of soils to cyclic loading also varies widely and depends on various factors. Therefore, cyclic behaviour of clay is a matter of interest in geotechnical engineering practice and has been studied extensively (e.g. Sangrey et al, 1969; Anderson et al, 1988; Malek et al, 1989; Zergoun & Vaid, 1994; Ishihara, 1996; Boulangar & Idriss, 2004).

Typically clayey soils show a progressive increase in cyclic strain and pore water pressure with loading cycles of significant cyclic stress amplitudes. This results in gradual loss of effective stress to certain extent as seen in Fig. 2.4 & Fig. 2.5; As a result they show certain strength loss or failure with increasing number of loading cycles. Sangrey et al (1969), Mitchell and King (1976), Azzouz et al (1989), Zergoun and Vaid (1994) and various others showed that a rapid increase in pore water pressure and a resulting effective stress loss occurs during the initial loading cycles and shear failure occurs if or when the effective stress path touches the monotonic effective stress failure envelope of that clay. However, complete loss of strength or true liquefaction such as in saturated loose sands is generally not a possibility (e.g. Ishihara, 1996; Perlea, 2000). But, clays can experience significant strength loss especially if they are sensitive (Ishihara, 1996; Boulangar & Idriss, 2004).

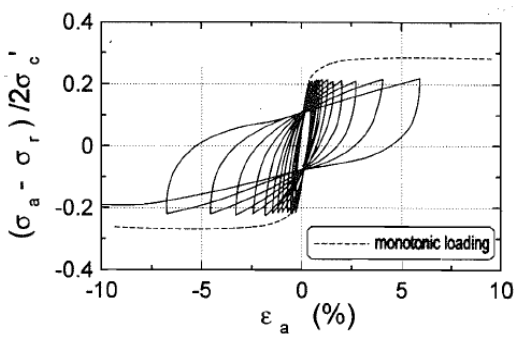

Fig 2.4 Typical Cyclic Stress Strain behaviour of clay (after Zergoun & Vaid, 1994)

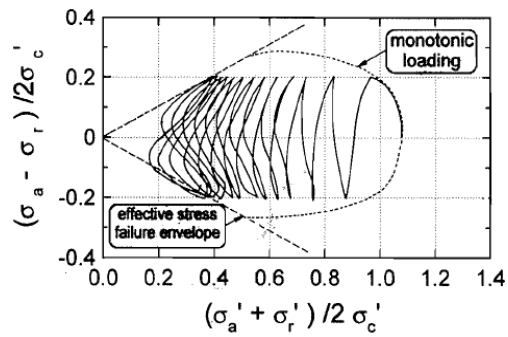

Fig 2.5 Typical stress path of clay during cyclic loading (after Zergoun & Vaid, 1994)

In addition, clays experience large cyclic shear strains (γ_c) as the number of cyclic loading increases, as seen in Fig. 2.4 due to significant reduction in the (secant) shear modulus or stiffness (Vucetic & Dobry, 1991). This cyclic strain is accompanied with or without increasing permanent or average shear strain (Kodaka et al, 2010; Azzouz et al, 1989). Large cyclic strains and accompanied strength loss is a matter of concern for engineers especially in earthquake response analysis. As a result, cyclic strength is usually defined in literature as the cyclic shear required inducing a significant shear strain under triaxial or simple shear conditions (e.g. Boulangar & Idriss, 2004). Furthermore, cyclic shear strength is generally expressed as a ratio (τ_{cyc}/S_u) which is a function of monotonic undrained shear strength of clay (S_u) (e.g. Andersen et al, 1988; Zergoun & Vaid, 1994; Lefebvre & Pfendler, 1996).

Cyclic strength of clays and their behaviour during cyclic loading depend on various factors. It depends on cyclic stress/strain amplitude, number of loading cycles, frequency/loading rate, effective vertical stress, consolidation path, over consolidation ratio (OCR), sensitivity, initial static/average shear etc.

2.4.1.1 Cyclic stress / strain amplitude

Cyclic shear stress (τ_{cyc}) amplitude is an important parameter in controlling the cyclic behaviour of soils. Even though cyclic response of soil depends on number of load cycles, frequency of loading etc, in some scenarios, especially in wave loading on foundation soils, the number of cycles is very large or infinite and frequency does not vary much. As a result, engineers are interested in determining the cyclic stress that is capable enough to fail the soil.

Sangrey et al (1969) investigated the effective stress response of moderately sensitive clays during undrained one way cyclic tests and concluded that a 'critical level of cyclic deviatoric stress' exists for clays irrespective of number of load cycles. They found that cyclic stress levels above this value are capable of producing substantial net pore pressure and a non recoverable strain with each loading cycle. As a result every cycle bring the stress path up to static failure envelope and eventually a shear failure may result. On the other hand, cyclic stress levels below this critical value may reach non failure equilibrium with closed stress strain loops. This critical cyclic stress level has been commonly termed as "threshold cyclic stress" in literature.

Mitchell and King (1976) performed repeated loading triaxial (without principal stress rotation) tests on sensitive Champlain Sea clays and observed effective stress failure, only when cyclic stress amplitude exceeds 50% of their monotonic strength (S_u). On the other hand, cyclic stress amplitudes less than 50% of S_u values do not create failure or excessive deformation. Similarly Raymond et al. (1979) and Zergoun and Vaid (1994) reported threshold strength values of about 55% of S_u for Canadian sensitive clays. Citing large variations in loading frequency or rate between monotonic and cyclic tests,

Lefebvre et al (1989) argues that comparisons of strength must be made at equal loading rate; they reported threshold cyclic strength of 60% for Eastern Canadian clays, based on cyclic tests conducted at a frequency value 'equivalent' to that of monotonic tests.

Houston and Herrmann (1980) conducted one way triaxial tests on various marine clays and reported threshold cyclic stress values from 18% to 90% of their static undrained strength. They concluded that the stability threshold is higher for clays with higher plasticity and decreases with decreasing Plasticity Index (PI) values.

Furthermore, Zergoun and Vaid (1994) showed that any increase in cyclic stress level in the middle of a cyclic loading would transfer the response of clay to the behaviour similar to the behaviour under the new cyclic stress amplitude. This means that cyclic response of clay does not depend on any previous loading history except accumulated permanent or average strains and generated pore water pressure during previous cyclic loading.

Along similar lines of threshold strength, some researchers (e.g. Vucetic, 1994) attempted to find a 'threshold volumetric strain (γ_{tv})', which is the cyclic strain amplitude beyond which significant permanent pore pressure or volume change develops and irrecoverable change of soil structure occurs. At the same time, cyclic strains below this threshold do not affect pore pressure or soil structure significantly even after large number of load cycles.

Macky & Saada (1984) performed cyclic triaxial torsional hollow cylinder tests on clays and found a threshold strain of 0.04%. Vucetic (1994) analysed experimental data from the literature and found that cyclic γ_{tv} generally increases with decreasing particle size and increasing Plasticity Index (PI) of soil.

Hsu and Vucetic (2006) tested a variety of soils and concluded cyclic γ_{tv} is higher in cohesive soils than that of cohesionless soils and γ_{tv} of clays seemed independent of effective vertical stress and test type. Rasmuseen (2012) reported that γ_{tv} in sensitive Leda clays resemble with the findings of Hsu and Vucetic (2006).

2.4.1.2 Number of load cycles

Number of load cycles (N) is important in the analysis of cyclic loading especially in short duration events such as earthquakes. Complex cyclic loading events such as earthquakes are simplified as certain number of 'equivalent' sinusoidal cyclic loads with a certain τ_{cyc} magnitude based on the intensity of the earthquake (Seed & Idriss, 1982). Moreover, design parameters such as cyclic stress ratio (CSR) are considered for a specific equivalent number of cycles for a loading event during soil dynamics analyses (Kramer, 1996). Hence, engineers are interested in cyclic resistance or strength of soils during a certain number of load cycles.

From the common understanding in literature (e.g. Ishihara, 1996; Vucetic & Dobry, 1991), it is obvious that "strength" of clays reduces with number of cycles with a certain stress amplitude as shown in Fig 2.6. Conversely, increasing pore water pressure build up results in increasing cyclic strain with N as shown in Fig 2.4.

Various researchers (e.g. Ansal & Erkin, 1989; Kodaka et al, 2010; Wichtmann et al, 2013) concluded cyclic shear stress amplitude shows a hyperbolic variation with number of load cycles. Also, there are correlations available in literature to find cyclic strength as a function of N (e.g. Ansal & Erkin, 1989).

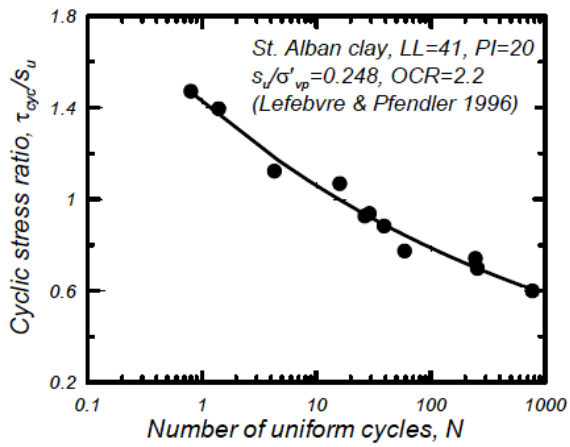

Fig 2.6 Typical variation of cyclic stress ratio of clays with number of load cycles (after Lefebvre & Pfendler, 1996 adopted from Boulanger & Idriss, 2004)

2.4.1.3 Loading frequency

Similar to the strain rate effects on the undrained monotonic strength of clays, loading frequency or rate also influences cyclic strength of clays (e.g. Lefebvre & LeBoeuf, 1987), even though this effect is not as significant in granular soils (e.g. Peacock & Bolton, 1968). A number of investigations on the effect of loading frequency on cyclic strength of clays show cyclic strength of clays increases significantly with loading frequency as shown in Fig 2.7 (Lefebvre et al, 1989; Anderson, 2009; Wichtmann et al, 2013).

Lefebvre et al (1989) reported 30% cyclic strength increase when loading frequencies have been raised through two log cycles (i.e. increased by about 100 times). Ansal and Erken (1989) reports that though cyclic strength increases with increasing frequency, the effect of frequency on strength diminishes when number of load cycles are higher.

Lefebvre and Pfendler (1996) noted that shear stresses in excess of about 40% of their undrained strength have been resisted for more than a cycle during undrained cyclic simple shear tests due to effects of very high loading frequency. They also found that

very high loading rates in cyclic loading partially compensate the strength degradation with number of loading cycles.

Lefebvre and LeBoeuf (1987) found loading frequency alters the strength envelope in structured or OC clays, but not influences the pore water pressure generation much. As a result failure occurs below the peak strength envelope in OC clays under slow loading rate. On the other hand in structured or NC clays, changes in loading frequency affects pore pressure generation. As a result, stress path is altered, but stress path meets a unique strength envelope at failure.

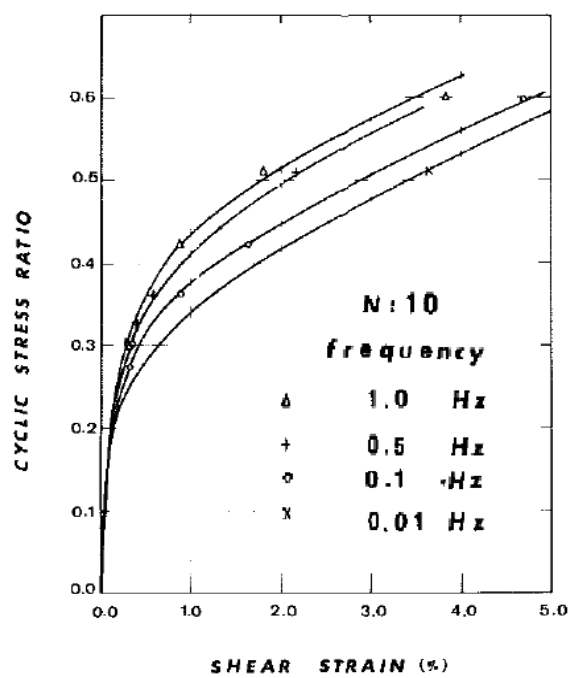

Fig 2.7 Effect of Cyclic Frequency on Cyclic Stress Ratio (after Ansal & Erken, 1989)

2.4.1.4 Effective vertical stress

Effective vertical stress (σ'_{vc}) is another factor that influences the cyclic behaviour of soils. For example cyclic stress ratio or liquefaction resistance of sand has a correction factor for effective overburden stress (e.g. Seed, 1983). However, Boulanger & Idriss

(2004) reports that the effect of σ'_{vc} is not as important in the cyclic behaviour of clays, as they have already been considered in their monotonic undrained strength (S_u), as cyclic strength of clays is usually expressed in terms of S_u. Few investigations have been found in literature on the effect of confining stress on cyclic behaviour of clay. Mitchell and King (1976) observed that pore pressure generation during cyclic loading increased with effective confining stress, and number of load cycles necessary for failure reached a minimum between effective confining stresses of 50 and 100 kPa for Champlain Sea clay in Eastern Canada.

Bray and Sancio (2006), observed severe strength loss/liquefaction in clayey soils in Adapazari, Turkey during the 1999 Kocaeli earthquake at locations near or close to buildings than open sites. Based on this observation they concluded that clays at low confining stress are more resistant to cyclic loading as in the case of cohesionless soils.

2.4.1.5 Consolidation method

Effects of consolidation method on cyclic behaviour of clays have also been investigated in literature. Lee (1979) reported sensitive clay specimens consolidated anisotropically showed higher cyclic strength than that of isotropically consolidated specimens. However, Ansal and Erken (1989) observed more shear resistance in isotropically consolidated specimens under simple shear conditions. It was reasoned that isotropic consolidation creates significant particle rotation/rearrangement that may result in more resistance in simple shear loading mode. Kodaka et al (2010) investigated effects of consolidation process using a specially built simple shear device that is capable of enforcing isotropic and anisotropic consolidation conditions. They concluded that cyclic strength of clay can be expressed in terms of σ'_{vc} disregarding consolidation method.

2.4.1.6 Over consolidation ratio (OCR)

Over consolidation ratio (OCR) seems to have similar effects on cyclic stress strain behaviour of clays, as it does in monotonic behaviour. For example, Azzouz et al (1989) reported that NC clays produced positive pore water pressure and shifted the stress path gradually towards origin during cyclic shearing. On the other hand, OC clays developed negative pore water pressure at the initial stages. Subsequently they generated positive pore pressure with increasing loading cycles until failure. Also, magnitude of initial negative pore pressure increases with OCR as in monotonic loading.

Different observations have been reported in literature regarding the effect of OCR on cyclic strength. Azzouz et al (1989) claimed that OC clays showed equal or stronger behaviour than that of NC clays under equal cyclic stress amplitudes. However, Anderson et al (1988) found number of cycles to failure is little bit lower in OC clays than that of NC clays.

2.4.1.7 Sensitivity

Various investigators (e.g. Mithcell & King, 1976; Lee, 1979) consider the possibility of disturbance of in situ structure or remoulding of clays as a result of cyclic loading. For example, Mitchell and King (1976) showed when sensitive Leda clays undergo a cyclic shear amplitude of at least 50% of their undrained strength, their in-situ structure is disturbed significantly. This eventually leads to an effective stress failure due to excess pore water pressure generation. Furthermore, mechanical remoulding also results in significant cyclic strength loss in clays due to their sensitivity as in the case of monotonic strength (e.g. Kodaka et al, 2010).

2.4.1.8 Initial static shear stress

Initial Static shear is another parameter that has been identified for its effects on cyclic behaviour of soils. Static shear is present and a concern for engineers in scenarios where they investigate stability of slopes under cyclic loading or wave loading on foundation soils. Effects of initial static shear on cyclic behaviour of sand has been investigated extensively over the years (e.g. Sivathayalan & Ha, 2011) and it was noted that initial shear increases or decreases cyclic shear resistance of sands depending on its density. However, an initial static shear tends to decrease cyclic strength in clays (e.g. Seed & Chan, 1966; Malek et al, 1989). According to Lefebvre and Pfendler (1996) investigation of static shear stress effect in clays is not easy, as the application of static shear itself depends and/or alters various factors such as strain rate, OCR etc. Thus, available data on the effects of initial static shear on clays is limited.

Malek et al. (1989) studied the effects of initial shear under simple shear conditions on NC Boston blue clay. They observed that initial static shear creates a bias in cyclic loading, which in turn gradually accumulates an average shear strain accompanied by relatively small cyclic strain with number of load cycles. Similarly, from their triaxial investigation on Norwegian clays, Wichtmann et al. (2013) concluded that clay subjected to intermediate initial shear (about 30% S_u) failed due to large cyclic strain amplitudes. However, clay with very small or no initial shear failed due to accumulation of permanent axial strains.

Furthermore, Malek et al (1989) found that cyclic shear have more impact on failure of clay than that of static average shear. An increase in cyclic stress amplitude reduces cyclic strength much more than that of a reduction due to an equal initial shear increase.

Lefebvre & Pfendler (1996) conducted simple shear tests on soft sensitive clays. They observed presence of initial shear stress reduces strength degradation rate (i.e. strength degradation with number of cycles), even though it decreases cyclic resistance of clay. It was also noticed that higher initial shear values reduces stress reversal which in turn reduces cyclic strain amplitudes.

2.4.1.9 Principal stress rotation

Direction and rotation of principal stresses also influences the cyclic behaviour of soil. Moreover, it is well known that cyclic loading itself alters principal stress directions. Seed & Chan (1966) and Yasuhara et al (1992) showed that two way cyclic loading under triaxial conditions in which principal stresses rotate has more damaging effects in clays. Significant cyclic strength reduction was observed when principal stresses rotated during loading. A number of studies have been carried out about the influence of principal stress rotation on behaviour of sand using triaxial and hollow cylinder devices (e.g. Vipulanantham, 2011; Sivathayalan et al, 2014) However, similar investigations in clays are limited in literature (Talesnick et al, 1991).

2.4.2 Post cyclic behaviour of clay

The post cyclic shear strength or behaviour of liquefied soil is a concern in modern engineering practice. Such knowledge is required to determine how much strength the soil can regain after a possible cyclic event. Importantly it is a concern for sensitive clays, as there are possibilities of severe strength loss due to remoulding during cyclic loading.

Observations from a wide variety of clays in literature show that clays regain significant undrained strength (about at least 80%) after a cyclic loading of magnitudes less than a critical threshold value. Researchers argue that cyclic stresses below the threshold value

does not change the structure of clay or produce significant pore pressure; as a result post cyclic strength has not been affected significantly. However, when clay experience large cyclic stress amplitudes or large cyclic strains capable enough to disturb their structure and produce significant pore pressure, clays show considerable strength loss following the cyclic loading (Lee, 1979; Castro & Christian, 1976; Lefebvre et al, 1989; Yasuhara et al, 1992).

Different schools of thoughts exist in literature regarding post–cyclic shear behaviour of clays. Various researchers (e.g. Anderson et al, 1980) investigated the stress–strain behaviour of clays following cyclic loading. They found that there is significant stiffness degradation during initial post cyclic behaviour at small strains. Similar stiffness reductions have been observed commonly in sand as well (e.g. Vaid & Thomas, 1995).

Some researchers (e.g. Yasuhara et al, 1992; Lefebvre et al, 1989; Azzouz et al, 1989) explain post cyclic stress–strain behaviour of clays based on the "apparent over consolidation" principle. They showed that undrained cyclic shearing in NC clays caused an "apparent over consolidation" as this shearing reduces the effective vertical stress. As a result, a NC clay that has undergone cyclic shearing will behave similar to a mechanically OC clay during an undrained monotonic shearing as shown in Fig 2.8. However, Anderson et al (1980) and Azzouz et al (1989) observed stiffness degradation caused by cyclic loading creates a positive pore water pressure during the initial stages, instead of a negative pore water pressure as in the case of mechanically OC clays. As a result clay shows a different behaviour at very small strain ranges.

Ho et al (2013) conducted slow cyclic triaxial tests on Singapore Marine clays in order to obtain reliable pore pressure measurements. Their investigation showed that post cyclic

behaviour of this clay depends on the pore pressure developed during cyclic loading and it is independent of effective consolidation pressure, cyclic strain amplitude and number of load cycles. They observed OC type stress path when pore pressure developed during cyclic shearing was more than 50% of initial confining stress and a NC or slightly over consolidated type behaviour in situations where pore pressure developed is lower during cyclic loading.

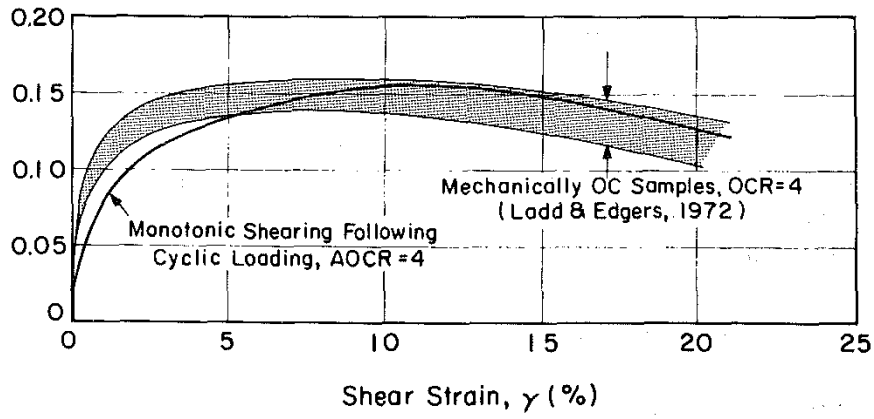

Fig 2.8 Post cyclic and pre cyclic undrained behaviour of clay (after Azzouz et al 1989)

Yasuhara and Anderson (1991) studied the effects of drainage after a cyclic loading, on a later cyclic loading event on NC and OC Drammen clay. They concluded that cyclic loading accompanied by a subsequent drainage makes NC clay more resistant and OC clay less resistant to a later cyclic loading. Moreover, Yasuhara et al (1992) & Yasuhara (1994) observed an undrained strength even more than their pre-cyclic strength in NC clays similar to secondary compression of clay, when drainage of generated excess pore pressure is allowed after cyclic loading.

2.4.3 Liquefaction potential in clays

Liquefaction is a phenomenon that occurs due to an increase in pore water pressure commonly during undrained cyclic loading. This pore pressure build up results in the loss

of effective stress in soils. As a result, contacts between particles are released and soil starts to behave like a ‘liquid’; severe strength loss and large permanent strains occur. Cohesionless soils such as loose sands encounter the danger of complete strength loss, as their shear strength depends solely on effective stress.

However, “Liquefaction” is a complex phenomenon that differs from one soil to another and the term "liquefaction" has been used to denote different phenomena in the literature. Seed et al (2003) and Boulanger and Idriss (2006) suggest the term “Liquefaction / Classic Liquefaction” is more appropriate to describe significant strength loss and excessive strains to soils that behave similar to sands, while a term such as “cyclic softening/sensitivity (strength loss)” could be used to describe the behaviour of clays or soils that show clay like behaviour.

2.4.3.1 ‘Liquefaction’ case histories in clays

Liquefaction observations are not very common in cohesive soils or clays as they are in sands, and clays are commonly considered ‘non liquefiable’ (Seed et al 1983) from the classical definition of the term. Furthermore, cohesive soils do not lose their strength completely as in loose sands during cyclic loading, because the presence of cohesion in these soils prevents complete separation of particles (Ishihara, 1996). However, ‘liquefaction’ is currently often used to denote significant strength loss and softening, and thus occurrences of liquefaction have been reported in cohesive soils in literature in that sense.

Initial evidence for liquefaction in cohesive soils has been reported by Wang (1979) due to some strong earthquakes that occurred in 1960s and 70s in China. Tuttle et al. (1990) examined ‘silty boils’ present during the Saguenay earthquake of 1988 in Canada and

found that clay content present in the boil was less than 10% while clayey silt below a depth of 10m with higher clay content did not present in these boils.

However, observations by Youd et al (1985) at Whisky Springs site during Borah Peak Earthquake, Boulanger et al (1998) at Moss Landing during the 1989 Loma Prieta earthquake and Bray et al (2004 (a), 2004 (b)) in Adapazari, Turkey during the 1999 Kocaeli earthquake showed cohesive soils with > 20% clay sized particles liquefied. Moreover, failure of sensitive clays due to strength loss have been reported by Olsen (1989) and Stark and Conteras (1998) during the 1964 Alaska Earthquake.

2.4.3.2 Liquefaction criteria for clayey soils

Liquefaction potential of sands has been studied extensively. As a result, broadly accepted and well-defined liquefaction criteria, such as the NCEER guidelines by Youd et al. (2001), are available for sands. Most of these criteria are based on field tests parameters such as SPT blow count, since sampling and testing of undisturbed sand specimen is not easier in practice. Comparatively, lesser research has been done in the liquefaction potential of clays and other cohesive soils. And available criteria are mainly based on moisture content, Atterberg limits and finer particle percentage of clays. Some researchers (e.g. Ishihara, 1996; Perlea, 2000) justify this as strength against liquefaction in cohesive soils depends on friction and bonds between individual particles due to cementation which is best represented by plasticity of clays.

A first well documented criterion for liquefaction in cohesive soils was proposed by Seed et al (1983) based on the observations made by Wang (1979). This is often referred as the "Chinese criteria" for liquefaction of cohesive soils and specifies that clayey soils

vulnerable for severe strength loss or liquefaction must meet all of the following characteristics:

Percent finer than 0.005 mm	< 15%
Liquid Limit	< 35
Water Content	> 0.9 x Liquid Limit

Some modifications to this criterion have been proposed for practice in North America by various researchers (e.g. Andrews & Martin, 2000; Finn et al, 1991) citing the discrepancy between Chinese practice and USA practice in clay size particle limit and determination of Liquid limit (Koester, 1992).

Later, many researchers (e.g. Seed et al, 2003; Bray et al, 2004 (a), 2004 (b); Boulanger & Idriss, 2006) found that the "percent clay finer" rule in Chinese criteria and other similar criterion is not appropriate, based on observations at various sites where soils with even higher clay content liquefied (e.g. Boulanger et al, 1998; Bray et al, 2004 (a), 2004 (b)). It was understood that what matters in liquefaction potential of cohesive soils is not the amount of clay size particles as proposed by "Chinese criteria" or other similar criterions, but the type of clay mineral present in the soil that influences the behaviour.

Based on this thought, Seed et al (2003) proposed a specification, based on the behaviour of soils, whether they are "Sand like" or "Clay like" or intermediate. Soils are divided into 3 zones as shown in Fig. 2.9 and have been defined as follows:

- Zone A: Soils with PI ≤ 12, LL ≤ 37 and water content > 0.8 LL susceptible to "Classic cyclically induced liquefaction" where significant strength loss occurs
- Zone B: Soils with 12 < PI ≤ 20, 37 < LL ≤ 47 and water content > 0.85 LL in which liquefaction potential must be determined by Laboratory tests.

- Zone C: Soils with PI >20, LL > 47, which are usually not susceptible to "classic liquefaction", however can undergo "Sensitivity" or "Strength loss".

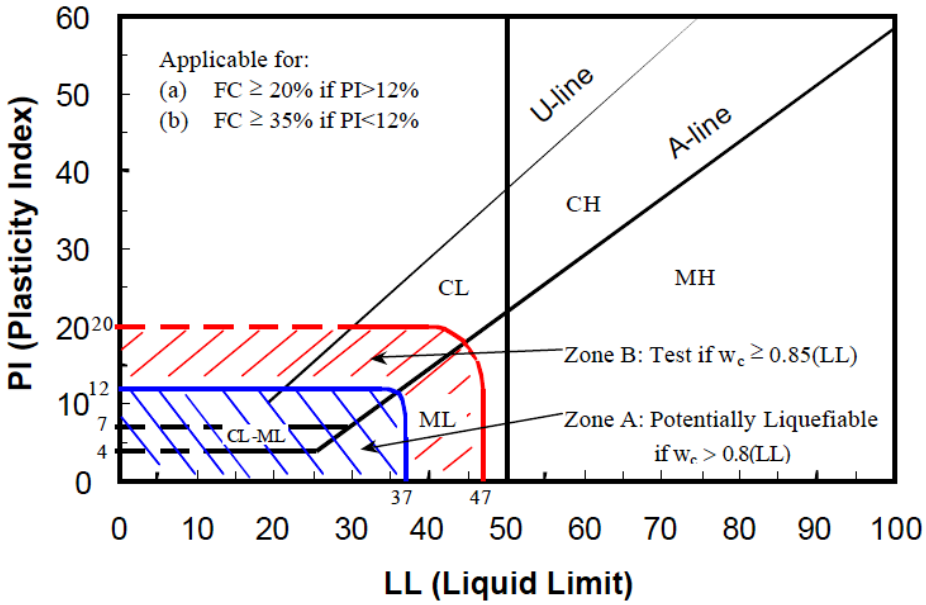

Fig 2.9 Specifications for assessment of "Liquefiable" Soil Types (after Seed et al, 2003)

Similar classifications have been proposed by other researchers as well (Bray & Sancio, 2006). Youd and Gilstrap (1999) suggested that sensitive clays classified as 'ML or CL' and with sensitivity index greater than four are vulnerable to severe strength loss during cyclic loading, if they have a liquidity index of at least 0.6 and a moisture content of at least 90% of LL.

On the other hand, most of these specifications were derived from field observations, thus questions arise whether they are applicable under all circumstances. Therefore, laboratory testing of high quality field clay samples is considered as the best means of characterizing liquefaction or potential strength loss in clayey soils (Seed et al, 2003; Boulanger & Idriss, 2004).

2.5 Modulus reduction and damping

One of the important and challenging tasks for geotechnical engineers is to evaluate the ground response due to a cyclic loading event such as an earthquake. "Ground response Analyses" are used to predict surface motions in order to produce design spectra, to determine liquefaction potential and to check the stability of various structures during cyclic loading events. In order to perform a ground response analysis, it is necessary to input the ground motion parameters and model the soil accurately. Modeling of soil in this analysis requires the challenging task of characterizing cyclic soil behaviour as accurately as possible using simple models.

There are many models such as equivalent linear model, cyclic non linear models and advanced constitutive models that can be used to represent cyclic stress strain behaviour of soils. Among these "equivalent linear model" is considered the simplest. It is commonly used in practice for earthquake response analysis (Kramer, 1996).

2.5.1 Equivalent linear model

Typical soil subjected to a single cyclic load would show a symmetric hysteresis loop of stress–strain behaviour as shown in Fig. 2.10. Two important parameters that govern the characteristics of this loop or cyclic soil behaviour are slope of the loop and its breadth. The slope or inclination of the loop depends on soil stiffness; it can be defined at any point of loop using tangent shear modulus (G_{tan}) or simply an average shear modulus called "Secant shear modulus (G_s)" as shown in Fig 2.10. Secant shear modulus (G_s) is the ratio between cyclic shear stress (τ_c) and respective cyclic strain (γ_c).

$$G_s = \frac{\tau_c}{\gamma_c}$$

The breadth depends on area of the loop, which is the measure of total energy released. Energy released is commonly represented by viscous damping ratio (ξ) (Jacobsen (1930) as per Kramer, 1996),

$$\xi = \frac{W_D}{4\pi\, W_s} = \frac{1}{2\pi}\,\frac{A_{loop}}{G_s\,{\gamma_c}^2}$$

where W_D is the dissipated energy, W_s is the maximum strain energy and A_{loop} is the area of hysteresis loop. Characterizing soil specific G_s and ξ is considerable interest in soil dynamics. These parameters are measured at very small strain amplitudes using equipments such as resonant column (e.g. Woods, 1994; Kim and Novak, 1981) and using cyclic triaxial or simple shear tests at large strain amplitudes (e.g. Vucetic & Dobry, 1988, 1991).

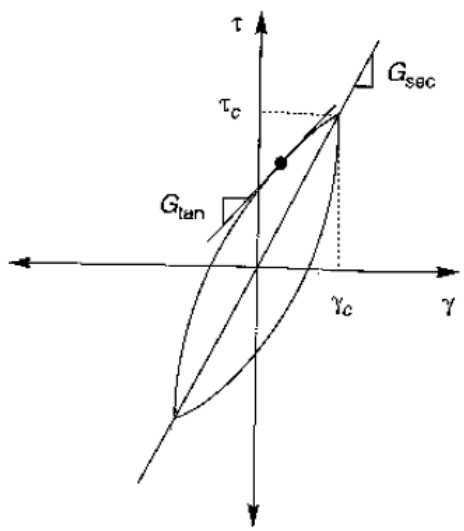

Fig 2.10 Secant shear modulus and Tangent Shear Modulus (after Kramer, 1996)

Modulus and Damping of a specific soil element varies with cyclic strain amplitude and they are given as a function of cyclic strain amplitude (γ_c). Usually secant shear modulus takes the maximum value (G_{max}) at very small strain amplitudes and reduces with

increasing strain. If a curve connecting the tips of hysteresis loops at various strain values is drawn, it will take a shape as shown in Fig 2.11(a). This curve is commonly called as "Backbone curve" and G_{max} can be represented as the tangent drawn at origin. It is also possible to represent this backbone curve as a "modulus reduction curve" drawn between $G/_{G_{max}}$ and γ_c as shown in Fig 2.11(b).

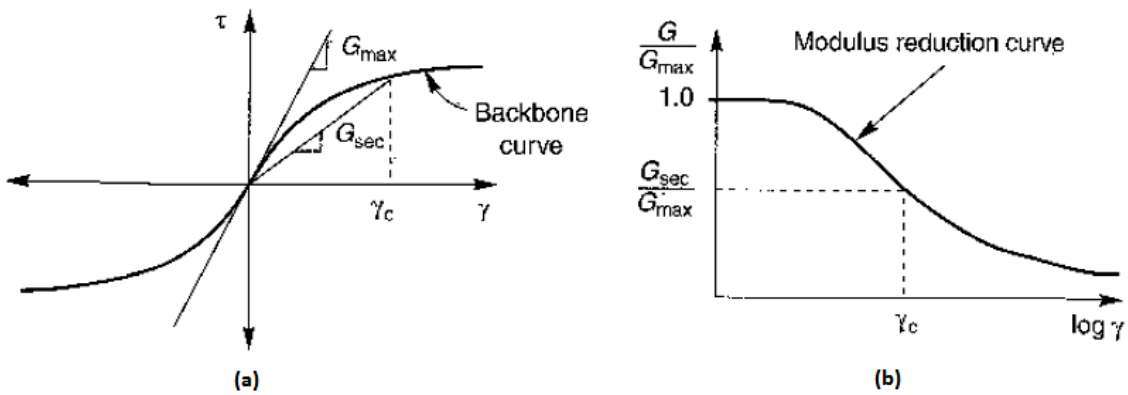

Fig 2.11(a) Backbone curve and (b) Modulus reduction curve of soil (after Kramer, 1996)

The breadth of hysteresis loops increase with cyclic strain amplitude (γ_c), which indicates that Damping ratio (ξ) increases with increasing cyclic strain amplitude (γ_c).

2.5.2 Maximum shear modulus (G_{max})

Maximum shear modulus (G_{max}) usually obtained at very small strain amplitudes and can be conveniently obtained from shear velocity of the soil (v_s) using, $G_{max} = \rho\, {v_s}^2$, where ρ is the density of soil. Geophysical methods are being used commonly at sites to measure shear wave velocity of soil (e.g. Yan & Byrne, 1990). Experimental methods such as resonant column or bender elements can be used to obtain G_{max} in the laboratory at very small strain amplitudes (Kim & Novak, 1981; Kramer, 1996).

In addition, there are number of correlation in the literature that can be used to determine G_{max}. Hardin (1978) proposed a correlation for G_{max} of soils in terms of PI, OCR, void ratio (e) and effective mean confining stress (σ'_m). According to this correlation G_{max} increases with OCR and with σ'_m and decreases with e. Similarly, Okur and Ansal (2007), and Jung et al. (2013) showed the maximum shear modulus depends mainly on confining stress level and OCR. Kim and Novak (1981) and Kokusho et al (1982) found that shear modulus is a time dependent property and increases with time in different amounts based on soil type and PI.

2.5.3 Modulus and damping

Shear modulus and Damping of soil are expressed as a function of cyclic strain amplitude and depend on various parameters such as effective confining stress, Plasticity index (PI), OCR/Stress history, number of load cycles, loading rate etc. Various researchers investigated modulus reduction and damping in various soils and the influence of the above mentioned factors.

Seed & Idriss (1970) originally provided modulus and damping curves as a function of cyclic strain amplitude for coarse and fine grained soils such as sands and gravels. Seed et al. (1986) analysed the available modulus data for sand and concluded that these curves mainly depend on effective confining pressure (σ'_m) and relative density (Dr) or void ratio (e). Furthermore they found modulus reduction and damping curves for sands falls within a narrow band and no other factors significantly influence these curves. As a result, Seed et al (1986) proposed an average modulus reduction curve and limiting curves based on relative density that can be used in practice for cohesionless sands.

2.5.3.1 Influence of Plasticity Index (PI)

Later, investigations on clay by various researchers demonstrated that mineralogy of clay, which is closely correlated to the Plasticity Index (PI) is the main controlling factor of modulus and damping curves over the other factors.

Kokusho et al (1982) from their experimental study on soft clays initially noted that modulus reduction and damping curves are highly influenced by Plasticity index (PI) values of clays. They also found that the G/G_{max} Vs γ_c curve locates higher and ξ Vs γ_c curve locates lower with increasing plasticity in clays. Moreover, the rate of degradation of shear modulus with cyclic strain amplitude is lower in high plasticity clays, than that of clays with lower plasticity. Later, reviewing a large number of experimental data Dobry and Vucetic (1987), and Sun et al (1988) also agreed with this observation.

Vucetic and Dobry (1991) examined experimental data in the literature from various devices such as resonant column, simple shear and triaxial and from diverse soil types with a range of OCR values. They confirmed that modulus reduction curve moves up and right, damping curve tends to move down with increasing Plasticity index (PI). Also, Vucetic and Dobry (1991), and Okur and Ansal (2007) preferred to present modulus reduction and damping curves for clay in terms of PI as it is independent of stress history or OCR and determining PI of clay is relatively easier in practice. As a result, Vucetic and Dobry (1991) presented modulus reduction and damping curves for clays for soil dynamics practice with respect to their PI values as shown in Fig. 2.12 & 2.13. These curves are commonly used for site response analyses for clay in geotechnical practice.

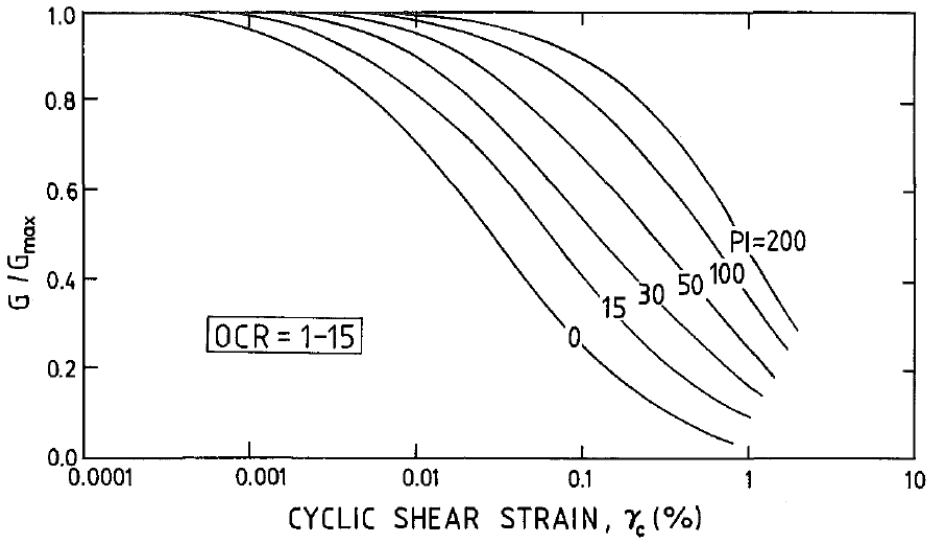

Fig 2.12 Modulus Reduction curves for clays with different plasticity (after Vucetic & Dobry, 1991)

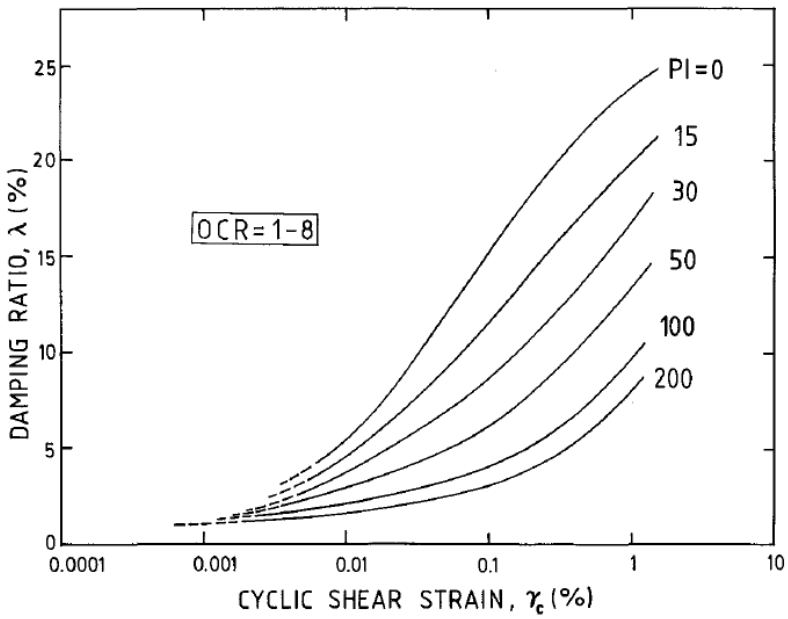

Fig 2.13 Damping curves for clays with different plasticity (after Vucetic & Dobry, 1991)

Modulus reduction behaviour of clays was also explained using a parameter 'linear cyclic threshold shear strain (γ_{tl})' by some researchers (e.g. Vucetic, 1994; Okur & Ansal, 2007). This essentially means a threshold cyclic strain below which soil behaves as a "perfectly elastic material"; as a result there is no Modulus reduction or significant damping increase below this strain value. Understandably, Vucetic (1994) found that this

threshold strain value increases with PI of clay which satisfies the previous observations in literature.

Vucetic and Dobry (1991) noted some considerable scatter or no clear trend in damping values at very small strain ranges disregarding PI values. Vucetic et al (1998) carried out an extensive investigation on damping behaviour at small strain level in various soil types using Double Specimen Direct Simple Shear device (DSDSS). They observed two distinctive behaviour patterns at medium / large (>0.01%) and very small (<0.005%) cyclic strain regions. While the damping behaviour of clays above γ_c of 0.01% agreed with the observations in literature, damping below 0.005% of γ_c showed a quite opposite trend; damping of clays found to be larger than that of sands and they are independent of PI values. Authors argued that at very small strain amplitudes clays are more linear, thus their behaviour is more influenced by viscous creep which gives roundness to the tips of hysteresis loop. It results in an increased area of the loop and provides larger damping.

At the same time Vucetic and Dobry (1991) noted and cautioned that modulus reduction and damping curves of very sensitive or quick clays could be significantly different from that of less/non sensitive clay with similar PI values. Rasmuseen (2012) compared some modulus values of sensitive Leda clays with that of Vucetic and Dobry (1991) curves and observed some deviation in behaviour of sensitive clay from that of non sensitive clays.

2.5.3.2 Influence of other factors

The influence of various other factors mentioned previously has been studied by many authors. Kokusho et al (1982), Vucetic and Dobry (1991) and Ishibashi (1992) showed that differences in consolidation history, effective confining stress, testing method and OCR do not have significant impact on the modulus reduction and damping curves of

clays. Vucetic et al. (1998) showed that even though damping decreases with OCR and effective confining pressure, effect of these parameters diminishes with increasing PI values and negligible in highly plastic clays.

Rasmuseen (2012) observed an increase in modulus values with loading frequency from tests on Leda clay, but they are not significant. Shibuya et al (1995) investigated the influence of loading frequency on modulus and damping at very small strains using cyclic torsional shear tests. They concluded that modulus values of soil are not sensitive to frequency variations; however, damping increases with decreasing frequency. However, Vucetic et al. (1998), found damping is independent of changes in loading frequency from tests conducted at very small strain amplitudes. Furthermore, Macky and Saada (1984) conducted torsional shear tests on clays and concluded modulus reduction and damping curves are sensitive to principal stress direction as well.

Ishibashi and Zhang (1993) analyzed the available experimental data on modulus reduction and damping of various soils ranging from non-plastic sands to highly plastic clays. They proposed simple formulae for modulus reduction and damping of soils in terms of G_{max}, γ_c, $\sigma_m{}'$ and PI of soils. Observations in literature suggest Plasticity index is the most influential factor over the others on modulus reduction and damping curves of clays. Vucetic and Dobry (1991) summarized influence of various factors from literature as shown in Table 2.2.

2.5.4 Stiffness degradation with number of cycles

Increasing number of load cycles disturbs in situ clay structure and produce pore water pressure. As a result, cyclic loading continues to degrade stiffness of soil causing the modulus reduction curve to move down with number of load cycles. However, number of

load cycles does not have any significant effect on damping (Macky & Saada, 1984; Vucetic & Dobry, 1991).

Idriss et al (1978) performed cyclic triaxial tests on normally consolidated soft clays and found that an irrecoverable degradation of stiffness occurs with load cycles and rate of this degradation depends on cyclic strain amplitude. Idriss et al introduced degradation index as 'δ' ($\delta = \frac{G_{sN}}{G_{s1}} = \frac{\tau_{cN}/\gamma_{cN}}{\tau_{c1}/\gamma_{c1}}$), and found δ varies linearly in a logarithmic plot for normally consolidated clays.

Table 2.2 Effect of various factors on G_{max}, G/G_{max}, and damping ratio of clays (modified after Vucetic & Dobry (1991))

Increasing Factor	**G_{max}**	**G / G_{max}**	**Damping ratio (ξ)**
Confining pressure (σ'_m)	Increases with (σ'_m)	Stays constant or increases with (σ'_m)	Stays constant or decreases with (σ'_m)
		(effect diminishes with increasing PI)	
Void ratio (e)	Decreases with e	Increases with e	Decreases with e
Geologic age (t_g)	Increases with t_g	May increase with t_g	May decrease with t_g
Cementation (c)	Increases with c	May Increase with c	May decrease with c
OCR	Increases with OCR	Not affected	Not affected
Plasticity Index (PI)	Increases with PI if OCR > 1; Stays constant if OCR =1	Increases with PI	Decreases with PI
Cyclic strain (γ_c)	-	Decreases with (γ_c)	Increases with (γ_c)
Frequency of loading (f)	Increases with f	Not affected significantly	Stay constant or may increase with f

Vucetic and Dobry (1988), Lanzo et al. (2009), and Mortezaie and Vucetic (2013) performed cyclic simple shear tests on clays and showed that the rate of cyclic stiffness degradation with number of load cycles, increases with γ_c. Vucetic and Dobry (1988, 1991) analysed stiffness degradation patterns with number of cycles in high and low plasticity clays. They concluded that stiffness degradation decreases with PI in clays and it is not as significant, when compared with the large effect of PI on modulus reduction curves for clays.

2.6 Behaviour of sensitive Eastern Canadian clays

2.6.1 Origin

Leda clay (also known as Champlain Sea clay) in Eastern Canada is a unique form of highly sensitive marine clay, which shows a tendency to change from a relatively stiff condition to a liquid mass when its natural structure is disturbed. In Canada, a large deposit of sensitive clay is found along St. Lawrence River and the Ottawa valley in Eastern Canada where Champlain Sea once existed as seen in Fig 2.14 (Canadian Mortgage and Housing Corporation, 2001).

Locat et al (1984) studied the mineralogy of the Eastern Canadian sensitive clays and found that plagioclase, quartz, microcline, hornblende, dolomite and calcite are the main minerals that make up these clays. It is thought that sensitivity is caused due to the presence of a metastable fabric, nature of cementation between clay particles, weathering, leaching or changes in the pore water chemistry etc (Sangrey, 1972(a, b); Locat et al, 1984).

Eastern Canadian sensitive clay has been the underlying cause of many deadly landslides in Canada. It has been associated with more than 250 mapped landslides, the latest in

Saint-Jude, Quebec, in 2010, which claimed the lives of a family of four. As, a result Leda clay is considered as a challenging material to deal with for geotechnical engineers.

Fig 2.14 Extent of sensitive clay in Eastern Canada (after Canadian Mortgage and Housing Corporation, 2001)

2.6.2 Monotonic undrained characteristics

The shear characteristics of Leda Clay have been investigated by few researchers. Lee (1979), Lefebvre and LeBoeuf (1987), and Lefebvre et al. (1989) indicate that Leda clays show a brittle and strain softening stress strain behaviour under in–situ effective stresses when they are structured or over consolidated. They are also strong and oriented in typical horizontal bedding planes. However, they behave more ductile once they become normally consolidated or restructured.

Lefebvre and La Rochelle (1974), and Lee (1979) noted that Leda clays show a typical bulging failure with well-defined shear planes when sheared in triaxial under undrained

conditions. Lee (1979) observed that axial strain at failure increases with increasing confining stress from the study on Outardes River sensitive clays in Quebec.

Mitchell (1970) conducted drained triaxial tests on Leda clay to characterize the long term stability and found that the behaviour of Leda clay is elasto-plastic, and that a yield curve can be established. These tests also showed that mode of failure in Leda clay depend on their average effective normal stress value 'p''. Mitchell observed failure along weaker plane and due to breakage of bonds in low and intermediate p' values, while work hardening type flow failure observed under higher p'.

Furthermore, Eden and Mitchell (1970), and Mitchell (1975) found different strength and mode of failure within Leda clays from different locations of Eastern Canada. Lee (1979) reported significant differences in sensitivity of Eastern Canadian clays even in close proximity in Outardes.

As in the case of other clays, strength characteristics of Eastern Canadian clays depend on various factors, such as effective consolidation stress, OCR, anisotropy, rate of loading etc. Silvestri et al (1989) showed that Leda clay has anisotropic strength characteristics, from direct simple shear tests conducted on specimens cut at different orientations. Similar observations were made by Mitchell (1970) and Alshawmar (2014) as well.

Moreover, consolidation pressure plays an important role in shear strength characteristics of Leda clays and their behaviour is different when they are consolidated above their pre consolidation pressure value (Lefebvre and LeBoeuf, 1987). Silvestri et al (1989) observed that while naturally over consolidated or structured Leda clays poses strength

anisotropy, once consolidated above their pre consolidation pressure they do not show strength anisotropy as their natural structure is damaged.

Lefebvre and LeBoeuf (1987) found that effect of loading rate on characteristics of Leda clay also depends on whether it is over consolidated or normally consolidated. Undrained strength increases with increasing strain rate in over consolidated state. But loading rate alters the stress path due to varying pore pressure development in normally consolidated state, while failure strength envelope remained same.

2.6.3 Cyclic shear characteristics

Various investigations (e.g. Sangrey et al, 1969; Mitchell and King, 1976; Raymond et al, 1979; Javed, 2011) show that the behaviour of Leda clay under cyclic loading is governed by their sensitive structure and pore water pressure response. Cyclic stresses above a certain threshold value are capable of disturbing their structure and produces higher excess pore water pressure. This excessive pore water pressure produces significant effective stress reduction. As a result, the effective stress path moves towards the origin and failed under shear at the monotonic strength envelope.

These studies also found that under lower cyclic stress amplitudes the intact structure of Leda clay has not been disturbed significantly; as a result there is no significant pore water pressure generation that is capable enough to bring the stress path towards the static failure envelope. Thus, these small cyclic stresses do not make significant impact on the behaviour of Leda clay.

Mitchell and King (1976) performed an extensive investigation on cyclic behaviour of sensitive Leda Clay samples from Ottawa under one way triaxial conditions. They found that critical/threshold cyclic stress that causes remoulding and associated significant

strength loss of these clays is at least 50% of their undrained monotonic strength (S_u). Similarly, Lefebvre et al. (1989) conducted undrained triaxial tests on Grande Baleiile River clay of Quebec. They reported a threshold cyclic stress of about 60% of their S_u and observed this threshold increases by about 30% when loading frequency is increased from 0.01 to 2 Hz. Javed (2011) conducted cyclic triaxial tests on Eastern Canadian Sensitive clays and found threshold cyclic shear strength depends on its sensitivity number.

Lefebvre and Pfendler (1996) reported cyclic strength of 40% in excess of their monotonic strength in Leda clay as a result of very high loading frequencies from cyclic simple shear investigation. Lefebvre and LeBoeuf (1987) observed effects of loading frequency on the cyclic behaviour based on OCR, similar to the strain rate effects on monotonic undrained strength. They observed a cyclic strength reduction with reducing frequency in OC clay, while normally consolidated clays do not show significant strength change. Moreover, Javed (2011) found that effect of loading frequency is only dominant in the initial loading cycles and diminishes with increasing number of cycles.

Based on cyclic simple shear tests with and without initial static shear on Leda clay from Quebec, Lefebvre and Pfendler (1996) concluded that application of undrained initial shear reduces cyclic strength of these clays. However, they found presence of initial static shear significantly reduced the rate of cyclic strength degradation with the number of load cycles.

Lee (1979) and Lefebvre et al (1989) investigated the post – cyclic strength of Eastern Canadian clays and concluded that when the clay do not fail due to cyclic loading or experience a cyclic stress amplitude less than the 'threshold stress', it will not suffer

significant post cyclic strength loss. This observation again agree with the basic understanding in the literature that when the in-situ structure of sensitive clay is not significantly affected by cyclic loading, these sensitive clays do not lose significant strength or do not show a significant change in their subsequent shear behaviour.

2.6.4 Modulus and damping properties

Vucetic and Dobry (1991) stated modulus and damping of highly sensitive or quick clays may be different from that of non/less sensitive clays disregarding their PI values. They suggested the use of material specific curves for important projects dealing with such clays. Rasmuseen (2012) carried out resonant column and cyclic simple shear tests on Ottawa area Leda clay and obtained modulus and damping values. It was compared with the curves of Vucetic and Dobry (1991) based on PI values and was found Leda clay does not behave in a similar way as non-sensitive clays.

However, various site response analysis in Eastern Canada have been, and are being carried out using Vucetic and Dobry (1991) modulus reduction and damping curves based on plasticity index of Leda clay, as there is no well-established material specific modulus reduction and damping curves for the Eastern Canadian sensitive clays. This specifies the need for more research on modulus and damping properties of Eastern Canadian sensitive clays in order to carry out safe and optimal designs and analysis.

3 EXPERIMENTAL PROGRAMME

The major emphasis of this research programme is an experimental investigation on Eastern Canadian Leda clay using the Carleton University simple shear (CU simple shear) device. In addition, improvements will be made to the existing simple shear device to enable additional tasks such as investigation at small strain levels. This experimental program involves monotonic and cyclic shear tests to determine strength parameters of Leda clay, and small strain tests to determine material specific modulus and damping curves.

This chapter presents details of the equipment used in the research program, and test material. The first part of the chapter elaborates the existing simple shear device, and the improvements and modifications made to this device to enhance its performance. The second part discusses the testing procedure including specimen preparation method. Then details of tests conducted are presented. The geotechnical properties of tested Leda clay are presented at the end of this chapter.

3.1 CU simple shear device

The simple shear device available at Carleton University is an NGI Type device (Bjerrum & Landva, 1966). It uses cylindrical soil specimens confined by steel-wire reinforced rubber membrane. The vertical consolidation stress is applied from the bottom and shear stress is applied at the top of the specimen. The original commercial device is from Seiken Incorporation (Model ASK DTC 148). However, it was rebuilt in house to permit strain controlled and stress controlled loadings in order to carry out monotonic and cyclic simple shear tests (Vipulanantham, 2011). A photograph and schematic diagram of CU

simple shear device are shown in Fig 3.1 & Fig 3.2 respectively. Fig 3.3 shows the interface that is used to control the data acquisition program.

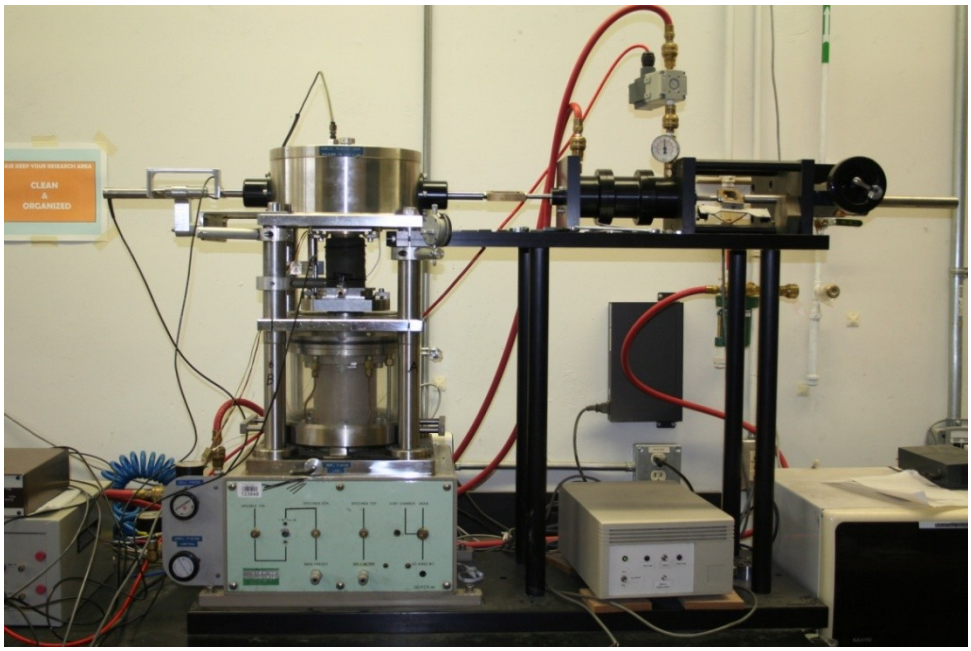

Fig 3.1 Carleton University simple shear device

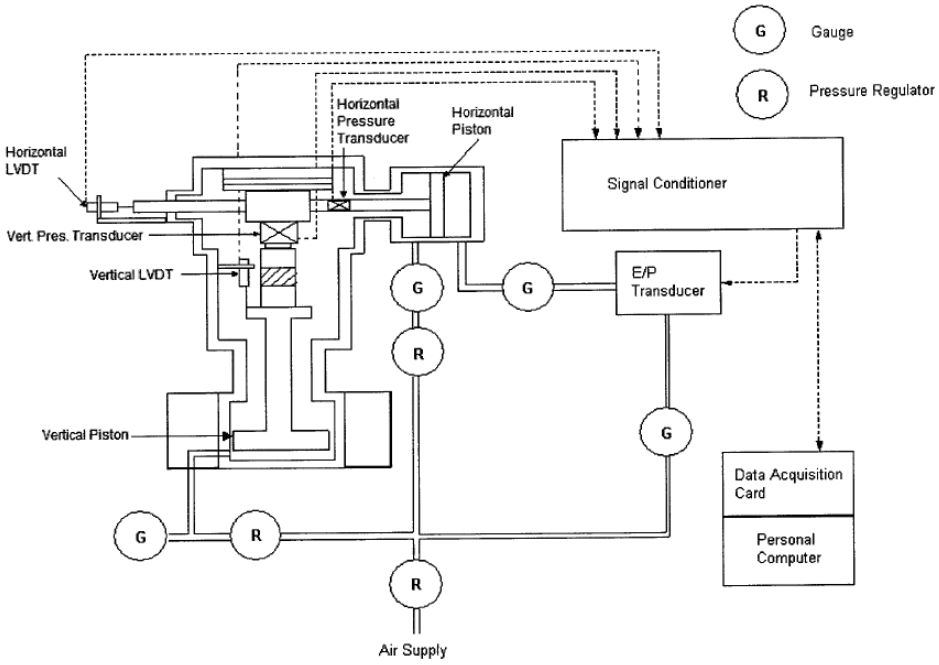

Fig 3.2 Schematic diagram of CU simple Shear device (after Ha (2003))

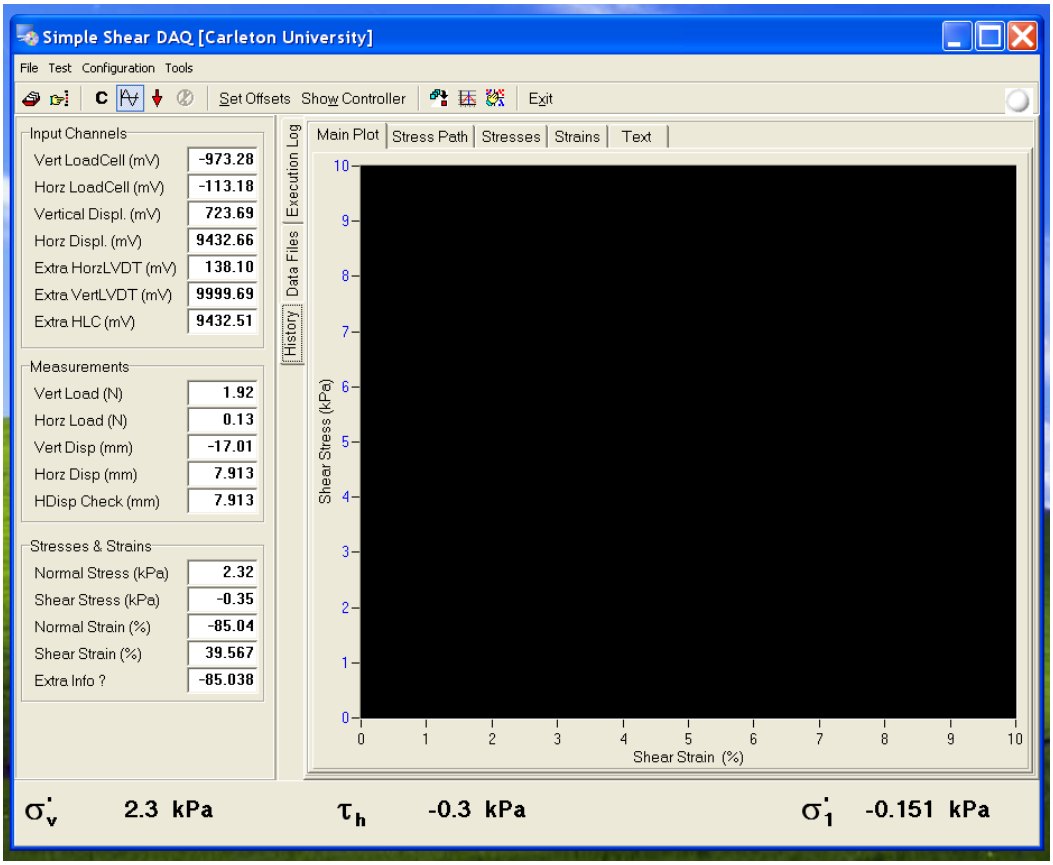

Fig 3.3 Computer interface of CU simple shear data acquisition program

This device has two sets of metal end platens (top and bottom). They are 63.5 and 70 mm of diameters which enables testing of either 63.5 or 70 mm diameter soil specimens. These end platens have thin ribs spaced evenly to ensure proper transfer of shear stresses to the soil specimen and to prevent sliding during loading. Moreover, a small circular shaped porous stone is fixed at the centre of these platens to facilitate drainage during consolidation stage. The top platen can be connected and clamped to a relatively rigid cross beam which is fixed in the top arrangement of this device. The bottom platen can be placed at a flat platform.

A typical soil specimen employed in this device is cylindrical in shape with either 63.5 or 70 mm of diameter and 20 mm of height. The large diameter to height ratio reduces the

stress non uniformities which arise due to lack of complementary shear (Kovacs & Leo, 1981). Lateral confinement by the wire reinforced rubber membrane leads to K_0 consolidation at all times. The photograph of a reinforced rubber membrane used for these experiments is shown in Fig 3.4.

Fig 3.4 Wire reinforced rubber membrane used for simple shear tests

A typical simple shear test consist two stages; consolidation and shearing. The consolidation is achieved by applying a vertical load by a pneumatic piston located at the bottom of the device. Lateral confinement is not applied externally, but mobilized naturally depending on the characteristics of the material with the help of reinforced membrane. This K_0 consolidation condition simulates the natural consolidation stress state of a soil element under level ground.

Undrained shearing conditions are achieved by maintaining constant volume of specimen as in a typical NGI simple shear test device. The height of the specimen is kept constant by clamping a vertical lock at the bottom of the device while reinforced membrane maintains constant cross section by preventing lateral deformations. CU simple shear

device is capable of performing monotonic and cyclic shearing. Monotonic shearing is strain controlled and a stepper motor (Model No. 5 624-39) employed in this system is used to apply strain controlled shearing. This motor is connected through electronic transducers to a computer which controls the motor. Furthermore, stress controlled cyclic shearing is carried out by a double acting pneumatic piston by varying pressure in pressure chambers using an electro-pneumatic transducer, which is also controlled by the computer.

3.1.1 Measurements of stresses and strains

Load cells and linear variable differential transducers (LVDT) are used to measure loads and displacements. The changes in loads or displacements produce changes in voltage in these transducers. These changes in voltage are processed by a signal conditioner and transferred to a data acquisition card of the computer. Two Honeywell load cells fixed at the top chamber measure vertical (normal) and horizontal (shear) stresses. The vertical load cell has capacity of about 6000 N and horizontal load cell has capacity of about ± 950 N. This enables the application of 1600 and 250 kPa normal and shear stresses in the 70mm specimen, and 1900 and 300 kPa normal and shear stresses in the 63.5mm specimen with resolutions better than 0.2 kPa. Vertical displacement is measured by a vertically fixed 'Honeywell' LVDT. Two LVDTs one with a small range and the other with a wide range, both fixed horizontally outside of the device measure horizontal displacements. The use of two LVDTs enhances measurement resolution at lower strains, and permit testing to larger strains.

3.1.2 Modifications to existing simple shear device

Major modifications have been made to the existing CU simple shear device in order to measure very small shear strains with minimal noise disturbance and to perform automated quasi cyclic shearing.

First major modification is the replacement of existing small horizontal D/C LVDT that is used to measure small shear strains with a more sensitive A/C LVDT in very small strain tests. The noise level in the existing LVDT readings was too large to measure very small strains accurately. In fact, it was never intended to measure shear strains up to 0.01% in the past. Hence, it is not possible to carry out very small strain controlled shearing to calculate Modulus and Damping with this LVDT.

Therefore it was decided to include a very small-range 'Honeywell' A/C LVDT. This LVDT has a range of ± 0.2 mm and has a very high resolution (0.02μm). It is less sensitive to electrical noise and other disturbances. The inclusion of this small LVDT facilitates measurements up to 10^{-3}% shear strain amplitudes. The fluctuation of this LVDT due to noise and other disturbances during a period of 8 hours is shown in Fig 3.5 and compared with that of existing horizontal D/C LVDT. Furthermore, this new small LVDT was fixed in a specially designed LVDT holder as shown in Fig 3.6. Such arrangement of this LVDT (measuring towards left) using the special assembly minimizes any accidental damage due to an unexpected impact or large strain event. This is important as the LVDT is vulnerable to damage due to its very small range and spring mechanism.

This LVDT is connected to the signal conditioner through a special device which transforms A/C signal from this LVDT to D/C signal. Furthermore, the data acquisition

programme has been modified to switch automatically between the two horizontal LVDTs when the shear strain exceeds the limits of small LVDT.

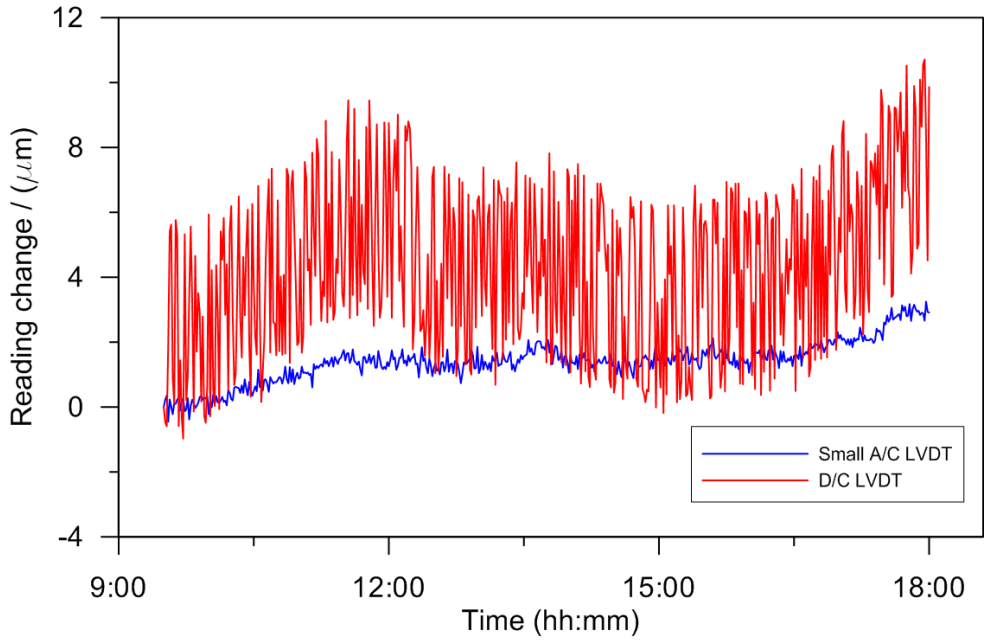

Fig 3.5 Noise level readings in horizontal small A/C and large D/C LVDTs

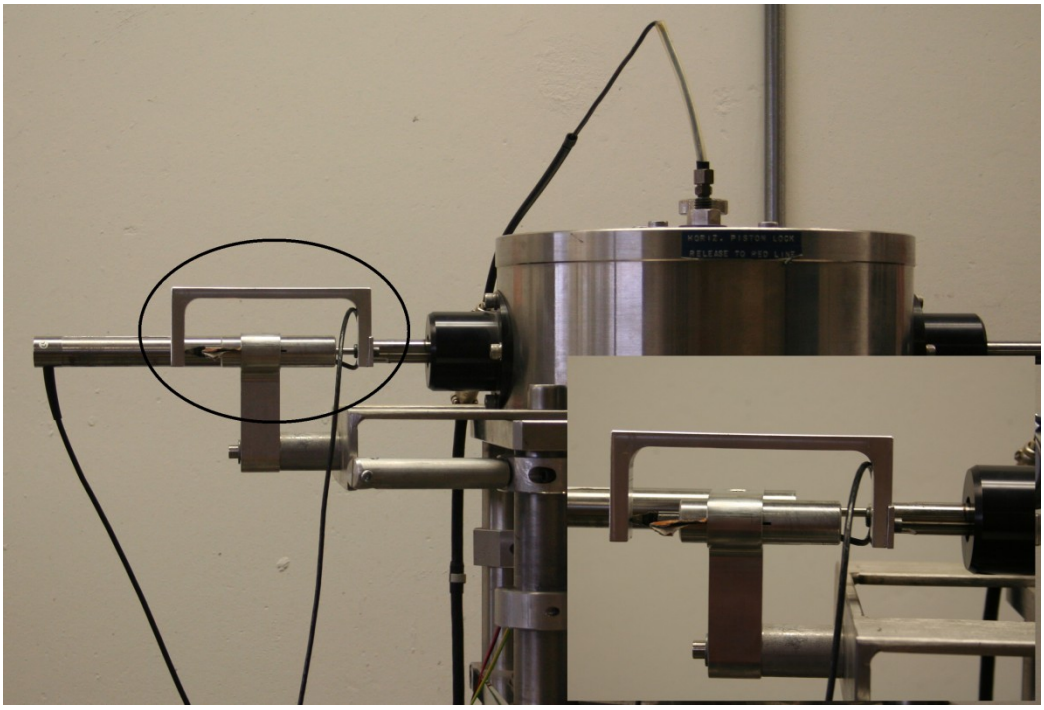

Fig 3.6 Small horizontal LVDT in a specially designed holder

Another important aspect of undrained simple shear tests is maintaining 'zero' volume change. Even though this is achieved by locking a vertical clamp in the device, it is important to closely monitor any kind of vertical displacement due to wear and tear and/or defects in the vertical clamp. The current large D/C vertical LVDT employed in the device to measure consolidation is vulnerable to noise disturbances, hence, less sensitive in measuring very small vertical displacements accurately during shearing phase. In addition, it shows false large displacement readings due to noise levels.

Therefore a second very sensitive small A/C LVDT (similar to the small horizontal LVDT) was included to monitor the vertical displacements during undrained shearing. By employing this LVDT, it was possible to confirm that vertical strains are smaller than 0.01% during shearing. The fluctuations of this small vertical LVDT during a period of 8 hours are compared with that of the A/C LVDT in Fig 3.7; it clearly illustrates that the small LVDT is less affected due to noise level and other fluctuations and provide reliable measurements.

Another major modification is made to the data acquisition programme to felicitate very small strain controlled quasi cyclic loading. Existing system enables the user to program only one loading step at a time in the monotonic mode. Therefore, user must interact at the end of each loading or unloading stage, in order to perform a strain controlled quasi cyclic loading. This can cause interruptions and delays which result in stress and / or strain variations at the end of each stage.

In order to overcome this issue, data acquisition system was modified to program all the loading steps at the beginning of the shearing through a user friendly interface as shown in Fig 3.8. Here, the user can define the maximum strain limit, direction of loading and

strain rate for each loading step. Hence a cyclic strain controlled loop can be performed without an interruption.

In addition, existing data acquisition system controls stepper motor based on pulse count during monotonic loading. Required strain is converted to the number of necessary pulses needed to be produced by stepper motor. Hence, any possible delays or inaccuracies would result in lack of target strain. The motor would stop even before achieving target strain, if the full pulses would have been applied. In order to overcome this issue, data acquisition system was modified to check the strain at the end of pulse count. It was programmed to calculate and apply required pulses to achieve target strain in case of a deficit. This new feature ensures the achievement of intended target strain without any deficit.

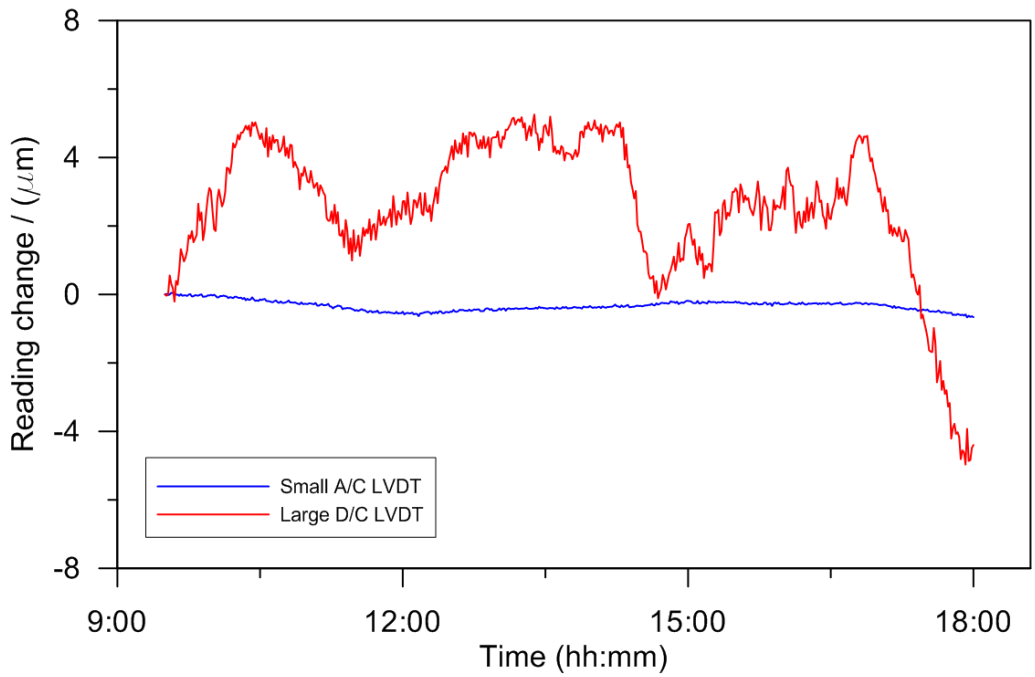

Fig 3.7 Noise level readings in vertical small A/C and large D/C LVDTs

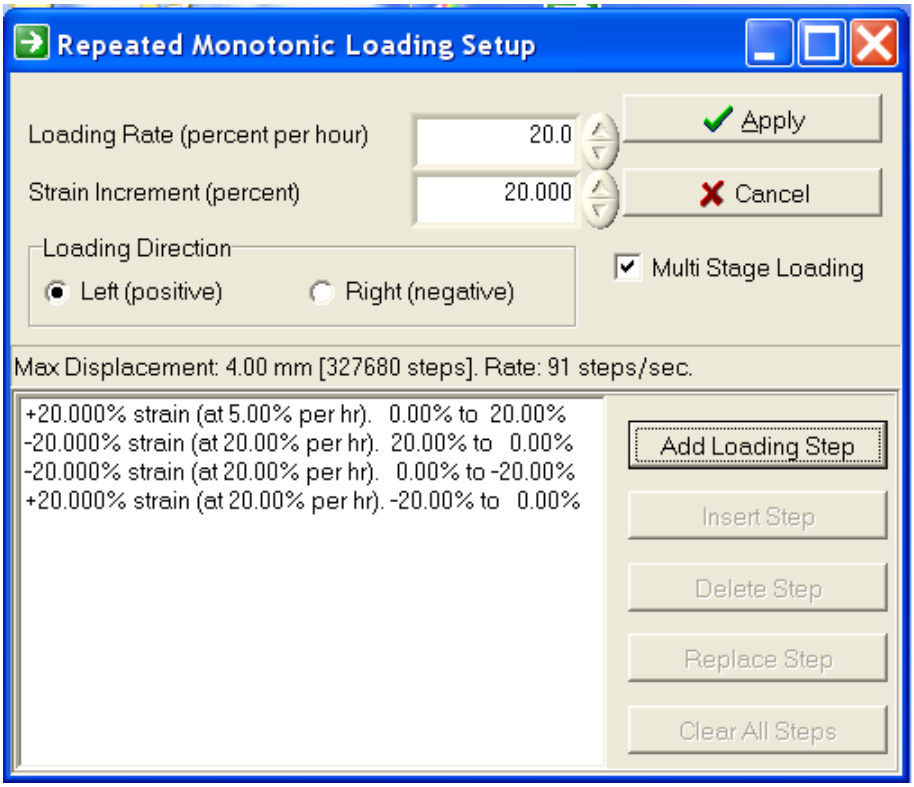

Fig 3.8 User interface for programming automated repeated monotonic loading

Similarly, a modification was made to the cyclic loading system as well. Existing system allows user to program one specific cyclic stress value (τ_{cyc}) at a time. This was modified to program multiple stages with different cyclic stress amplitudes at the beginning of shearing similar to multi stage strain controlled loading. Parameters such as the required number of cycles, frequency and stop criterion can be specified for each step. This feature is helpful in performing a multi stage cyclic stress controlled test with different cyclic stress amplitudes without any interruption.

These modifications enable CU simple shear device to run tests smoothly and provide confident measurements at small strain and stress amplitudes which cannot usually achieved in a typical simple shear device.

3.1.3 Calibration of Load cells and LVDTs

All the load cells and LVDTs have been calibrated at the beginning of the research programme. LVDTs have been calibrated using an LVDT calibration device (Starret No.716); Voltage change is measured for each known displacement and gain values (μm / mV) were obtained from the gradient of the plot between voltage and displacement. Load cells cannot be removed easily from the device. Therefore, a separate load cell, which has been calibrated using known calibration weights, was used in series to calibrate vertical and horizontal load cells. Calibration charts for small horizontal LVDT and Vertical load cell are shown in Figures 3.9 and 3.10 respectively.

Moreover, small horizontal LVDT which works on a spring mechanism as shown in Fig 3.6 exerts small stiffness / resistance force to shearing when the spring is undergone a displacement. This force has been determined as a function of LVDT reading in mV as shown in Fig 3.10. Based on the data obtained the resistance is approximately 0.1 kPa for a strain level of 1%. A correction has been made to accommodate this force for the measured shear stress in the data acquisition programme. In addition, effects of noise and other disturbances on these transducers had been monitored for a long time and were found to be insignificant.

3.2 Test procedure

Undisturbed clay specimens have been prepared and tested in simple shear device. These specimens were prepared away from the device and then moved into the apparatus. The following procedure was adopted in preparing specimens for simple shear experiments.

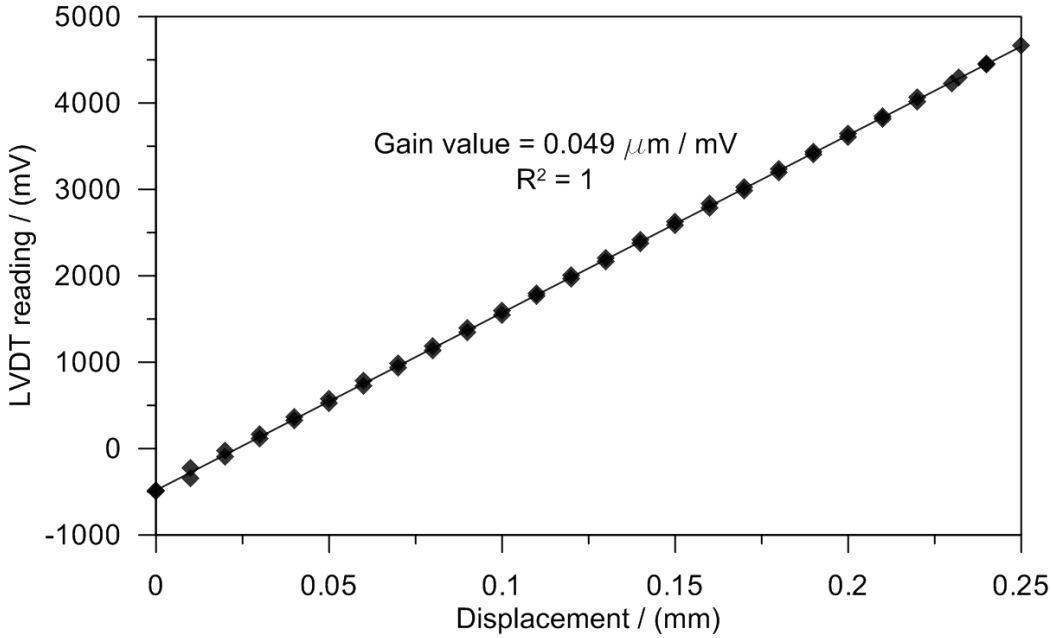

Fig 3.9 Calibration curve for small horizontal D/C LVDT

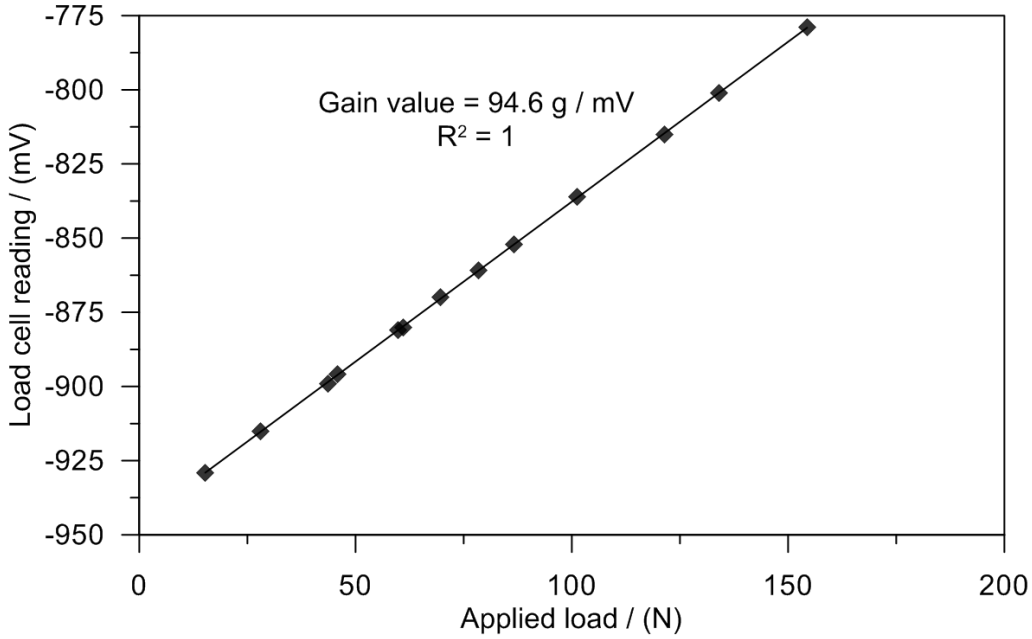

Fig 3.10 Calibration curve of Vertical load cell

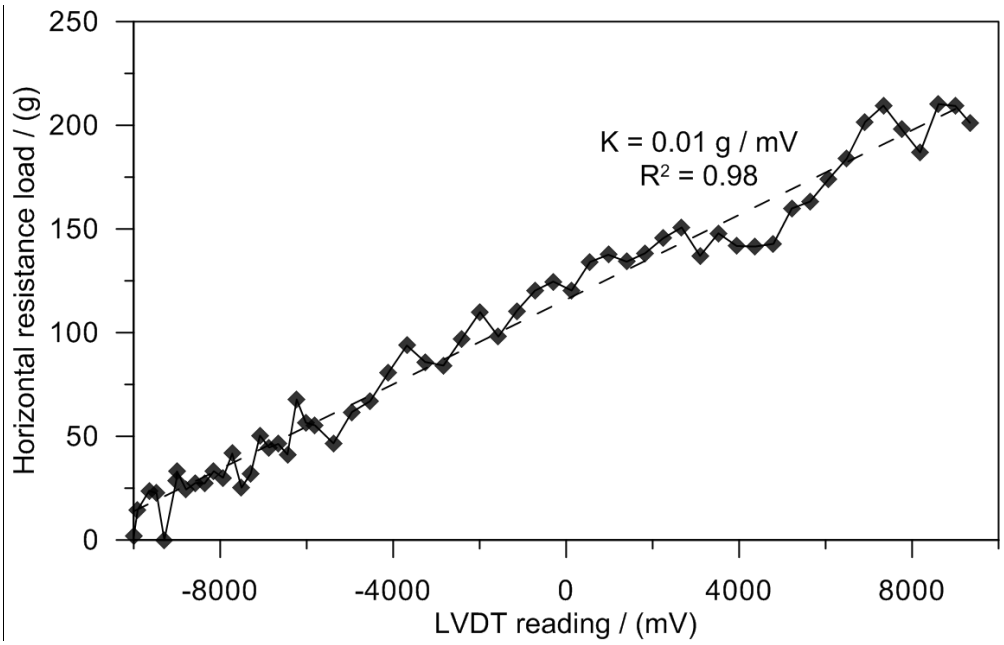

Fig 3.11 Variation of resistance of small horizontal LVDT with mV reading

3.2.1 Specimen preparation method

Initially the simple shear programme was started on the computer and initial readings of the load cells and LVDTs ('offsets') were taken and made offset. A dummy sample block of known height was used to establish a reference height offset in order to determine the height of the specimen.

Initially, the sample was trimmed by a wire saw few millimetres larger than the actual diameter and height of the specimen. A special soil lathe trimmer (Fig 3.12) was used to achieve these dimensions. Then a cutting ring with the diameter of the specimen and height few millimetres larger than specimen height was inserted into the trimmed sample using the soil lathe as shown in Fig 3.12. Once the soil was collected into the ring, top surface was levelled carefully with a wire saw. Then necessary height (20mm) of the specimen was achieved by inserting the extension collar of the ring. Then the bottom surface was levelled using a wire saw and weight of the sample was determined. Fig 3.13

shows a levelled clay specimen in the cutting ring. Also, samples were collected from the trimmings to determine moisture content of soil.

The reinforced membrane was placed on the bottom pedestal and sealed with an O – ring. Then, two halves of split moulds were placed around the membrane and this mould was held tight with a clamp. Top part of the membrane was flipped over the top edge of the mould. At this point a small suction was applied through the two ports in the mould in order to hold the membrane and create a membrane lined cavity. Then, the soil sample was transferred to this cavity carefully while drainage valve of the bottom pedestal was kept open. Once soil is placed into the cavity, the top pedestal was placed in a way that the ribs are perpendicular to shearing direction in order to prevent slip. Then membrane was flipped and sealed with another O – ring. After that, specimen was moved to the platform of simple shear apparatus.

3.2.2 Testing

Following the placement of the sample in the base platform of the device, the pressure in the vertical piston was increased slowly in order to move the platform up and to make contact with the top loading assembly. Once the top platen made contact, initial height was taken at a vertical stress level of about 10 kPa. Then, top and bottom pedestals were clamped tightly with their respective platforms and the split mould was removed carefully. After that, a predetermined consolidation stress (σ'_{vc}) was applied in stages by increasing pressure using the vertical piston. Usually, the target vertical stress was reached with a load increment ratio of two ($\Delta\sigma = 2$). All the initial consolidation stresses were allowed for one hour and the last consolidation stress (target stress) was applied for about 12 hours (overnight) to ensure complete consolidation and dissipation of pore water

pressure. Both stresses and strains were monitored throughout the consolidation process. The final sample height was recorded at the end of consolidation and the vertical clamp tightened to arrest the vertical displacement. This imposition of constant height, combined with the use of steel wire reinforced membrane results in constant volume.

Monotonic (single stage or multi stage) or cyclic shearing was carried out under constant volume conditions. Usually this shearing starts from zero towards positive (left direction) and continues. Normal Stress (σ'_v), Shear stress (τ), vertical and horizontal displacements have been recorded by the data acquisition system throughout the shearing process. Pore water pressure in a constant volume test can be determined using change in total vertical stress in simple shear (Dyvik et al, 1987). A slow strain rate of 5% per hour was used in the case of monotonic loading conditions in order to ensure pore pressure equalization within the clay specimen. Stress controlled cyclic loading was applied at a period (T) of 10 sec in cyclic loading tests. At the end of the shearing sample was removed and final weight of sample was taken to determine changes in density and water content.

3.3 Test programme

The test program was intended to determine strength characteristics of Leda clay and to determine modulus and damping values. Monotonic and cyclic tests were conducted on block samples obtained at two different depths from Ottawa area; one at around 15 to 17 m (50 – 57 ft) and the other is at a shallower depth around 3 m (11 ft).

Fig 3.12 Cutting ring pushed into clay sample using soil lathe

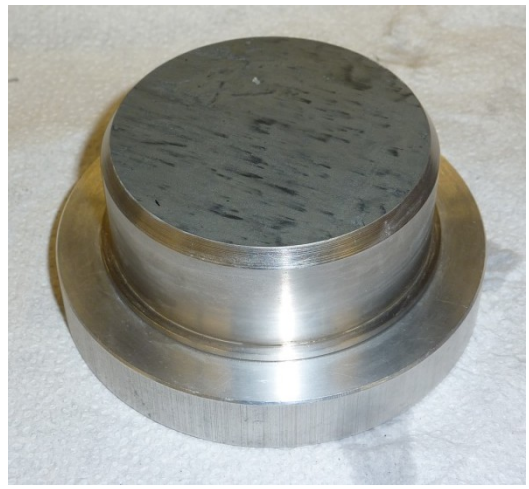

Fig 3.13 Clay specimen after collected in the ring for simple shear testing

Typical monotonic tests were carried out to investigate monotonic strength characteristics of Leda clay under different consolidation stress levels (σ'_{vc}). These σ'_{vc} levels varied from 50 to 800 kPa in these tests. Strength parameters such as cohesion (c') and effective internal angle of friction (φ') were also determined. In addition, strength reduction characteristics of Leda clay were also investigated by continuing some of these tests to large strain cycles, and evaluating the strength characteristics during subsequent loading events.

Cyclic tests were carried out to find liquefaction characteristics, cyclic resistance ratio (CRR) and post cyclic behaviour of Leda clay. These tests were carried out at different effective vertical stress levels (σ'_{vc}) and at specific cyclic stress values (τ_{cyc}) or cyclic stress ratio (CSR). These cyclic tests were followed by a post cyclic monotonic test in order to investigate characteristics of Liquefied clay. Table 3.1 and 3.2 lists the monotonic and cyclic tests conducted in this research programme to determine strength characteristics of Leda clay.

Small strain controlled quasi cyclic tests were conducted to find modulus and damping curves of Leda Clay. These tests were conducted at the above two depths and at different σ'_{vc} levels in order to assess the dependence on modulus and damping curves on potential natural material variability and effective stress level. These tests have been carried out by shearing a clay sample to different small γ_c amplitudes. Moreover, few stress controlled multi stage cyclic tests were also carried out to obtain additional points in these curves at very small strain levels. These tests were conducted by carrying out few small stress controlled cycles with different magnitudes. Table 3.3 lists the details of small strain / stress tests.

The details, reasoning and intention of each kind of tests are discussed extensively in the results section.

Table 3.1 Details of monotonic tests on Leda clay

Test ID	Borehole	Consolidation stress (σ'_{vc}) / (kPa)
MON – 01	BH 2 – 50’ 10” C	100
MON – 01 – A	BH 2 – 50’ 10” C	100
MON – 02	BH 2 – 50’ 10” C	200
MON – 03	BH 2 – 50’ 10” C	400
MON – 04	BH 2 – 50’ 10” C	800
MON – 05	BH 1 – 50’ 08” B	50
MON – 06	BH 2 – 11’ 07” B	100
MON – 07	BH 2 – 11’ 07” B	400
MON – 08	BH 2 – 55’ 11”	100
MON – 09	BH 2 – 55’ 20”	200
MON – 10	BH 2 – 57’ 11” A	400

Table 3.2 Details of cyclic tests on Leda clay

Test ID	Borehole	Consolidation stress (σ'_{vc}) / (kPa)	CSR	No of cycles (N)
CYC – 01	BH 2 – 55’ 11” B	100	0.240	53
CYC – 02	BH 2 – 55’ 11” B	100	0.275	19
CYC – 03	BH 2 – 55’ 11” B	100	0.310	10
CYC – 04	BH 2 – 55’ 11” B	100	0.340	4
CYC – 05	BH 1 – 55’ 10” C	200	0.150	37
CYC – 06	BH 1 – 55’ 10” C	200	0.175	21
CYC – 07	BH 1 – 55’ 10” C	200	0.205	9
CYC – 08	BH 1 – 55’ 10” C	200	0.235	5
CYC – 09	BH1 – 57’ 10” A	400	0.143	60
CYC – 10	BH1 – 57’ 10” A	400	0.168	23
CYC – 11	BH1 – 57’ 10” A	400	0.178	6
CYC – 12	BH1 – 57’ 10” A	400	0.190	5
CYC – 13	BH2 – 11’ 07” C	100	0.260	65[1]
CYC – 14	BH2 – 11’ 07” C	100	0.300	38
CYC – 15	BH2 – 11’ 07” C	100	0.330	27
CYC – 16	BH2 – 11’ 07” C	100	0.338	5

Table 3.3 Details of single and multistage small strain or stress controlled tests

Test ID	Borehole	Consolidation stress (σ'_{vc}) / (kPa)	Stage	Cyclic strain amplitudes (γ_{cyc}) / % or Cyclic stress amplitudes (τ_{cyc}) / (kPa)
MON – M&D - 01	BH 2 – 55’ 10” C	100		γ_{cyc} (%) = 0.01, 0.02, 0.05, 0.1,0.2,0.5,1,2 &5
MON – M&D - 02	BH 2 – 55’ 10” C	200		
MON – M&D - 03	BH 2 – 55’ 10” C	400		
MON – M&D - 04	BH 2 – 55’ 10” C	800		

[1] Estimated value

MON – M&D - 05	BH 2 – 11’ 07” B	50		γ_{cyc} (%) = 0.01, 0.02, 0.05, 0.1,0.2,0.5,1& 2
MON – M&D - 06	BH 2 – 11’ 07” B	100		
MON – M&D - 07	BH 2 – 11’ 07” B	200		
CYC – M&D - 01	BH 1 – 57’ 11” B	100	1	τ_{cyc}(kPa) = 1,2,4,8,14,20
		200	2	τ_{cyc}(kPa) = 1,2,4,8,15,25
		400	3	τ_{cyc}(kPa) = 1,2,5,10,20,30
		800	4	τ_{cyc}(kPa) = 2,4,10,20,40
CYC – M&D - 02	BH 2 – 11’ 07” C	50	1	τ_{cyc}(kPa) = 0.5,1,2,4,7,10
		100	2	τ_{cyc}(kPa) = 1,2,4,8,13
		200	3	τ_{cyc}(kPa) = 1,2,5,10,17,24
		400	4	τ_{cyc}(kPa) = 1,2,5,10,20,30

3.4 Material tested

Monotonic and cyclic tests have been carried out in Leda clay obtained from the Museum of Nature site. Samples were obtained from two zones; a relatively shallow zones at a depth of about 3-4m, and deeper zone ranging from 15-17.5m. As far as clays are concerned geotechnical characteristics such as Atterberg limits, natural moisture content, compressibility, particle size distribution or percent fine particles and chemical composition are important in their behaviour. Therefore index tests have been carried out in these samples in order to determine these properties.

3.4.1 Atterberg Limits

Liquid limit (LL) and Plastic limit (PL) tests were carried out on samples from both zones. Liquid limit varies from 25 to 55, and Plastic limit from 10 to 30 in these samples. The higher values correspond to the shallow zones, and these values closely agree with the values reported for this site by Alshawmar (2014). Table 3.4 shows the details of Atterberg limit values and related indexes of tested soil at different depths.

Table 3.4 Atterberg limit and moisture content properties of tested Leda clay

Borehole ID	Depth / (m)	Natural moisture content (w) / (%)	Liquid Limit (LL) / (%)	Plastic Limit (PL) / (%)	Plasticity Index (PI)	Liquidity Index (LI)
BH 2 – 50' 10" C	15.5	28	24	14	10	1.75
BH 2 – 55' 10" C	17	34	27	15	12	1.7
BH 1 – 57' 10" A	17.6	32	27	15	12	1.7
BH 2 – 11' 07" B	3	80	55	25	30	1.9

3.4.2 Particle size distribution

Particle size distribution at the depths of around 16m and 3m were determined using Hydrometer and wet sieving method. Percentage of fine particles that passes through No 200 sieve were determined from wet sieving. Based on these results, percent of particles finer than 75 μm were 92% and 100% at depths of 16 and 3 m respectively. Hydrometer tests were conducted to determine the finer particle size distribution of this clay. Percent of clay sized particles were determined as 35% and 75% at 16m and 3m depths respectively. Fig 3.14 shows the particle size distribution of tested soil.

3.4.3 Compressibility

Alshawmar (2014) conducted detailed investigation on the consolidation characteristics of these Leda clay samples using odometer. $e\ vs \log \sigma'_v$ curves presented for similar depths from this investigation are shown in Fig 3.16.

3.4.4 Chemical analysis

The chemical composition of Leda clay was analyzed (by commercial laboratories) to determine the amount of metals such as Potassium (K), Sodium (Na), Calcium (Ca), Magnesium (Mg) and Chloride (Cl). The compositions of these chemicals by weight and in leachate are listed in Table 3.5.

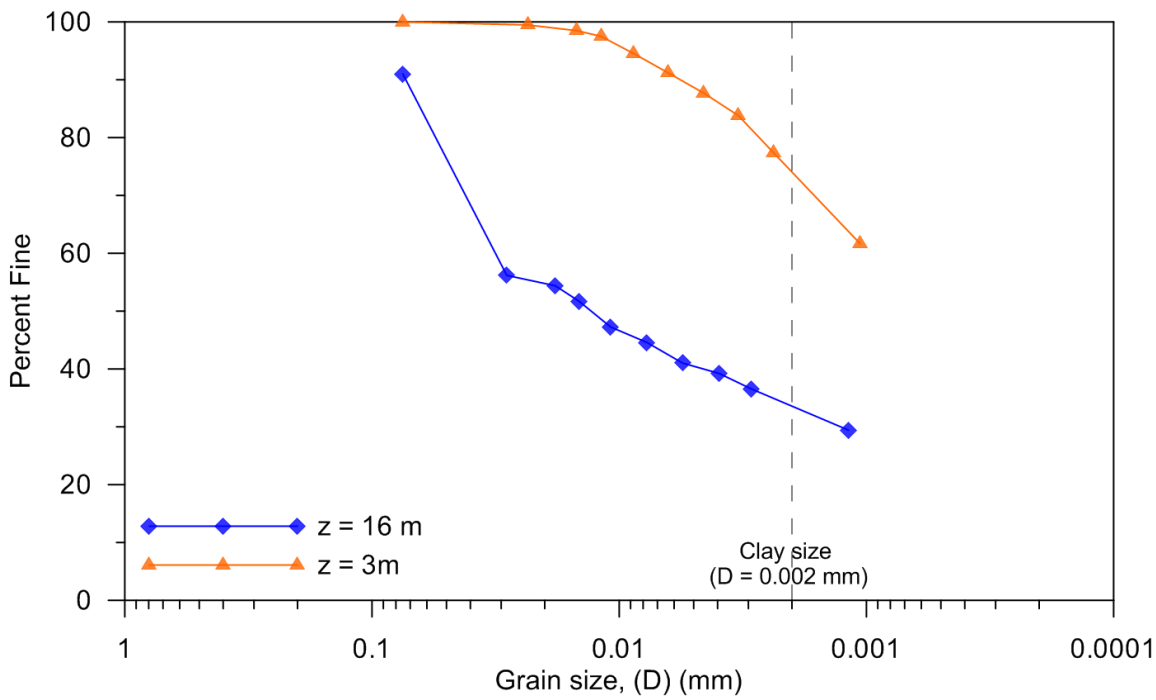

Fig 3.14 Particle size distribution of tested Leda clay

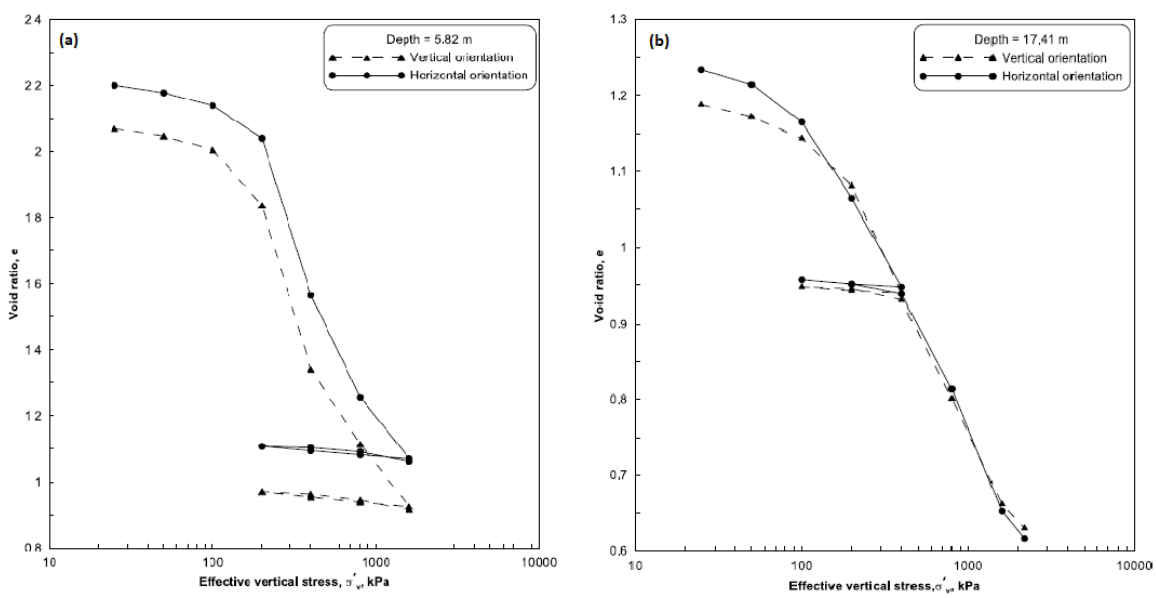

Fig 3.15 Consolidation curves of Ottawa area Leda clay from similar depths (a) z = 5 m (b) z = 17 m (after Alshawmar, 2014)

3.4.5 Sensitivity

The sensitivity of Leda clay obtained from about 16m depth was determined to be in the range of 12 to 24. The laboratory vane shear strength of the undisturbed clay was about 24 kPa. The strength of the remoulded clay was measured to be in the range of 1 to 2 kPa. A more accurate value was not available due to the resolution of the laboratory vane.

Table 3.5 Chemical composition of Leda clay at different depths

Depth	Composition of solids by weight (μg / g)					Composition in leachate (mg / L)		
	Cl	Ca	Mg	K	Na	Ca	Mg	K
3	1600	14300	25500	15200	3180	84	25	11
15	10	24200	14000	5470	1440	440	36	13
17	7	25000	16400	6840	1880	420	52	15

4 MONOTONIC AND CYCLIC BEHAVIOUR OF LEDA CLAY

4.1 Introduction

One of the objectives of this research study is to investigate the monotonic and cyclic strength characteristics of Leda Clay under simple shear loading condition. This chapter presents monotonic and cyclic shear characteristics of Leda clay under different consolidation stress levels and discusses the effect of prior stress / strain history. The first part of the chapter presents results obtained from monotonic tests carried out in simple shear device. Cyclic, post cyclic behaviour and liquefaction susceptibility are discussed in the next section. Finally the strength reduction characteristics of sensitive Leda clays are investigated during and after cyclic and monotonic loading events of different magnitudes.

4.2 Monotonic behaviour of Leda clay

Undisturbed samples of Leda clay retrieved from a site in the city of Ottawa were tested to determine the monotonic stress strain characteristics of Leda clay. Monotonic tests were carried out on samples obtained at two different depths (15m & 3m) and consolidated to a range of vertical effective stress (σ'_{vc}) values. Specimens obtained from a depth of 15 m (BH2 50'10") were tested under σ'_{vc} values from 50 to 800 kPa. Similarly monotonic behaviour of specimens obtained at a shallower depth (3m) was investigated under a σ'_{vc} range from 50 to 400 kPa. These σ'_{vc}values were selected to yield a range of states, from over consolidated (OC) to normally consolidated (NC).

Typical stress strain behaviour and stress path of a sensitive Leda clay specimen tested under σ'_{vc} of 100 kPa is shown in Fig 4.1. This clay shows a typical strain softening stress-strain behaviour. This resembles the observations made on Leda clay in literature

(Lee, 1979; Lefebvre & LeBoeuf, 1987; and Lefebvre et al, 1989). Initially, shear stress (τ) increases with shear strain (γ) and reaches a peak strength (S_u). Upon reaching a peak state, the sample strain softens and the shear strength approaches a residual value at very large strains. Pore water pressure steadily increases during shearing and as a result σ'_v reduces continuously. Thus, stress path moves towards the origin in a τ vs σ'_v space as seen in Fig 4.1 (b). The details of monotonic tests carried out in this research are given in Appendix A.

4.2.1 Effects of effective vertical consolidation stress (σ'_{vc})

Fig 4.2 compares the monotonic behaviour of Leda clay tested from 15 m depth under different σ'_{vc} values. Undrained strength (S_u) increases with σ'_{vc}. Stress-strain curves exhibit a clear peak state followed by strain softening at 400 and 800 kPa consolidation stress level, but no such peak is seen in the tests at 50 and 100 kPa. It should be noted that consolidation stress level is only partially responsible for the behavioural differences observed in Fig 4.2. The density of the material and the consolidation ratio are also different between the tests. In specimens that exhibit a peak state, the peak strength (S_u) was obtained at γ values between 3.5 and 6.5 % under these σ'_{vc} levels. These strain values are consistent with the range at which peak strength was observed for Eastern Canadian Leda clays in the literature (Lefebvre & Pfendler, 1996; Rasmussen, 2012; Alshawmar, 2014). In addition, shear strain (γ) at failure slightly increases with increasing consolidation stress level. This is similar to the observations made by Lee (1979) on Outardes River clay.

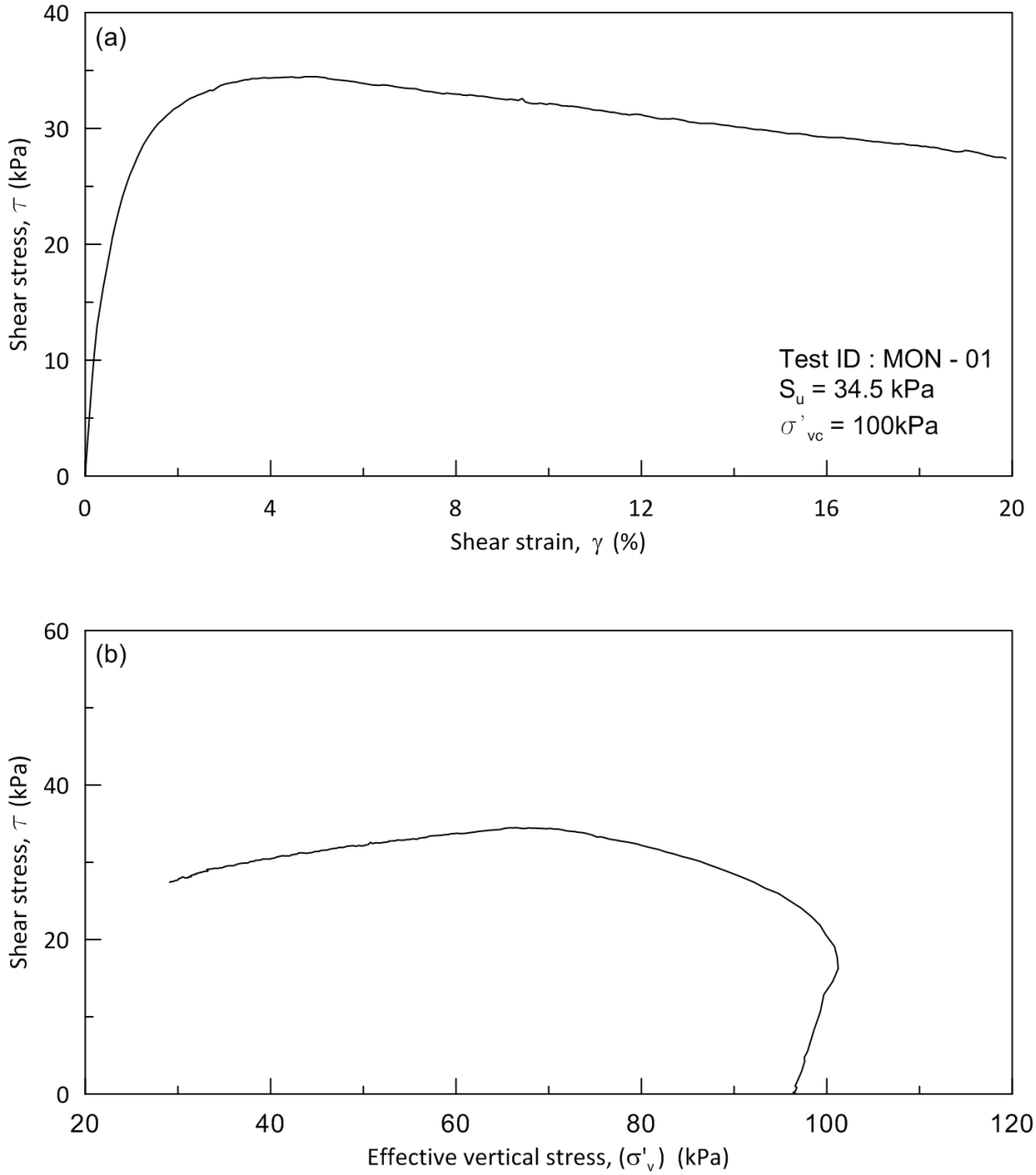

Fig 4.1 Monotonic behaviour of Ottawa Leda clay under σ'_{vc} of 100 kPa (a) Stress strain behaviour (b) Stress path

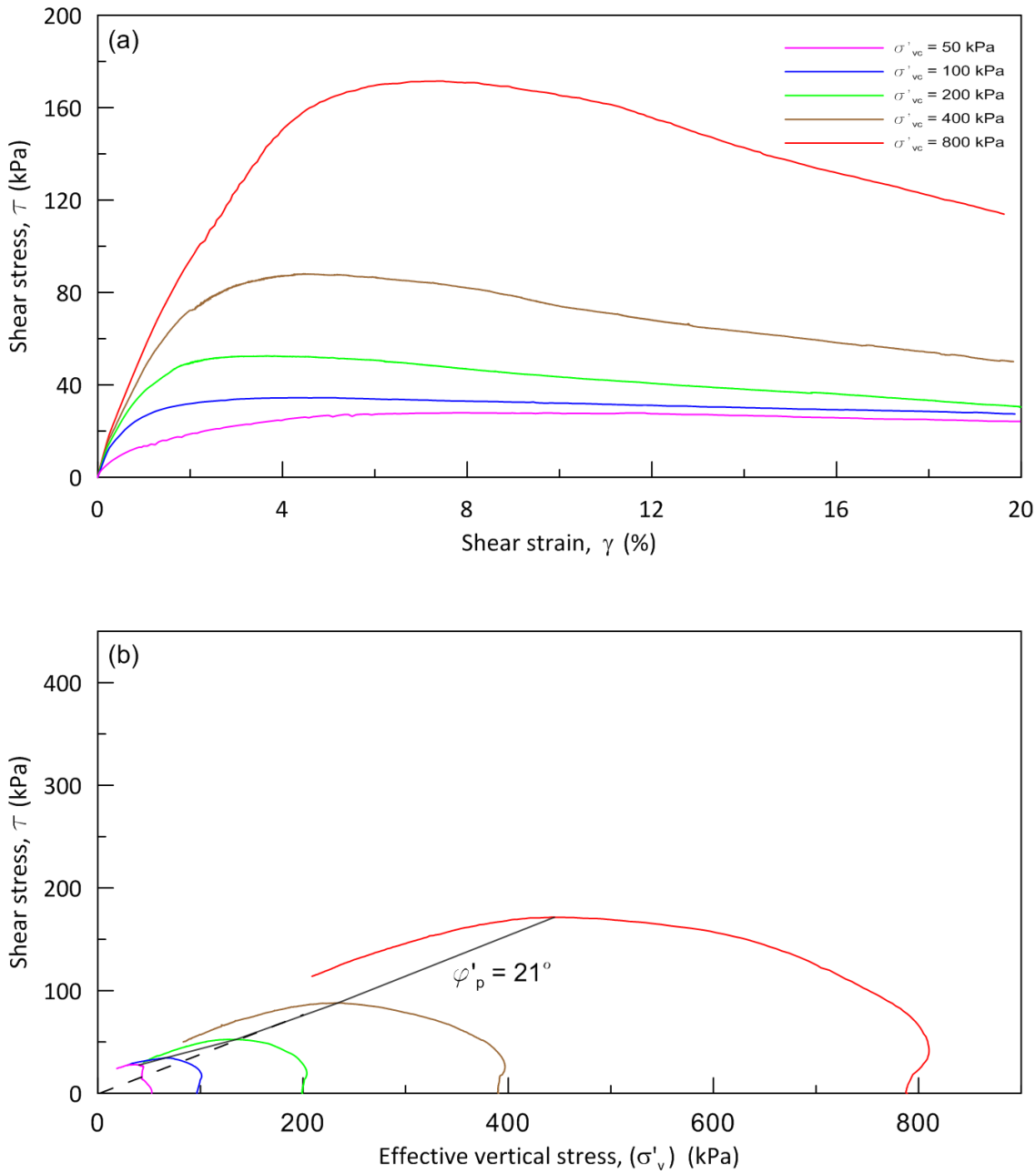

Fig 4.2 Comparison of monotonic behaviour of Leda clay at z = 15 m under different σ'_{vc} (a) Stress strain behaviour (b) Stress path

Alshawmar (2014) reported pre consolidation pressure values (σ'_{pc}) around 200 kPa from odometer tests conducted on this clay at depths from 5m to 20m. Hence σ'_{pc} at the depths of 15m and 3m are around 200 kPa (Fig 3.15). Moreover, data obtained from consolidation stages of simple shear tests also agree with these σ'_{pc} values. Therefore

specimens consolidated below $\sigma'_{vc}$of 200 kPa were in an over consolidated (OC) stage and specimens consolidated above 200 kPa were in a normally consolidated (NC) state. In addition the structure of the clay undergoes considerable collapse as the effective stress increases to over 400 kPa as seen from the consolidation curve in Fig 3.15 (Alshawmar, 2014).

4.2.2 Effect of OCR

Test results shown in Fig 4.2 for specimens tested from 15 m depth are plotted in a normalized form in Fig 4.3 to highlight the effect of over consolidation on the stress-strain characteristics. As noted previously, clay specimens tend to show a peak at a narrow γ range in the normally consolidated region. At the same time specimens in the over consolidated region do not show a clear specific peak. There is some softening in these tests, but the stress strain curves tend to stay at a peak state for relatively larger γ range. Fig 4.3 shows that when shear stress (τ) is normalized with σ'_{vc}, the stress–strain curves of specimens in the NC state converges within a fairly narrow band. Peak values of τ/σ'_{vc} reach 0.22 for specimens consolidated under σ'_{vc} values of 400 and 800 kPa. Moreover, they follow almost the same normalized stress path as seen in Fig 4.7 (b). This behaviour again confirms the principles in literature about NC clays that normalized behaviour of normally consolidated clay is essentially unique (e.g. Roscoe et al, 1958; Ladd, 1991).

On the other hand, specimens consolidated under σ'_{vc} values of 200, 100 and 50 kPa shows peak τ/σ'_{vc} values of 0.25, 0.35 and 0.52 respectively. As noted earlier the pre consolidation pressure of this clay has been determined to be about 200 kPa, and thus the

tests at 50 and 100 kPa correspond to OCR values of about four and two respectively. Clearly, over consolidated strength envelope of clay is higher than that of normally consolidated clays, and the shear strength increases with OCR. This finding is consistent with the observations made in OC clays by many researchers (e.g. Hvorslev, 1930; Ladd & Foot 1974) and show that general behavioural characteristics of sensitive Leda clay is not dissimilar to the understanding established in the literature.

Normalized monotonic behaviour of samples from a shallower 3m depth also confirms the above findings. Clay in the NC state (σ'_{vc} = 400 kPa) shows τ/σ'_{vc} of 0.23. However, τ/σ'_{vc} of over consolidated Leda clay at this depth increases with OCR from 0.24 to 0.84 when σ'_{vc} varies from 200 to 50 kPa.

Normalized stress path of clays obtained at 15 m depth is shown in Fig 4.6 (b). This figure illustrates the pore pressure characteristics of clays during undrained loading. NC clay shows continuous development of positive excess pore pressure during undrained shearing (specimens consolidated under σ'_{vc} of 400 and 800 kPa). Slightly over consolidated clays such as the specimens consolidated to σ'_{vc} of 100 and 200 kPa also follows a similar trend and develops positive pore pressure. At the same time, a moderately or highly over consolidated clay such as the one consolidated under 50 kPa shows a trend to develop negative pore pressure (dilative behaviour) after an initial excess pore pressure development, however, the data in this test shows some inconsistency.

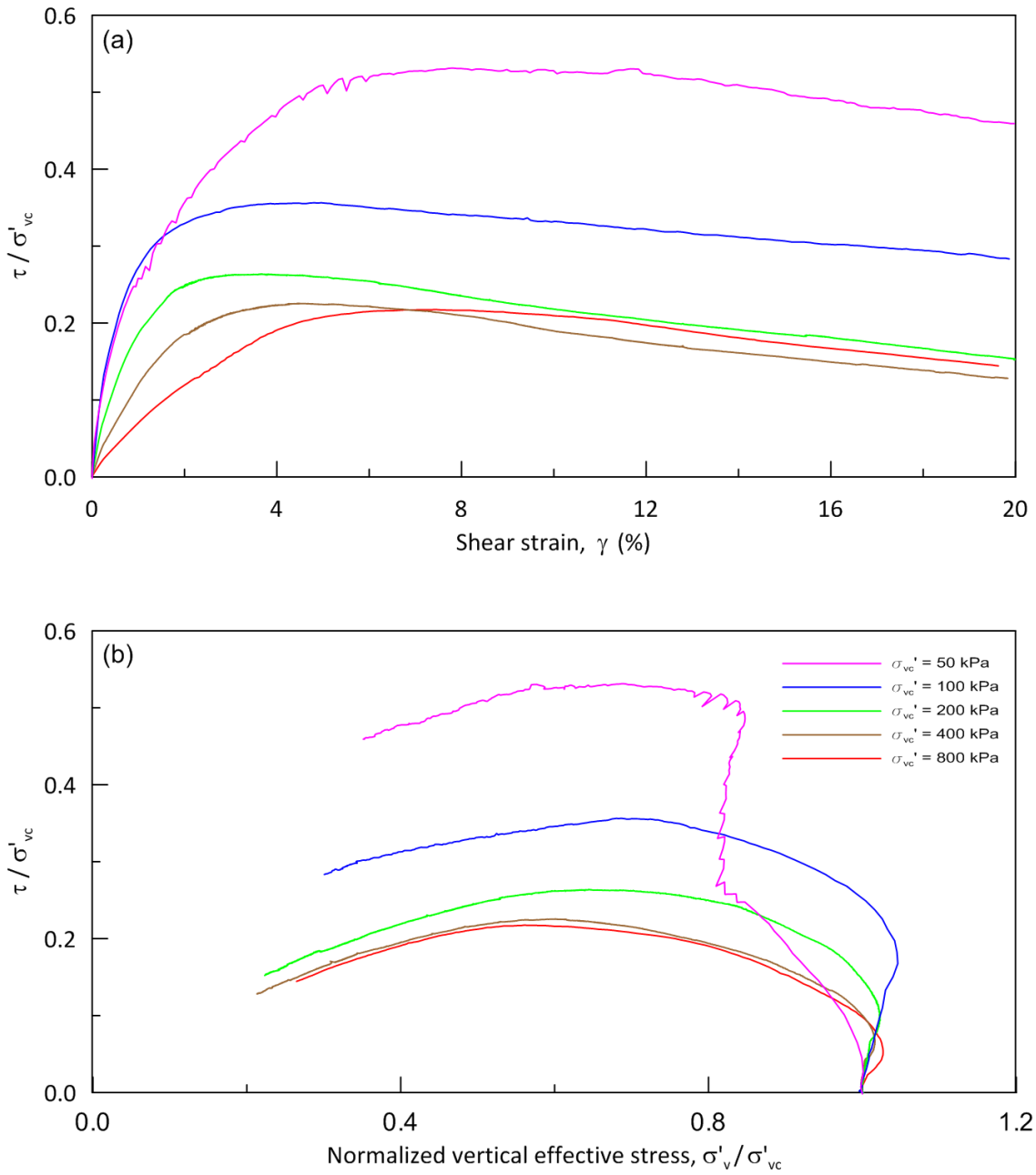

Fig 4.3 Comparison of normalized monotonic behaviour of Leda clay at z = 15 m

4.2.3 Monotonic strength parameters

Monotonic strength parameters such as internal effective friction angle (φ') and effective cohesion (c') can be obtained from stress path plots as the one shown in Fig 4.2 (b).The data points corresponding to the peak stress states for specimens obtained from a depth of

15 m can be approximated by two straight line segments in the stress path plot (Fig 4.2 (b)) above and below 200 kPa consolidation stress. Similarly, the peak shear stresses (S_u) against their respective σ'_v values are plotted in Fig 4.4 for specimens tested at both depths (z = 15m and 3m). This plot also shows two distinct responses based on the region of σ'_v.

The points in the normally consolidated region at 15 m depth can be approximated by a single line with an approximate slope of 21° (i.e. $\varphi'_p = 21°$). In addition, this friction angle at peak strength (φ'_p) value of NC Leda clay closely matches with the φ'_p value reported by Alshawmar (2014). Also, this line almost passes through the origin when extended back. This observations show Leda clay is almost cohesionless in the normally consolidated state and S_u increases linearly with σ'_v. The only data point in NC state of clay obtained from 3m depth also falls closer to this NC line. This suggests that soils at both depths have similar friction angle at peak state, but more data is necessary to confirm the φ'_p value at shallower depths, since the percent clay particle size is significantly higher in the shallower regions.

Similarly, clay specimens in the over consolidated region show a reduced friction angle at peak state, but shows cohesion (c'). In addition, strength envelope of Leda clay in this region is not a straight line (e.g. Silvestri et al, 1989), but can be approximated as a straight line as shown in Fig 4.4. c' of over consolidated clay at 3m depth is somewhat higher compared to the value obtained at 15 m depth. This variation in c' could possibly be attributed to the significant differences in particle size distribution at these depths. Sample obtained from 3m depth contains over 75% of clay sized particles and shows larger cohesion than the material from 17 m depth which has much lower clay

percentage. Moreover PI values are also higher at shallower depths which would add more cohesive behaviour. On the other hand, strength increases slower with σ'_v in OC region than in the NC region. This behaviour is also similar to the trend reported in Eastern Canadian clays under simple shear conditions by Silvestri et al (1989).

Even though these tests do not show a clear residual strength, shear stresses obtained at a larger γ value (i.e. γ = 20%) can be assumed as a 'residual state'. The residual values obtained at both depths are plotted against their respective σ'_v in Fig 4.5. The slope of the lines connecting these points makes an angle (φ'_r) of 26.5° at 3m depth and 25.5° at 17 m depth. In spite of the differences in clay fraction, there is no significant variation in φ'_r (friction angle at residual state) with depth. However, there are uncertainties in stress measurements at very large shear strains in simple shear as the specimen deforms significantly. Thus, it is important to note that strength parameters obtained at large strain levels may not be very accurate.

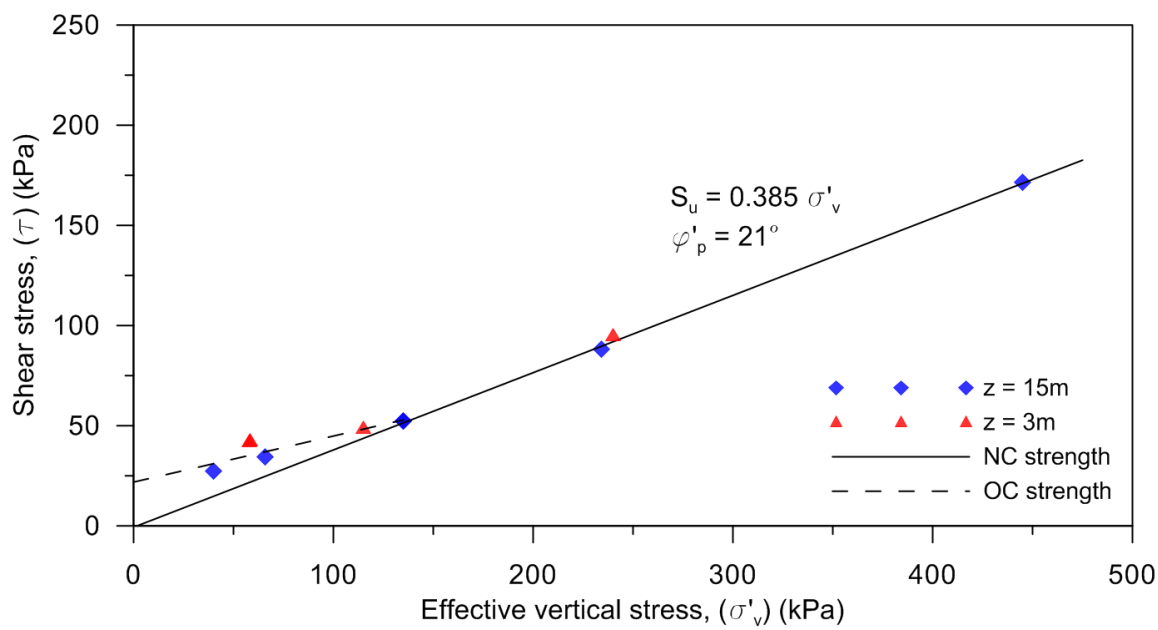

Fig 4.4 Peak strength parameters of Ottawa Leda clay

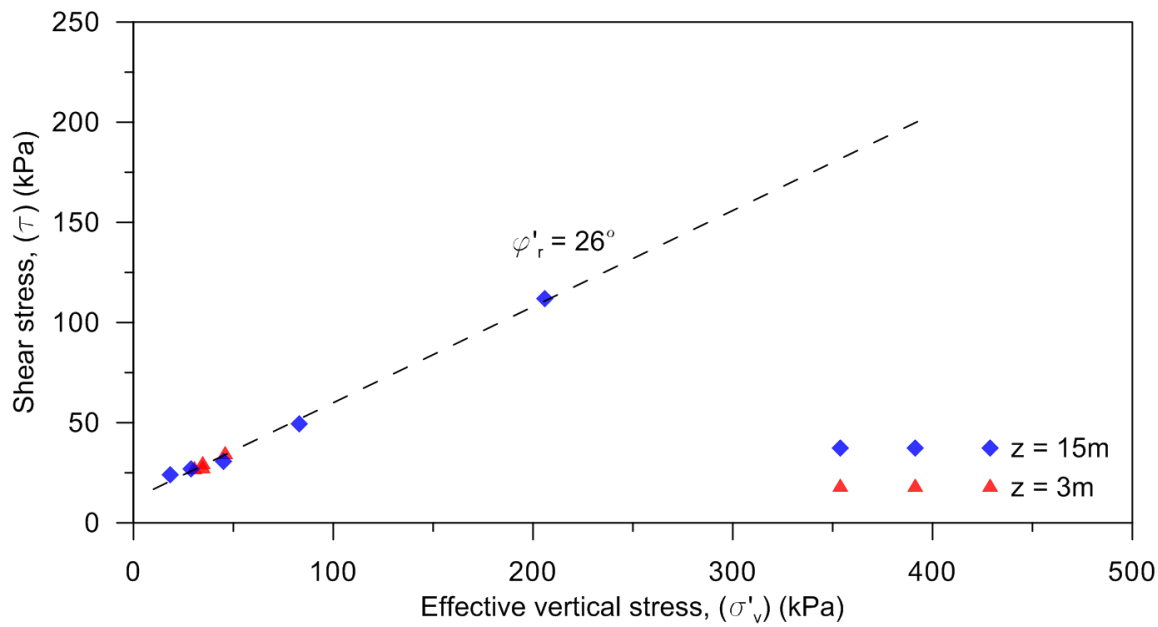

Fig 4.5 Residual strength parameters of Ottawa Leda clay

4.3 Cyclic and Post cyclic behaviour

4.3.1 Cyclic behaviour

Quebec city and Ottawa regions, which are underlain by significant amounts of Leda clay deposits, are rated by the National Building code of Canada (NBCC 2010) to face a very high seismic hazard –second only to Vancouver. Many earthquakes have rattled these cities and an earthquake of M = 6.0 to 6.5 might cause significant ground shaking in these regions. Therefore, the behaviour of Leda clay during cyclic loading events is of paramount importance given the potential for strong seismic shaking. Undisturbed clay samples obtained from the same bore holes (as in the monotonic tests discussed previously) at a depth of 16 to 17.5m and around 3m were tested to assess their cyclic strength characteristics under simple shear loading. These clays were subjected to various magnitudes of stress controlled cyclic loading intensities under different σ'_{vc} levels.

The cyclic loading was carried out by varying pressure across the chambers of a pneumatic piston in simple shear device as elaborated in Chapter 3. Predetermined cyclic stress amplitudes (τ_{cyc}) were applied to the soil specimen in a sine wave form as shown in Fig 4.6 (a). The frequency of the loading was maintained at 0.1 Hz (T = 10s) in all cyclic tests. This was required to maintain a consistent cyclic loading intensity (regardless of the level of cyclic strain development) and enabled confident measurements of stresses and strains. It is recognized that actual earthquake loading contains a range of frequency content (0.5- 5Hz range is probably the strongest modes in this region), but the effect of cyclic loading frequency on liquefaction characteristics of clays is rather secondary (e.g. Vucetic & Dobry, 1991). Further, the lower frequency used would result in a conservative estimate of the cyclic resistance of the material.

The cyclic loading was usually continued until the soil specimen reaches cyclic shear strain amplitude (γ_{cyc}) of 3.75% or until a larger number of cycles (N) such as 1000 in case the soil did not develop large cyclic strains. The γ_{cyc} amplitude of 3.75% is considered as the limit for 'liquefaction' under simple shear condition in this study as per the recommendations of NRC (1985). Clays usually do not develop 100% excess pore water pressure during cyclic loading; hence, they do not undergo 'true liquefaction' as in the case of loose sands, as discussed in the literature review (e.g. Ishihara, 1996; Perlea, 2000). Therefore, the alternative definition of liquefaction used by some researchers based on a 100% excess pore pressure generation is not appropriate for clays.

Fig 4.6 shows the results of a typical cyclic test where a constant amplitude cyclic shear stress τ_{cyc} of 27.5 kPa was applied to a specimen consolidated to σ'_{vc} of 100 kPa. Strain development is rather gradual, and the soil specimen develops a pore pressure

continuously due to this loading with increasing number of cycles. This pore pressure development produces a loss in effective stress. The associated reduction in secant modulus (G_s) is considered responsible for the gradual increase in cyclic strain amplitude (γ_{cyc}) as shown in Fig 4.6 (c).

Fig 4.7 shows the stress-strain behaviour and effective stress path of this specimen under cyclic loading. γ_{cyc} and pore pressure (Δu) increase as the cyclic loading progresses. Pore pressure development is higher during the initial loading cycles as seen in Fig 4.7 (b) than that during the consequent cycles. This behaviour agrees with the observations commonly made in other clays by many researchers (e.g. Mitchell & King, 1976; Zergoun & Vaid, 1994). Pore pressure develops continuously until cyclic loading was discontinued at γ_{cyc} = 3.75%. The final pore pressure is about 80 kPa as shown in Figures 4.6 (b) and 4.7 (b) which results in a loss of about 80% of initial σ'_v.

4.3.1.1 Effect of cyclic stress amplitude and number of cycles

Important consideration in an investigation of cyclic behaviour of soil is to determine cyclic stress amplitude (τ_{cyc}) and number of cycles (N) necessary for failure. τ_{cyc} amplitude that is capable of producing liquefaction or failure is commonly given as a function or percentage of S_u or defined using the term cyclic stress ratio, CSR (τ_{cyc}/σ'_{vc}). Therefore, a series of cyclic simple shear tests were conducted at different cyclic loading intensity values to generate a relationship between the number of cycles to liquefaction and the cyclic loading intensity under various σ'_{vc}levels. This permits an examination of the number of cycles (N) required to liquefy the clay and its dependence of the confining stress level and cyclic loading intensity.

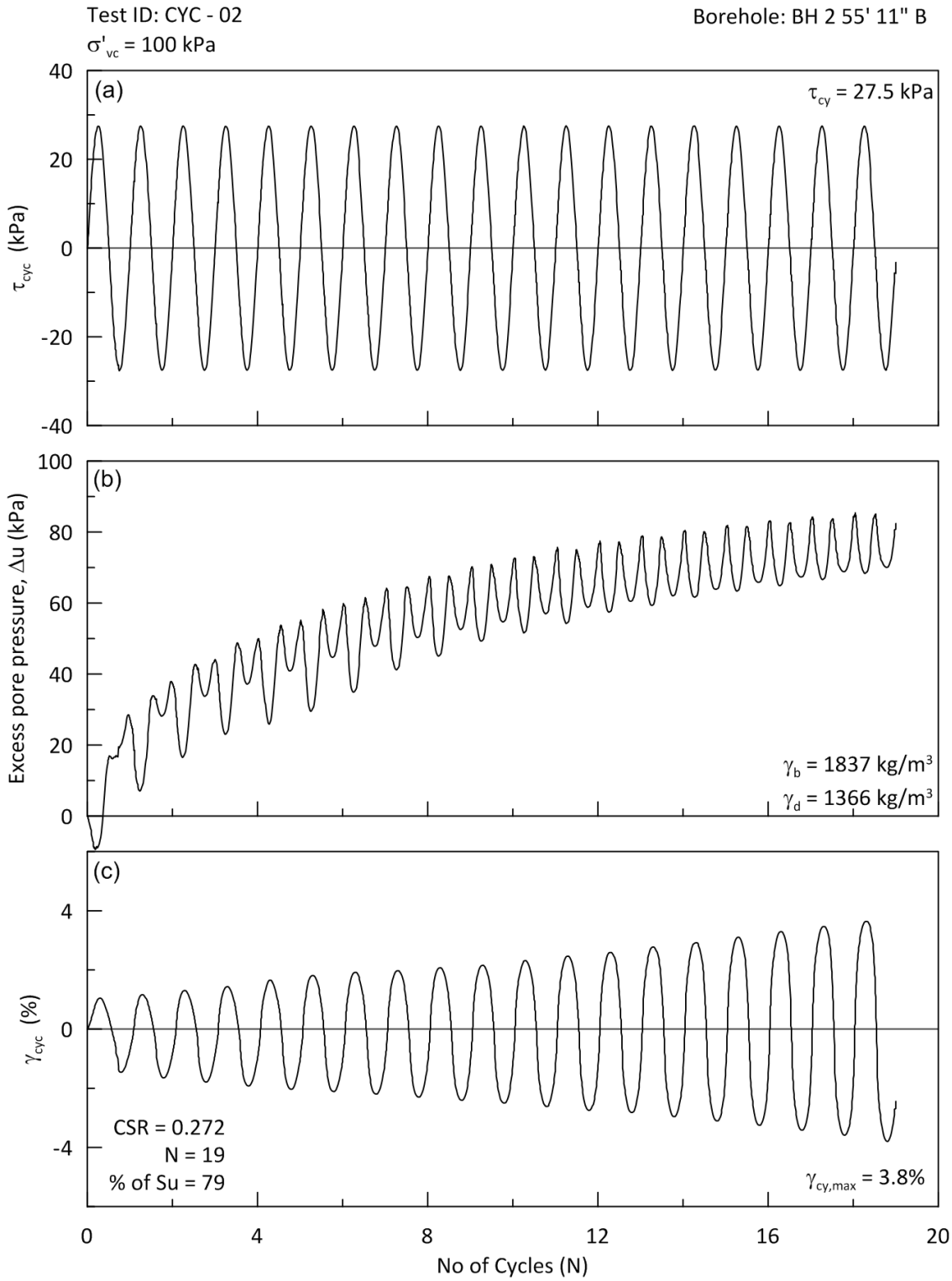

Fig 4.6 Cyclic behaviour of clay subjected to τ_{cyc} = 27.5 kPa (a) $\tau_{cyc}$with N (b) PWP with N (c) γ_{cyc} with N

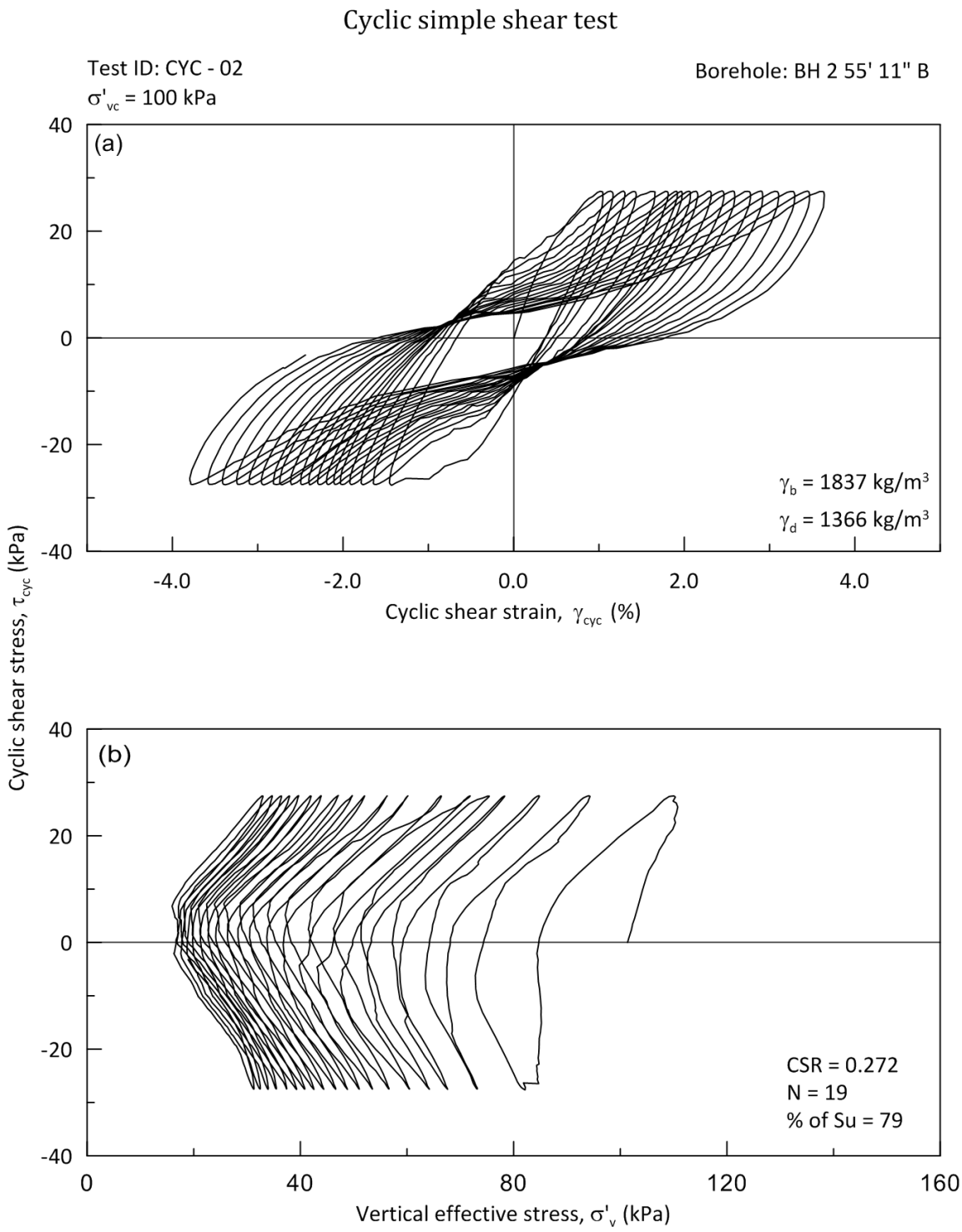

Fig 4.7 Cyclic behaviour of clay subjected to τ_{cyc} = 27.5 kPa (a) Stress strain behaviour (b) Stress path

Cyclic tests were conducted under three different σ'_{vc} levels of 100, 200 and 400 kPa. Initially, a monotonic test was performed under the specific σ'_{vc} level to determine the strength (S_u) of clay. Then τ_{cyc} amplitudes for subsequent cyclic tests were decided based on this S_u value. Also, it is a common approach to present cyclic strength in terms of S_u (e.g. Zergoun & Vaid, 1994; Lefebvre & Pfendler, 1996). At least 3 or 4 cyclic tests were performed under each σ'_{vc} in order to obtain the variation of cyclic stress or CSR with N.

4.3.1.1.1 Cyclic tests at $\sigma'_{vc} = 100\ kPa$

Cyclic simple shear tests were conducted on samples from depths of 16.5m and 3m under σ'_{vc} = 100 kPa. The undrained strength (S_u) of sample from 16.5m depth was determined as 34.7 kPa from a monotonic test (MON – 08). The first cyclic test was conducted with a cyclic shear stress of about 70% of this S_u value (i.e. τ_{cyc} = 24 kPa). Based on the number of cycles taken to reach the required target strain, τ_{cyc} for the subsequent cyclic tests were determined. In this case, γ_{cyc} of 3.75% was reached in 53 cycles in this test. As a result, τ_{cyc} values for the subsequent tests were decided as 27.5, 31 and 34 kPa. These values are approximately 80, 90 and 98% of the S_u values respectively. N values required under these cyclic stresses were 19, 10 and 4 respectively. Generally, even the most energetic earthquakes do not result in more than about 30 equivalent load cycles (e.g. Seed & Idriss, 1982) of shaking. Therefore, the relationship between CSR and N is of importance in this range. Hence, τ_{cyc} values were decided accordingly to obtain more data points in this range. Similar procedure was adopted in the series of test conducted on specimens obtained from 3m depth under σ'_{vc} of 100 kPa.

Details all cyclic tests conducted in this research programme are provided in Appendix B. Fig 4.8 shows a plot between τ_{cyc} or CSR and N for the series of tests conducted under σ'_{vc} of 100 kPa on specimens obtained from 16 m depth. A line fitted as shown in Fig 4.8 through the data points illustrates the variation of the cyclic stress ratio (CSR) or the cyclic loading intensity τ_{cyc}/S_u in Leda clays with number of load cycles (N). This curve is generally termed as the cyclic resistance curve.

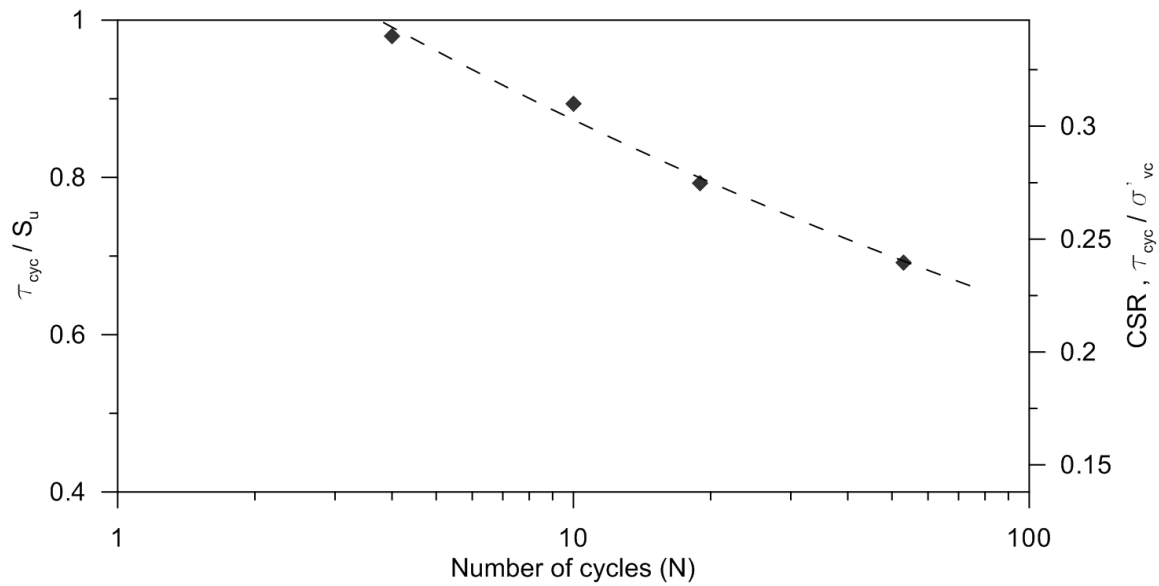

Fig 4.8 Variation of cyclic stress with N under σ'_{vc} of 100 kPa at z = 16.5 m

Fig 4.8 shows that cyclic resistance of clay reduces with N or number of cycles necessary for 'liquefaction' reduces significantly under larger cyclic stress magnitudes. Similar observations were made by a number of researchers (e.g. Lefebvre & Pfendler, 1996; Boulanger & Idriss, 2004) in a variety of clays.

4.3.1.1.2 Cyclic tests at $\sigma'_{vc} = 200\ kPa$

Similarly a series of cyclic tests were conducted under σ'_{vc} = 200 kPa with τ_{cyc} values of 30, 35, 41 and 47 kPa for samples obtained from bore hole BH1 55' 02" B (z =

16.7m). As in the previous case, a monotonic test was conducted on this clay to determine the $S_u$value. The τ_{cyc} values mentioned above are approximately 50, 60, 75 and 80% of S_u (= 58.5 kPa). The typical normalized cyclic behaviour of a specimen tested under τ_{cyc} = 47 kPa under σ'_{vc} of 200 kPa is shown in Figures 4.9 and 4.10.

4.3.1.1.3 Cyclic tests at $\sigma'_{vc} = 400\ kPa$

Similarly another series of tests was carried out under σ'_{vc} = 400 kPa for samples obtained from BH 1 57' 11" A at a depth of 17.5 m. The approximately 1m difference in the depth of the samples was expected not to cause significant differences between the samples tested at 400 kPa compared to those at 100 and 200 kPa.

4.3.1.2 Effects of effective vertical stress on cyclic behaviour

The cyclic test data obtained from these tests at different σ'_{vc} levels and from two different depths are summarized in Fig. 4.11. These curves shows cyclic strength characteristics of Leda clay and the effects of the consolidation stress level on cyclic resistance.

Fig 4.11 clearly shows that the cyclic strength of Leda clay, in terms of the cyclic loading intensity τ_{cyc}/S_u and CSR, under σ'_{vc} = 100 kPa is larger compared to those at 200 or 400 kPa. Furthermore, clay at shallower (3m) depth shows more resistance to cyclic shearing than that the clay at 16 m depth under equivalent σ'_{vc} level (=100 kPa). The best fit line for cyclic resistance at 3m depth locates higher and almost parallel to that of 17 m depth. This behaviour could possibly be attributed to the significantly higher clay content (75%) and larger liquid limit (LL = 55) or plasticity index (PI = 30) associated with the clay obtained from 3m depth. Similar observation was made in literature as well (Houston & Herrmann, 1980)

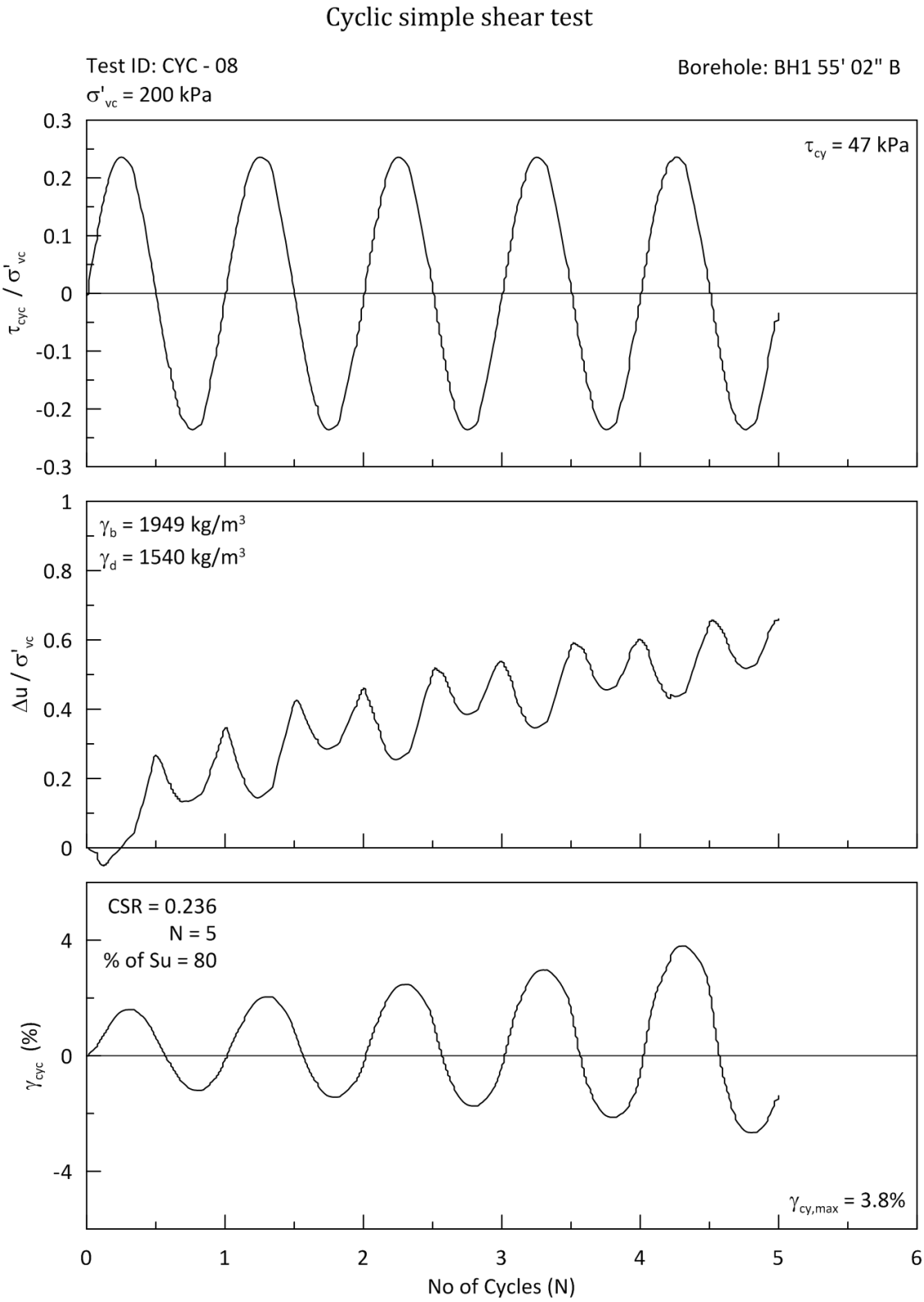

Fig 4.9 Normalized cyclic behaviour of clay subjected to τ_{cyc} = 47 kPa (a) τ_{cyc} with N (b) Pore pressure with N (c) γ_{cyc} with N

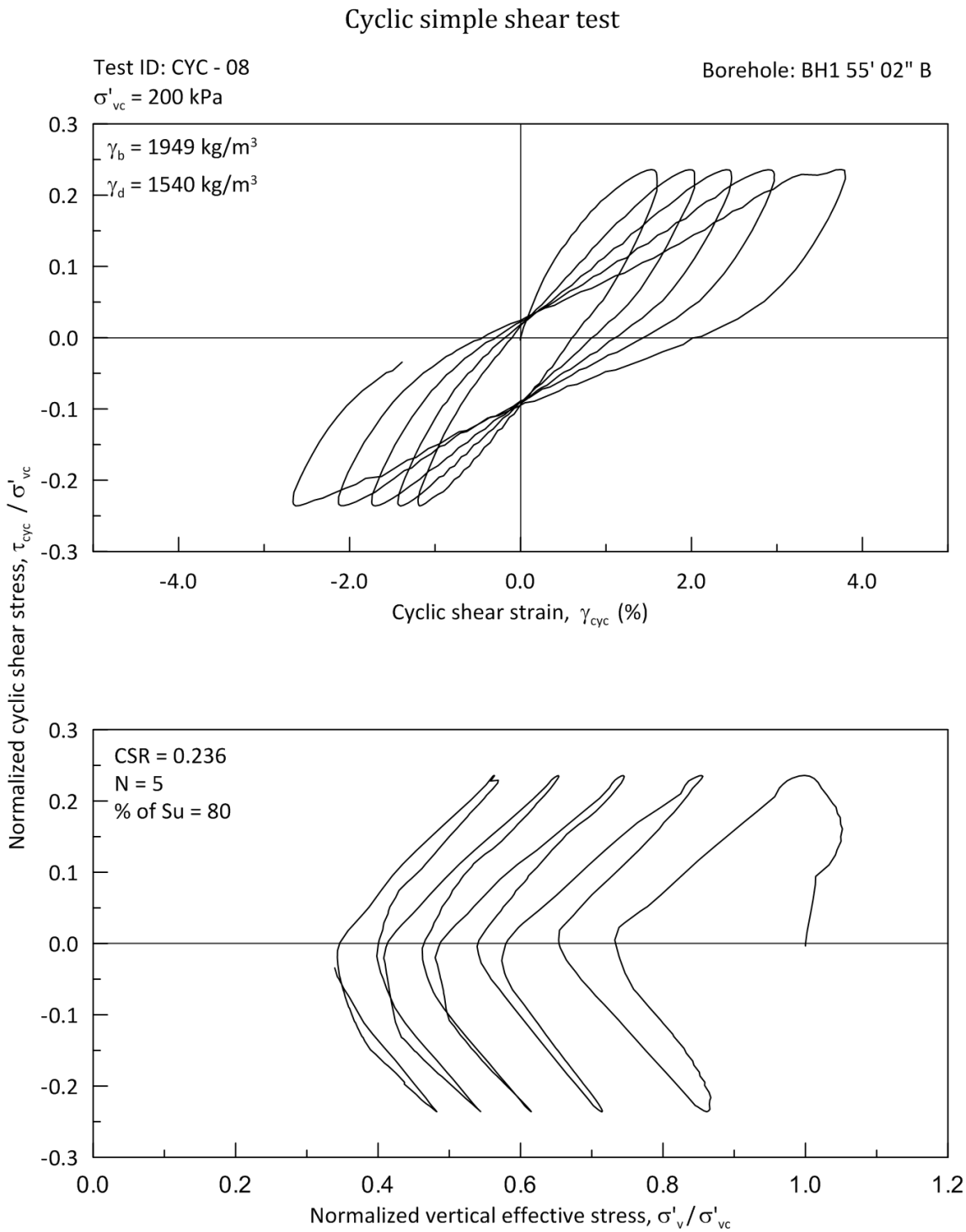

Fig 4.10 Normalized cyclic behaviour of clay subjected to τ_{cyc} = 47 kPa (a) Stress strain behaviour (b) Stress path

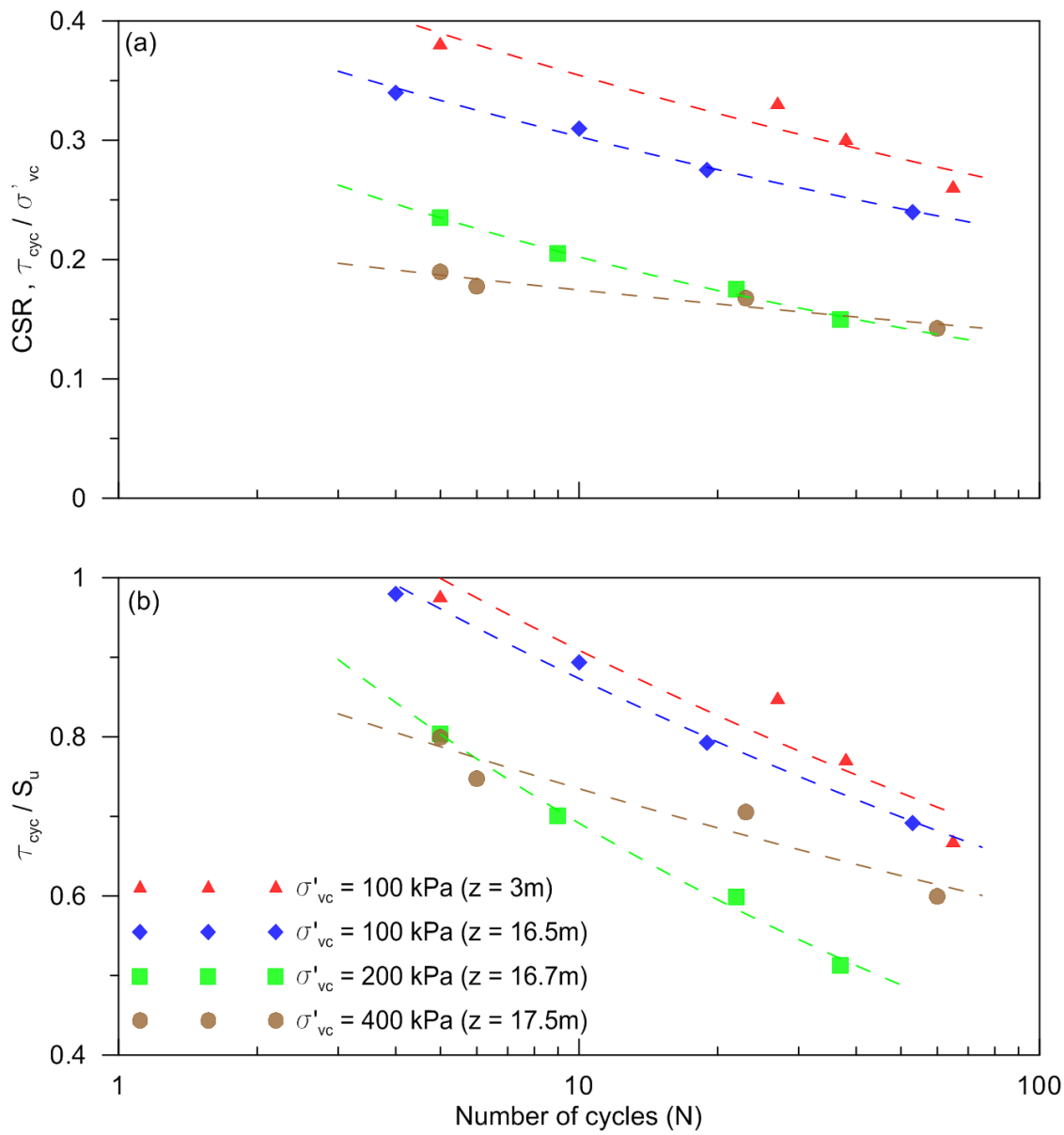

Fig 4.11 Variation of cyclic resistance under different σ'_{vc} levels (a) CSR with N (b) τ_{cyc}/S_u with N

However, cyclic resistance or strength under σ'_{vc} levels of 200 and 400 kPa do not show a clear trend. As noted previously, the clay specimens at this depth range have a pre consolidation pressure of about 200 kPa and thus the specimens tested under 200 and 400 kPa effective stress levels can be considered normally consolidated for practical purposes. Clearly the clay at 100 kPa is over consolidated, and it is postulated that the higher OCR is primarily responsible for the high cyclic resistance. Similar effects of over

consolidation in the cyclic behaviour of sands have been discussed in the literature (Sivathayalan et al, 2014)

A same general trend prevails when the data are evaluated in terms of cyclic stress ratios (CSR) as shown in Fig 4.11 (a). But, due to the differences in the normalized S_u/σ'_{vc} values the relative trends are somewhat different. CSR variation with N under σ'_{vc} levels of 100 and 200 kPa falls almost parallel to each other. However, the differences between 200 and 400 kPa at smaller number of cycles have disappeared, and the clay at 400 kPa exhibits higher strength when larger number of cycles were required for liquefaction. The slope of the cyclic resistance curve under σ'_{vc} of 400 kPa is comparatively smaller compared to the other two curves of this depth.

Overall Leda clay shows reducing cyclic strength with σ'_{vc} at least up to 200 kPa. This agrees with some observations made in literature (e.g. Bray & Sancio, 2006). The behaviour under σ'_{vc}values of 200 and 400 are not very obvious especially at equivalent τ_{cyc}/S_u levels. One possible reason might be the changes in the internal structure of the clay as the sensitive clay collapses under the applied 400 kPa consolidation stress which moves the clay well beyond its pre consolidation pressure. This would have an effect on the cyclic strength behaviour, as the clay would have been restructured, and in addition would be at a much higher density due to the increased compressibility (See $e - log\ (p')$ curve in Fig 3.14). Observations in the literature about the cyclic strength of clay in OC and NC states is contentious as well (e.g. Azzouz et al, 1989; Anderson et al, 1988) and this suggests that there are many factors that influence the behaviour of the clay as it straddles the pre consolidation pressure. Thus, a more detailed investigation about the

effects of OCR the role of pre consolidation pressure and the associated structural collapse in sensitive clays on cyclic strength behaviour of Leda clay would be required to provide a better understanding of this phenomenon.

4.3.2 Liquefaction susceptibility of Leda clay

The Leda clay specimens tested under simple shear were obtained at depths of 16 to 17.5m and 3m. Leda clay from 16 to 17.5 m depth has a liquid limit (LL) of about 25, and percent clay fraction of 35%. Based on Chinese criteria for liquefaction, this clay can be deemed non liquefiable as it does not satisfy the percent of clay sized particle rule. However, it satisfies the other two criterions as its Liquid limit (LL) is less than 35 and the natural moisture content is higher than 90% of its LL. Therefore, based on the other school of thought in literature (e.g. Seed et al, 2003), it is not possible to completely rule out the possibility of liquefaction of this Leda clay. In fact, the classification by Seed et al (2003) places the Leda clay from this depth in a zone (LL = 26 and PI = 12) where experimental investigation is important to determine potential liquefaction or strength loss (Fig 2.9).

On the other hand Leda clay from 3m depth has LL of 55 and PI of 30. Also, the percent of clay size particles are higher than 75%. Hence, this clay does not satisfy any rule of Chinese criteria. Even though Seed et al (2003) criterion disregards the percent clay size rule in Chinese criteria, the higher Atterberg limits places this clay in the 'non liquefiable zone' in this criteria as well (Fig 2.9). This is consistent with the cyclic test data provided in the previous section. Leda clay obtained from 3m depth showed higher resistance to cyclic shearing than the clay from 16 to 17.5 m depth.

The charts presented in Fig 4.11 provide some data to predict the liquefaction susceptibility of Leda clay. Based on this data, any cyclic loading event that produces cyclic stress levels lesser than 50% of its undrained strength would not produce liquefaction or large strains within 50 cycles. The lowest τ_{cyc}/S_u that can produce 'liquefaction' in less than 50 cycles is 0.5 under σ'_{vc} of 200 kPa (Fig 4.11). Similar ratios for $\sigma'_{vc}$of 100 and 400 kPa are about 0.7 and 0.6 respectively. These observations closely resembles with the cyclic threshold stress vales reported in literature for Eastern Canadian clays by various researchers such as Mitchell and King (1976) and Lefebvre et al (1989) even though these threshold values were related to 'failure'.

Furthermore, design earthquakes in Eastern Canada are commonly considered to be M = 6.5 to 7.0, hence it is appropriate to consider the cyclic resistance ratio CRR at 10 equivalent cycles (less than a magnitude of 7.5 event which is associated with 15 equivalent cycles). Hence in such circumstances it can be stated that the CSR values necessary to cause liquefaction (or Cyclic Resistance Ratio (CRR)) in Leda clay is approximately 0.3, 0.23 and 0.17 under σ'_{vc} values of 100, 200 and 400 kPa respectively (Fig 4.11). Therefore, the CRR values are relatively higher in Leda clays and they show a reducing trend with σ'_{vc}. This trend would support the observation of Bray & Sancio (2006) who observed severe liquefaction in the vicinity of large buildings than open sites as discussed in literature review.

4.3.3 Post cyclic behaviour

All the cyclic tests in this research program were followed by a strain controlled post cyclic shearing to large shear strains. These tests were intended to investigate the post cyclic behaviour of Leda clay. All cyclic tests were terminated at shear strain levels between 3.75% and 4% (except two tests due to some error in stop criterion definition) to evaluate the behaviour of clay without it being affected by the differences in pre-stain history.

The typical behaviour of a liquefied Leda clay specimen from 16.7 m depth, that was consolidated under 200 kPa and underwent a cyclic loading magnitude τ_{cyc} of 40.5 kPa (from test CYC – 07), is shown in Fig 4.12 (Details of post cyclic results are presented in Appendix B). The clay shows stiffness degradation at very small strains and a subsequent increase in secant modulus with strain (Fig 4.12 (a)) against the typical reduction in secant modulus in monotonic shearing. This behaviour is commonly observed in clayey soils following cyclic loading (e.g. Anderson, 1980). However, the behaviour is quite different from those seen in granular materials following cyclic loading that generated similar levels of shear strain. The initial effective stress state in granular soils is often close to zero following cyclic loading leading to liquefaction, and they deform with an essentially zero stiffness in the early stages of post cyclic loading (e.g., Sivathayalan & Yazdi, 2013). In contrast, the initial shear modulus in post cyclic loading in Leda clay is relatively high. But, it is obviously much lower compared to initial shear modulus observed in monotonic loading.

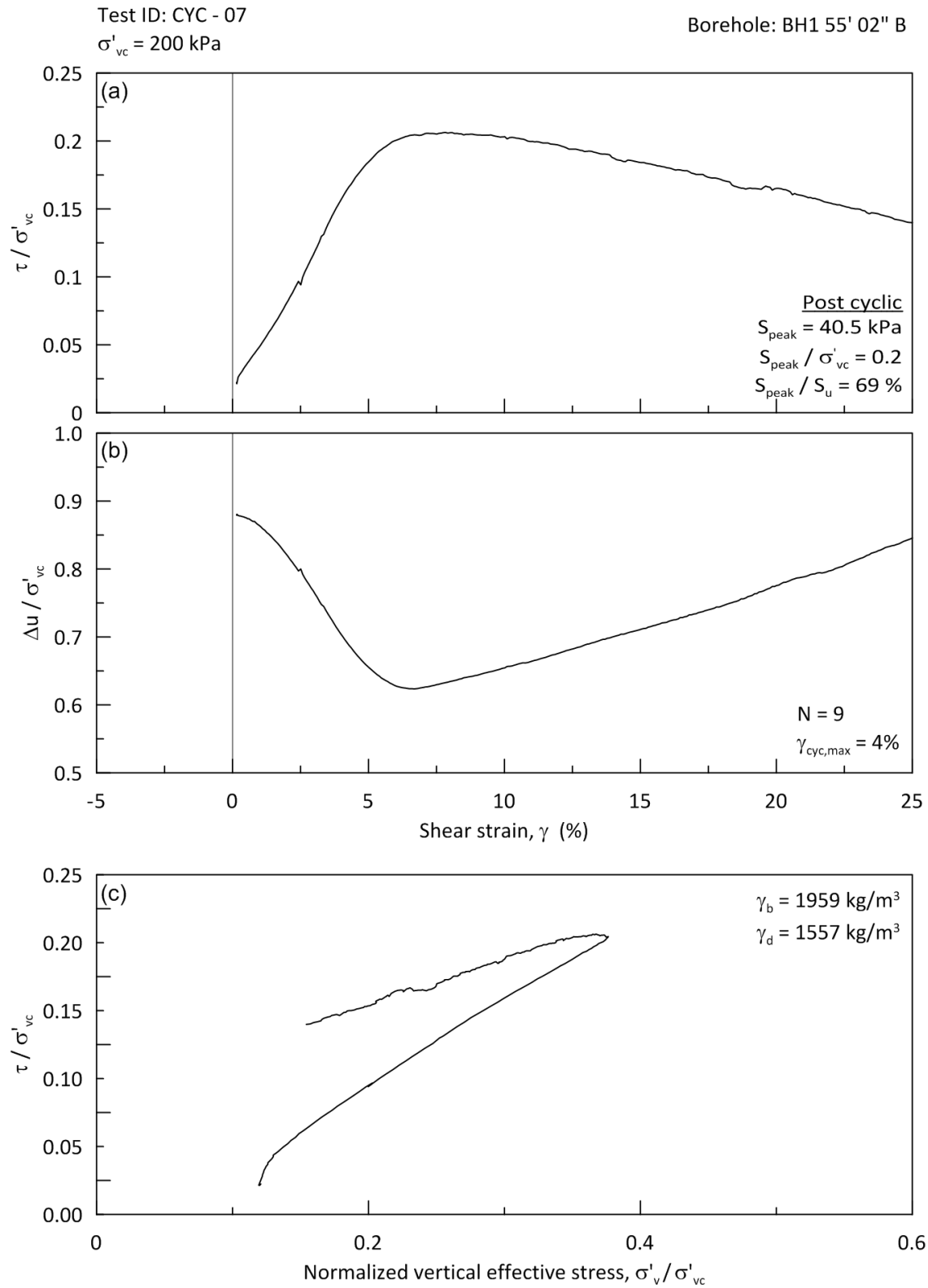

Fig 4.12 Normalized post cyclic (monotonic) behaviour of Leda clay under σ'_{vc} of 200 kPa (a) Stress strain behaviour (b) Variation of pore pressure (c) Stress path

Furthermore liquefied clay develops negative pore pressure (from its initial state at post cyclic shearing) as shown in Fig 4.12 (b, c), hence behaves similar to an over consolidated clay as observed by Anderson (1980) and Azzouz et al (1989). This behaviour possibly occurs due to "apparent over consolidation" produced due to cyclic shearing. Moreover, liquefied clay regains its strength and reaches a peak at larger strain values possibly due to dissipation of excess pore pressure.

Leda clay develops a considerable post cyclic strength; approximately 80%, 65% and 80% of their initial undrained strength (S_u) under σ'_{vc} values of 100, 200 and 400 kPa. The minimum strength gain observed in these series of tests is 55% of S_u under σ'_{vc} of 200 kPa and a maximum is 88% of S_u under σ'_{vc} of 100 kPa. Moreover, mostly clay that develops lesser pore pressure at the end of cyclic shearing (or at the start of post cyclic shearing) develops higher post cyclic strength during post cyclic shearing. For example, the specimen that developed a pore pressure of 75% of σ'_{vc} in a test conducted under σ'_{vc} of 200 kPa developed the largest post cyclic strength among the specimens tested at same stress level which developed significantly higher pore pressure (about 90% of σ'_{vc}) during cyclic shearing. Similar observations were made by Ho et al (2013) based on post cyclic investigation in Singapore marine clays.

4.4 Strength reduction / Sensitivity

One of the major challenges in sensitive clays is the strength reduction or sensitivity due to remoulding that occurs during cyclic and monotonic loading events. Therefore the vulnerability of strength reduction in Leda clay is a matter of significant interest to geotechnical engineers. Hence, the behaviour of Leda clay following loading events that cause small strain and vary large strain was investigated in this research program.

4.4.1 Effects of small strain cycles

A specific monotonic test (MON – 01 – A) was conducted to study the effects of few small strain amplitude cycles on subsequent monotonic shear behaviour. A specimen was obtained from the borehole sample (BH2 50'10") from which the specimen for test MON – 01 was obtained earlier. This specimen (MON – 01 – A) was also consolidated under σ'_{vc} of 100 kPa. It was given five small strain controlled quasi cycles with varying magnitudes of γ from 0.1% to 2% prior to monotonic shearing. This was intended to represent the loads anticipated in limited seismic shaking due to a small to moderate earthquake. The normalized stress strain behaviour and stress path of this specimen is compared in Fig 4.13 with that of the specimen sheared under the same σ'_{vc} without any initial cycles.

These two specimens show almost same stress strain behaviour and follow almost a same stress path as can be seen on Fig 4.13 (a) and (b). In addition, no significant strength loss is noticed in the specimen that had been given few small amplitude strain controlled cycles. This observation agrees with the understanding in the literature for Leda clays by various researchers (e.g. Mitchell & King, 1976; Raymond et al, 1979). Hence, it can be concluded that a few small-amplitude strain controlled loading cycles well below the strain amplitude at peak state do not produce any significant change in the in situ structure of Leda clay. As a result clay do not suffer any significant strength loss due to partial remoulding or change in its structure during subsequent shearing.

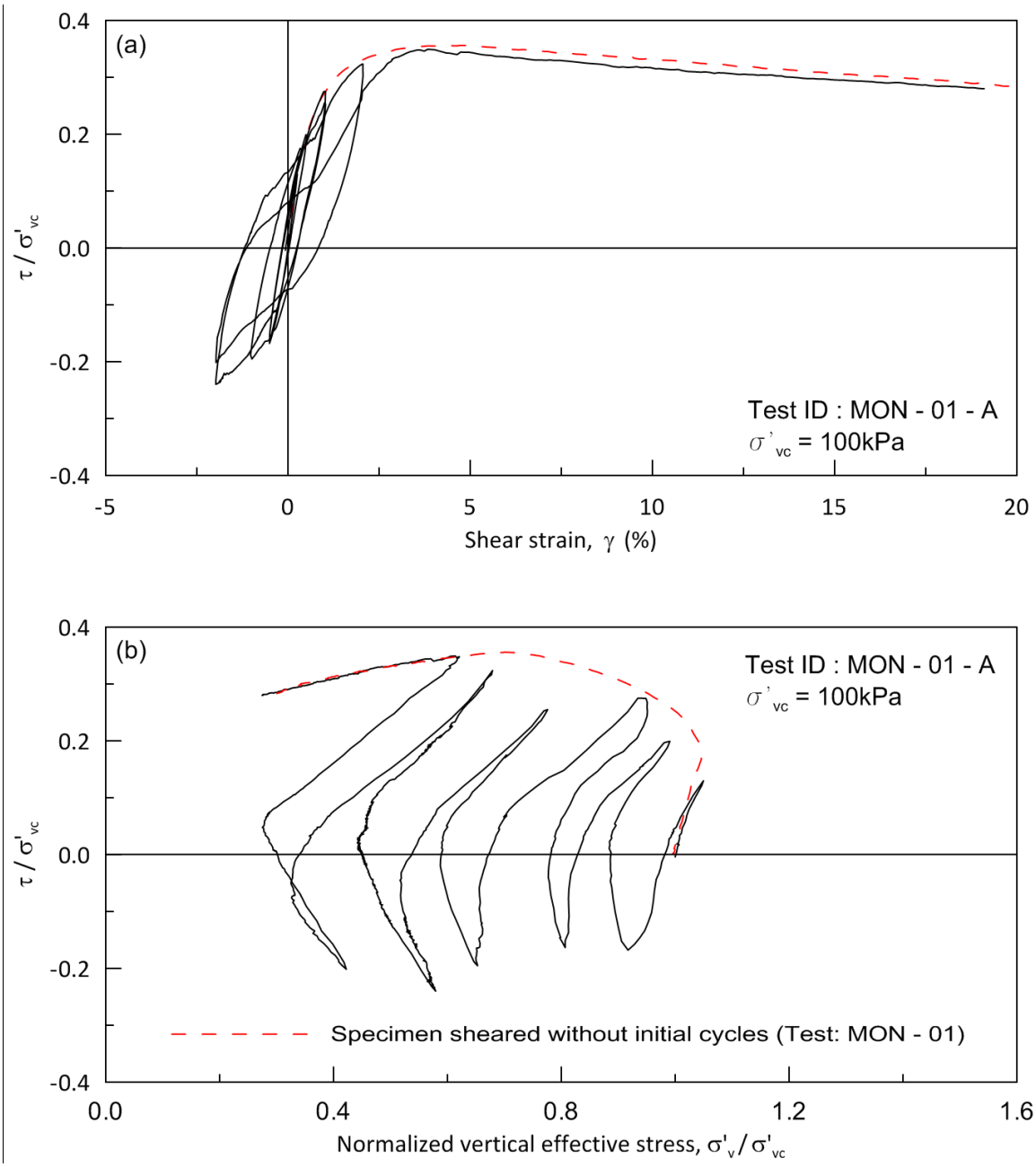

Fig 4.13 Normalized monotonic behaviour of Leda clay subjected to initial small strain cyclic shearing (a) Stress strain behaviour (b) Stress path

4.4.2 Effects of large strain cycles

This series of tests were intended to assess the behaviour of Leda clay after a relatively extreme loading event. A strong shaking during an earthquake or a heavy rainfall and the associated reduction in the effective stresses might trigger large deformation. But, due to the unloading pulses in the earthquake shaking or drainage of ground water and the associated dissipation of pore water pressure, the material might remain stable after the loading event. The potential implications of this loading history on the subsequent behaviour of the clay is of paramount importance, as any changes to the strength capacity of the material could adversely affect the perceived safety factors

This approach was intended to investigate the strength loss/sensitivity due to extreme loading that does not lead to a complete remoulding of the sensitive clay. A loading event that produces a very large strain (i.e. $\gamma = 20\%$) was simulated. Specimens were initially sheared towards the left (positive) direction. The applied shear stress was removed (unloading) upon reaching 20% strain. Subsequently, the specimens were again sheared, but towards right (negative direction) until reaching -20% strain. This loading sequence was repeated 2 to 3 times to simulate multiple loading events in the field. As a result the soil would have undergone few large quasi-cyclic shearing and leading to possibly some remoulding.

Fig 4.14 shows the behaviour of a specimen subjected to the above loading under σ'_{vc} of 200 kPa. It can be noticed that clay reached a peak shear stress of just over 50 kPa and was approaching a residual shear strength ($S_{\gamma=20\%}$) of 30.5 kPa during the initial loading. However, the material did not exhibit a peak state in subsequent loading, and the residual strength at subsequent cycles dropped to 13.5 and 10.7 kPa respectively. These results

indicate that Leda clay loses more than 50% of its initial residual strength during the second loading under this stress level and this strength loss continues further. Even though the material had not been fully remoulded during the loading cycles, it shows a progressive reduction in its load carrying capacity following loading events that impart large strains. This is a reflection of the gradual re-structuring (or destruction of the natural structure) in the clay as the loading progresses. The sensitivity of clay represents a complete loss of its natural structure, which leads to a catastrophic loss of strength.

Similar observations were made in other monotonic tests conducted on samples obtained from depths of 15m and 3m under various σ'_{vc} levels. The percentage strength loss from initial residual strength (S_r) and from undrained strength (S_u) is shown in Fig 4.15. The strength loss appears higher under large σ'_{vc} levels than the loss under small σ'_{vc} levels. Moreover strength reduction is much higher after first quasi-cycle than that of any other subsequent quasi-cycles.

These observations clearly suggest that cyclic strains / stresses of larger amplitudes are capable enough to produce significant strength loss due to gradual remoulding or restructuring of the in-situ structure of clay. As much as 80% of strength loss has been measured when the clay is pre-loaded to 20% shear strain. On the other hand, a few cycles of small strain amplitudes ($\gamma < 2\%$) would not produce significant remoulding or strength loss in Leda clay.

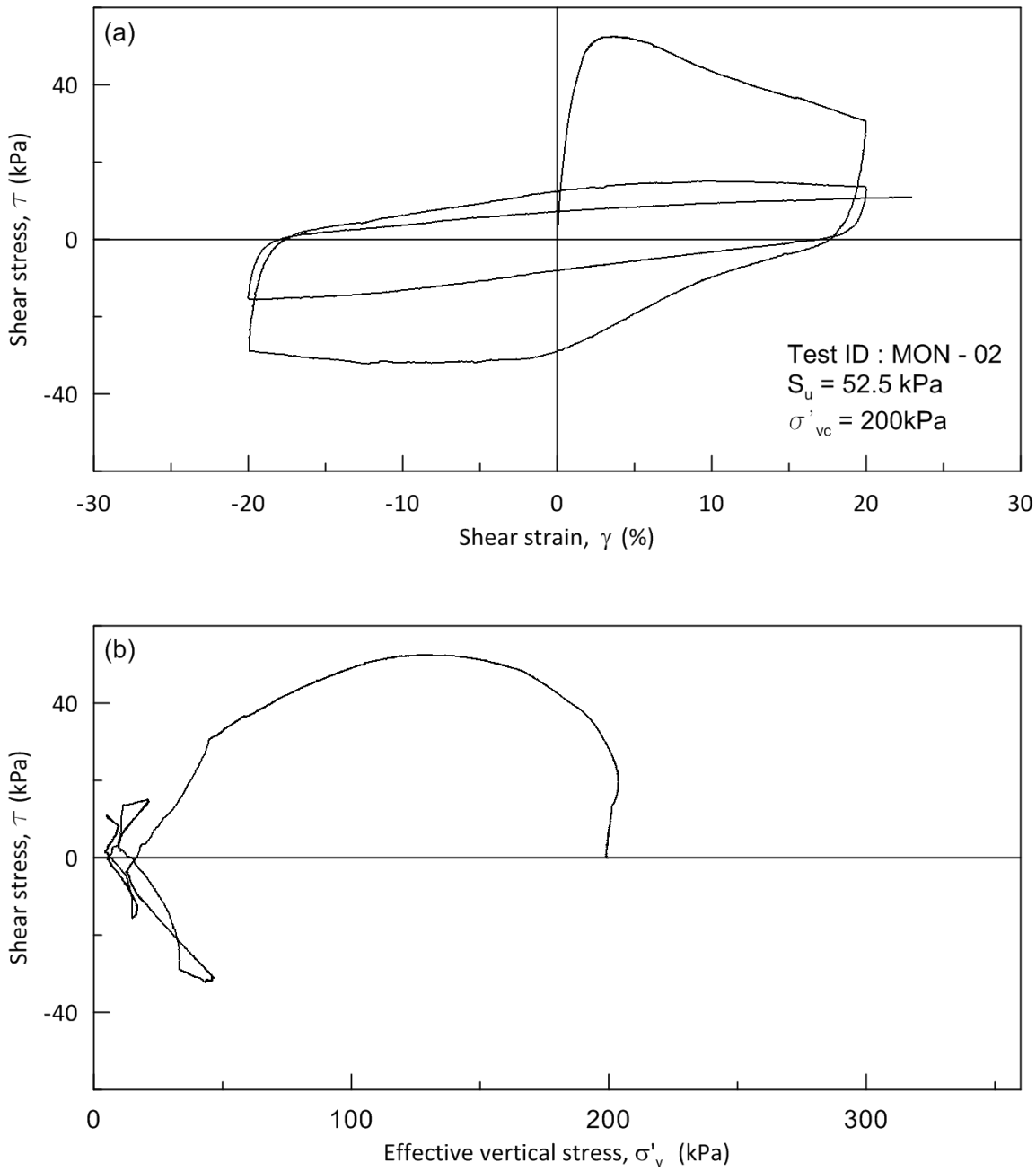

Fig 4.14 Typical behaviour of Leda clay during large strain quasi cycles (a) Stress-strain (b) Stress path

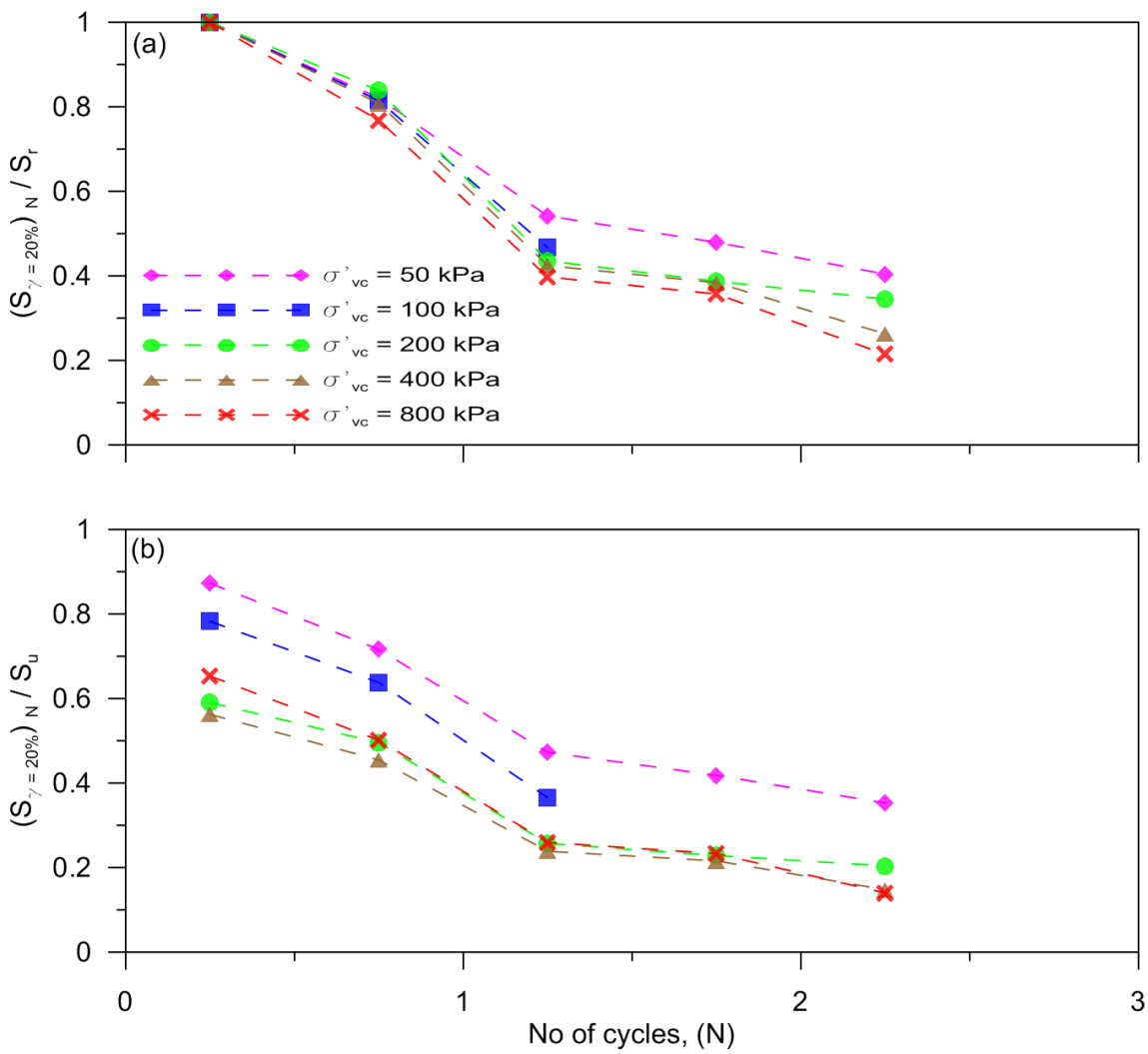

Fig 4.15 Variation of strength with number of large strain quasi-cycles in terms of (a) S_r (b) S_u

5 MODULUS AND DAMPING BEHAVIOUR OF LEDA CLAY

5.1 Introduction

One of the principal objectives of this research study is to propose modulus and damping curves for sensitive Leda clays of Eastern Canada. While it is recognized that test devices that are capable of measuring very low strains (e.g. resonant column device) are more suitable for this task, the availability of such devices is scarce and data interpretation is complex. In addition, this attempt would also enable an assessment of the potential effect, if any, of the loading mode on modulus reduction and damping characteristics. Modulus and damping are a functions of shear strain (γ) and can be obtained from stress or strain controlled cyclic loading. The modifications made to the existing data acquisition system and the use of highly sensitive low-range horizontal LVDT as mentioned in the experimental programme section facilitate testing at relatively low strain levels to enable the development of modulus reduction and damping data using this simple shear device.

This chapter provides details about various test methods used to obtain cyclic stress strain loops which can be used to determine modulus and damping values for the Ottawa area Leda clay that was tested. Modulus and damping curves obtained from the experiments conducted at different consolidation stress levels (ranging from 50 – 800 kPa) are presented. Finally a site-specific ground response analysis was performed using the modulus reduction and damping curves established for the site and the results are compared to those obtained using conventional practice.

5.2 Single and Multi stage quasi-cyclic testing

The comparison of monotonic behaviour of Leda clay with and without small strain cycles (Fig 4.13) discussed in the previous chapter confirmed that few initial small stress

or strain quasi-cycles do not affect the in- situ structure of Leda clay significantly. Hence, it is possible to apply few small stress or strain controlled cycles of varying stress or strain amplitude to a single specimen in staged-testing under various σ'_{vc} levels to generate the required data. In addition, this procedure avoids the uncertainty due to natural heterogeneity among different samples, and is cost effective for clays. Moreover, this procedure is important considering the available amount of undisturbed Leda Clay samples.

Two testing methods were used to conduct the quasi-cyclic tests. One procedure is using a new sample for each stage (i.e. each σ'_{vc} level) and applying various small stress or strain controlled cyclic loading to this specimen [Single stage tests]. The other procedure is re-testing the same specimen after progressively increasing the consolidation stress levels (σ'_{vc}) [Multi stage tests]. The largest amplitude of the cyclic stress or strain applied during each σ'_{vc} was limited to a lower value in the case of multi-stage tests.

5.2.1 Single stage tests

Single stage tests were adopted usually in strain controlled quasi-cyclic loading tests in this research programme. Initially a specimen was consolidated to the required σ'_{vc} level and was given quasi cyclic strain controlled shearing of varying amplitudes typically starting from γ_c = 0.01% up to large γ_c levels (e.g. γ_c = 5%). Following these small strain cycles, samples were monotonically sheared to failure (considered to be about 20% shear strain if no residual state was realized). This last stage of shearing enables comparison of the behaviour and stress paths of these specimens with those sheared monotonically (at similar or adjacent depths).

Fig 5.1 shows the behaviour of Leda clay obtained from a depth of 16.8 m (55' 10") during small strain quasi-cyclic and subsequent monotonic shearing under σ'_{vc} of 400 kPa from a single stage test. A gradual softening of the material as the amplitude of the strain cycles increase is noted. Similarly, the hysteresis loops remain fairly narrow until the strain amplitude exceeds about 1%, and the area of the loop becomes fairly large as the applied loading leads the material to its peak state, which is realized at a shear strain level of about 5%.

The monotonic behaviour of specimen obtained at a depth of 15 m (BH2 50' 10") which was monotonically sheared at the same σ'_{vc} level (without any initial cycles) is also shown by broken lines in the Fig 5.1. The specimen that underwent initial small strain quasi–cycles and the specimen sheared without initial cycles show a fairly similar behaviour both during small and large strains. Similar results were obtained for specimens consolidated to other stress levels as well. Atterberg limits conducted on samples obtained from the two locations yielded a liquid limit of 27 and 24, and plasticity index of 12 and 10 respectively. While these values are fairly similar, it is probable that part of the deviations noted in the behaviour might also be attributed to potential variations in the samples as these two samples were obtained at a depth difference of about 1m. Regardless, the response is considered quite similar for practical purposes, and this observation again confirms that 'small' strain controlled quasi-cyclic loading does not alter the subsequent behaviour of soil. As a result, it can be inferred that in-situ structure of Leda clay is reasonably preserved under small strain loading. Thus, each strain controlled cycle would reasonably provide the behaviour similar to shearing that specific amplitude (γ_c) cycle without any prior cycles.

5.2.2 Multi stage tests

In this testing approach, the same specimen was used to test under different consolidation stresses (σ'_{vc}). Each specimen was initially consolidated to a smaller σ'_{vc} (typically 50 kPa) and was subjected to stress or strain controlled quasi cyclic loading well within its peak state. Instead of continuing this shearing to larger stress or strain amplitudes as in the case of single stage test, the specimen was brought back to a state of zero shear strain and reconsolidated again to the next consolidation stress level (Stage 2). Again a similar quasi-cyclic shearing applied at that stress level or stage. And same procedure was continued to the following stages. Based on the previous results and discussion a small number of initial cycles within the peak state did not affect the structure of clay significantly, and therefore the staged-testing procedure was considered acceptable and practical.

For example, the post cyclic behaviour of specimen obtained at a depth of 3m is shown in Fig 5.2. This specimen was given small numbers of varying cyclic stress controlled quasi-cycles at each stage under σ'_{vc} levels of 50, 100, 200 and 400 kPa. After completing small quasi cyclic loading at final stage (σ'_{vc} of 400 kPa), this specimen sheared monotonically (post cyclic). The post cyclic behaviour again compared with the monotonic behaviour of a specimen obtained at same depth and sheared under same σ'_{vc} (MON – 06).

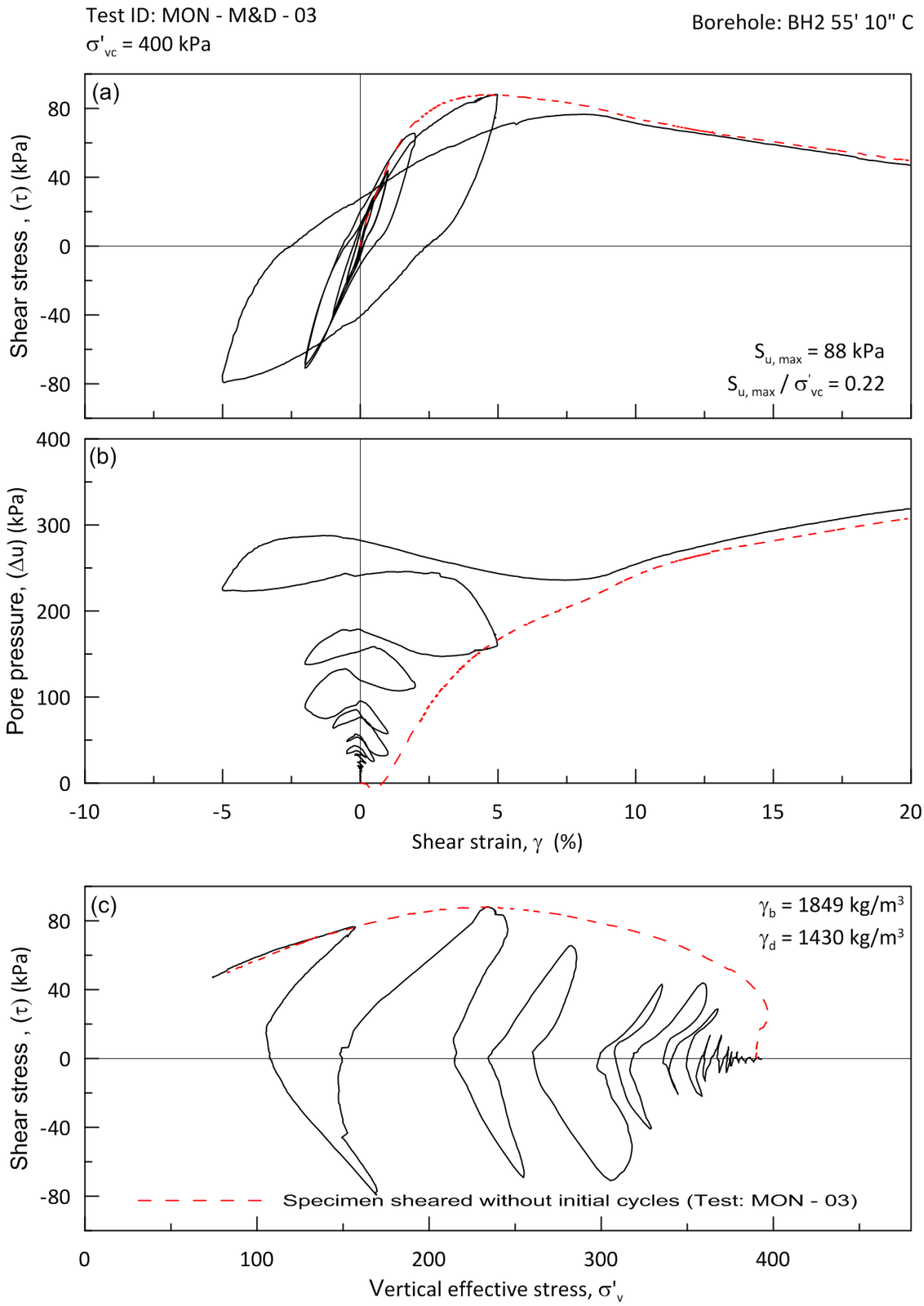

Fig 5.1 Typical behaviour of Ottawa Leda clay during quasi-cyclic and subsequent monotonic shearing (a) Stress strain behaviour (b) Stress path (c) Variation of pore pressure

The stress strain behaviour and stress path agrees extremely closely as shown in Fig 5.2. Even though the starting point of stress path of the specimen that had undergone multi stage testing is different due to excess pore pressure developed during quasi-cyclic shearing, it follows the same path afterwards. This observation again confirms and validates that multi stage testing process can be an effective and practical tool in this clay.

In this study, multi stage testing was adopted in stress controlled quasi–cyclic tests, since the sharing time is very small due to the higher loading frequency of stress controlled shearing. Therefore, employing a new specimen for each stage is not optimal. However, this procedure was successfully tested in strain controlled quasi-cyclic loading as well on Leda clay samples from other locations (e.g. Kinburn) during another project that is being carried out in CU simple shear device. Moreover, this procedure is very useful as it provides a significant amount of modulus and damping data from a single specimen. It thus saves cost and time significantly, and enables testing on identical material without issues related to natural heterogeneity.

5.3 Small stress / strain simple shear tests

Small amplitude stress and strain controlled quasi-cyclic simple shear tests were carried out in order to obtain modulus and damping characteristics of Leda clay in Ottawa area. These tests were carried out on samples obtained from two different zones: One towards the lower end of the clay layer at the site at approximate depths of 16 to 17m, and another near the surface at 3m depth.

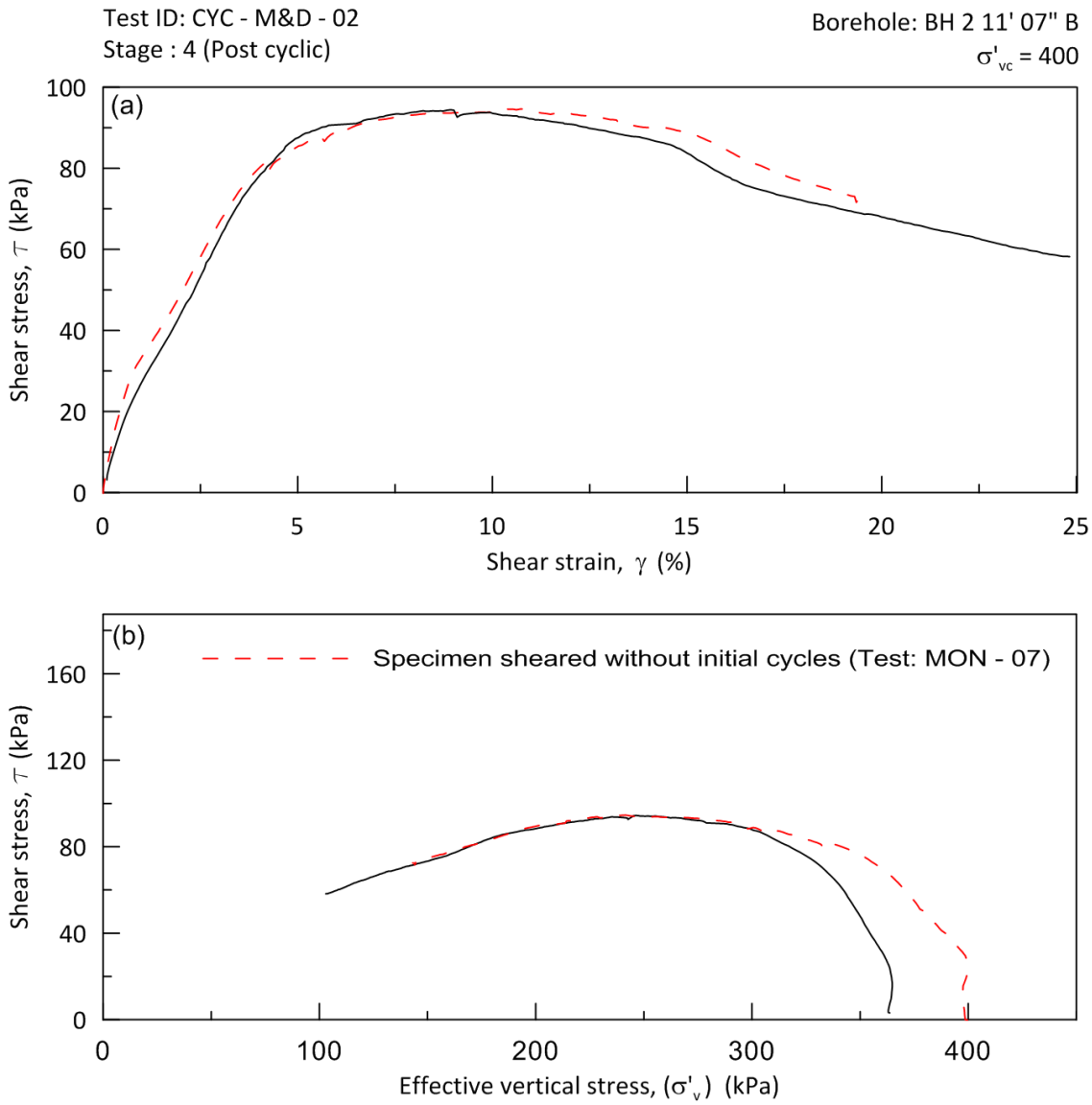

Fig 5.2 Post cyclic behaviour of Leda clay after multi stage stress controlled quasi-cyclic shearing (a) Stress – strain behaviour (b) Stress path

5.3.1 Stress controlled quasi-cyclic tests

The series of multi-stage stress controlled quasi-cyclic tests (CYC – M&D – 01 & 02) carried out were intended to yield modulus and damping values at very small strains levels such as 0.001%. These tests are essentially similar to the cyclic tests discussed in the previous chapter, but with small cyclic shear stress amplitudes that were changed after a certain number of load cycles. The small-magnitude cyclic shear stresses were

applied subsequently at each stage under each σ'_{vc} level. Specimen obtained at a depth around 17m was tested under 4 stages (or σ'_{vc} levels) of 100, 200, 400 and 800 kPa. Similarly, specimen obtained at 3m depth was tested under σ'_{vc} values of 50, 100, 200 and 400 kPa.

Each τ_{cyc} amplitude was applied for 4 numbers of cycles at a period (T) of 10 sec. Each τ_{cyc} amplitude produced a specific stress strain loop. These loops can be considered in order to calculate the secant shear modulus (G_s) and damping ratio (ξ) from the slope and the total area of the loop respectively. The calculated G_s and ξ values can be assigned to the cyclic strain amplitude (γ_c) of that specific loop. A typical quasi-cyclic stress controlled loading and response of the specimen consolidated under σ'_{vc} of 200 kPa is shown in Fig 5.3. The cyclic stress – strain loops produced from this loading is shown in Fig 5.4.

5.3.1.1 Issues encountered in stress controlled testing

The main objective of the stress controlled quasi-cyclic tests is to obtain stress-strain loops in order to obtain modulus and damping values at very small strain amplitudes. The strain controlled quasi-cyclic tests that can be carried out in the monotonic mode using the modifications made in the simple shear device could not achieve reliable measurements below shear strain level of 0.01%. Hence, the stress controlled tests were intended to provide modulus and damping data from very small (less than 0.01%) to moderate γ_c amplitudes. Then this data can be merged with the data obtained from strain controlled quasi cyclic tests that can be carried out at small, medium and large strain levels. This would produce complete modulus and damping ratio curves starting from very small γ_c amplitudes.

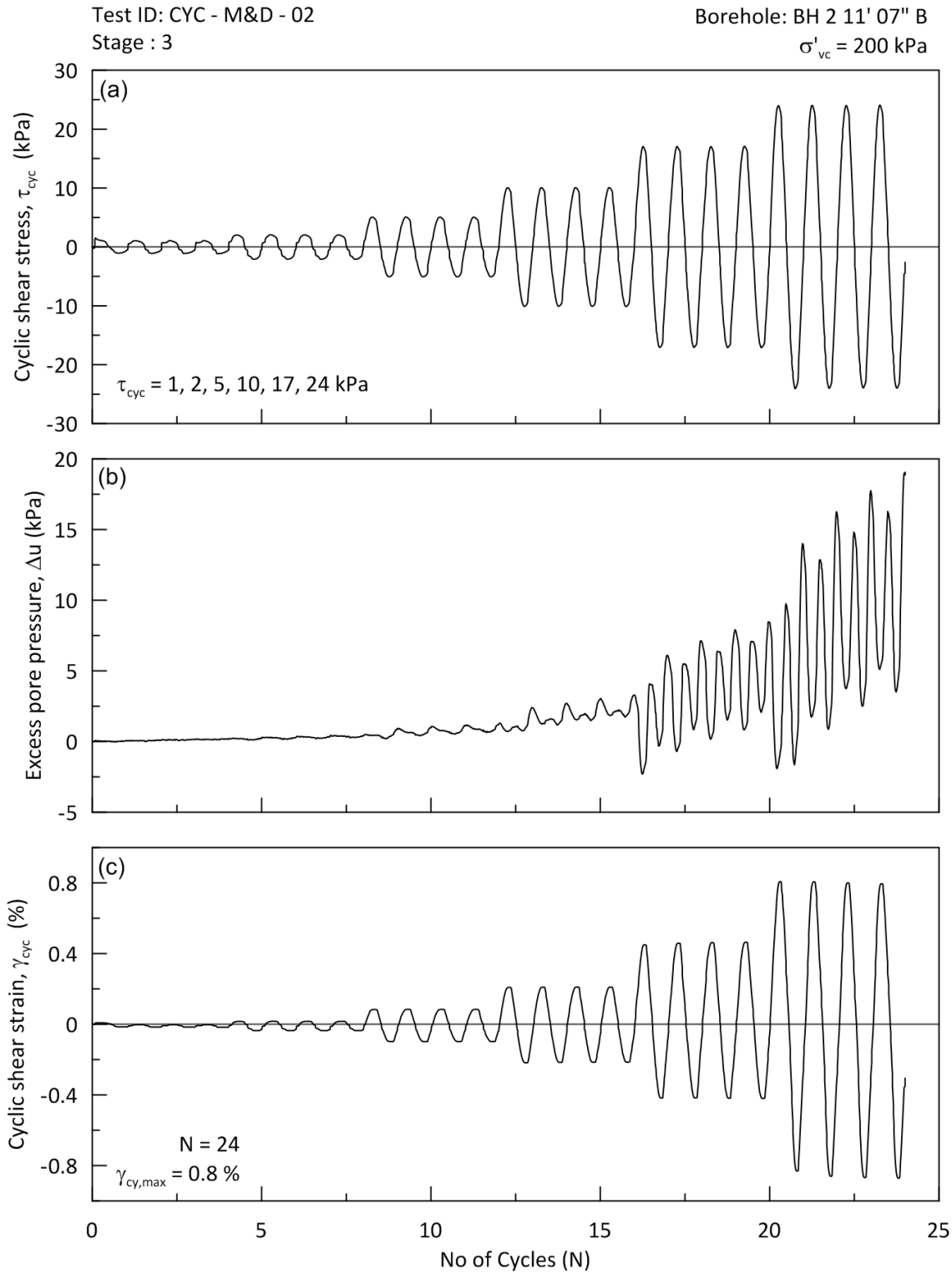

Fig 5.3 Typical behaviour of Leda clay during quasi-cyclic stress controlled loading (a) τ_{cyc} with N (b) Pore pressure with N (c) γ_{cyc} with N

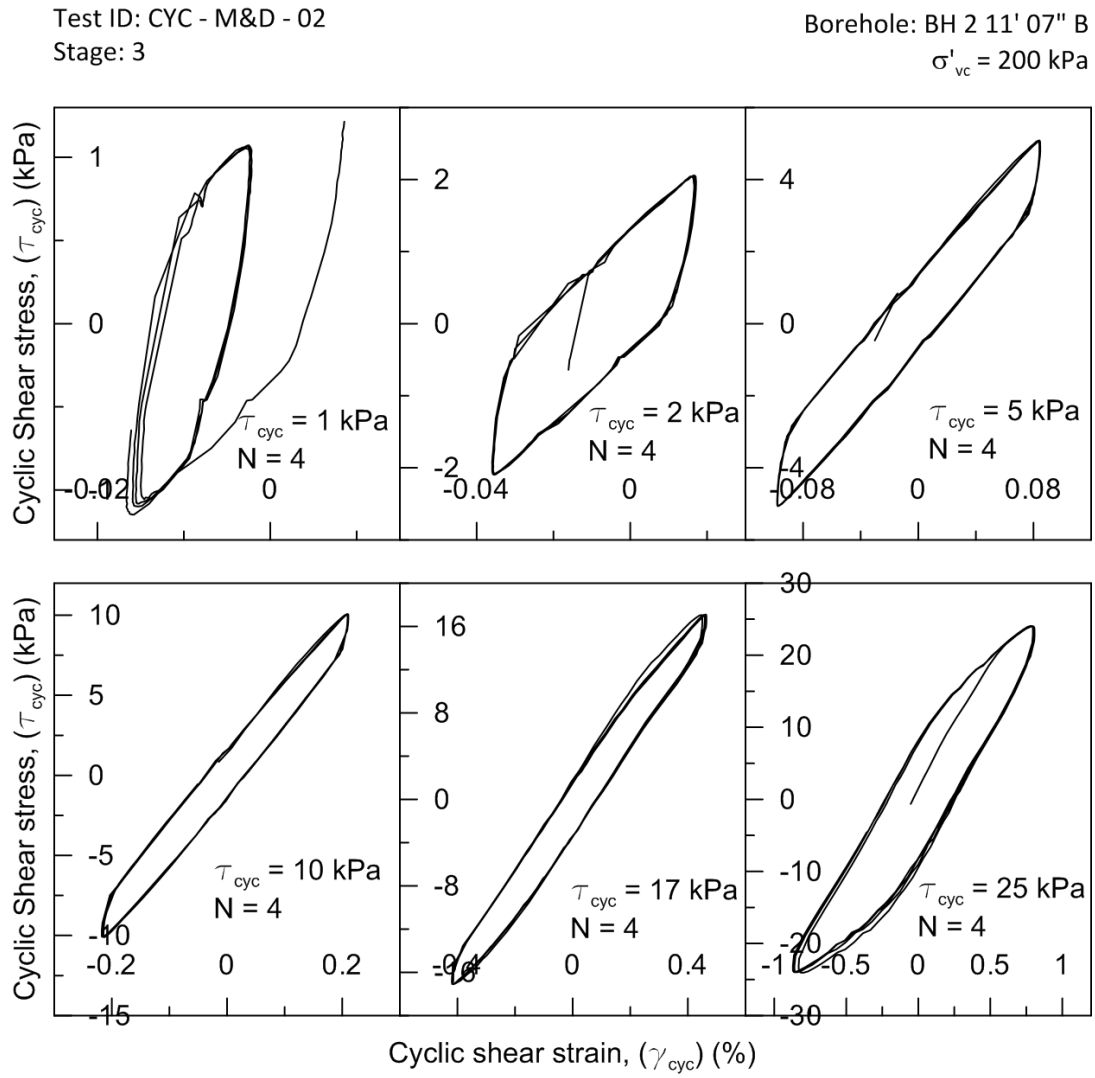

Fig 5.4 Cyclic stress-strain loops obtained from multi stage quasi-cyclic stress controlled shearing under σ'_{vc} = 200 kPa

However, the stress controlled cyclic loading which should vary as a uniform sine wave shape could not be achieved at very smaller stress levels as shown in Fig 5.5 (a). The data points were concentrated at the toe of the sine wave and the shape is not uniform. The initial 1 kPa shear stress amplitude resulted in a measured strain of 0.018% and the subsequent unloading to -1 kPa took the clay to 0.009% strain as shown in Fig5.5 (b) and (c). However, the subsequent stress cycles gave a consistent hysteresis loop between 0.009% and 0.013% (0.004% strain amplitude for ∓1 kPa shear stress). Clearly, unload-

reload loops are not expected to yield a modulus larger than the initial modulus, and therefore the initial reading has to be a false strain that was recorded possibly due to system compliance. The actual displacements in this range of strains are quite small (0.002% strain corresponds to $0.4\ \mu m$ of actual displacement for a 20mm high specimen –and the system has recorded about 8 data points within this $0.4\ \mu m$). Also, the small fluctuations of τ_{cyc} are quite significant at these smaller stress amplitudes. Due to these reasons, cyclic stress strain loops obtained at very small stress levels are not accurate and modulus and damping values obtained from these tests involved decisions based on engineering judgment. For examples, the modulus corresponding to the first cycle can be calculated by considering the actual loops without considering the erroneous initial reading.

Some possible reasons for this inaccuracy could be the friction in the loading arm or failure to balance pressures accurately across the pressure chambers of the loading mechanism at smaller stress levels. Even a very small 'play' in the fittings can also lead to 'large' errors. Furthermore, the current simple shear device was not designed for testing at such lower stress levels. A remodelling of the device would take an ample amount of time, as a result, stress controlled quasi-cyclic tests were not continued further in this research programme.

Data obtained from these tests were not taken into account in the proposed modulus and damping values. However, the modulus and damping data that was obtained at medium γ_c amplitudes agreed well with the results obtained from strain controlled quasi-cyclic shearing. Hence, it is possible to obtain accurate results at very smaller stress levels as well, by remodelling or fixing the mechanical issues of this device in future studies.

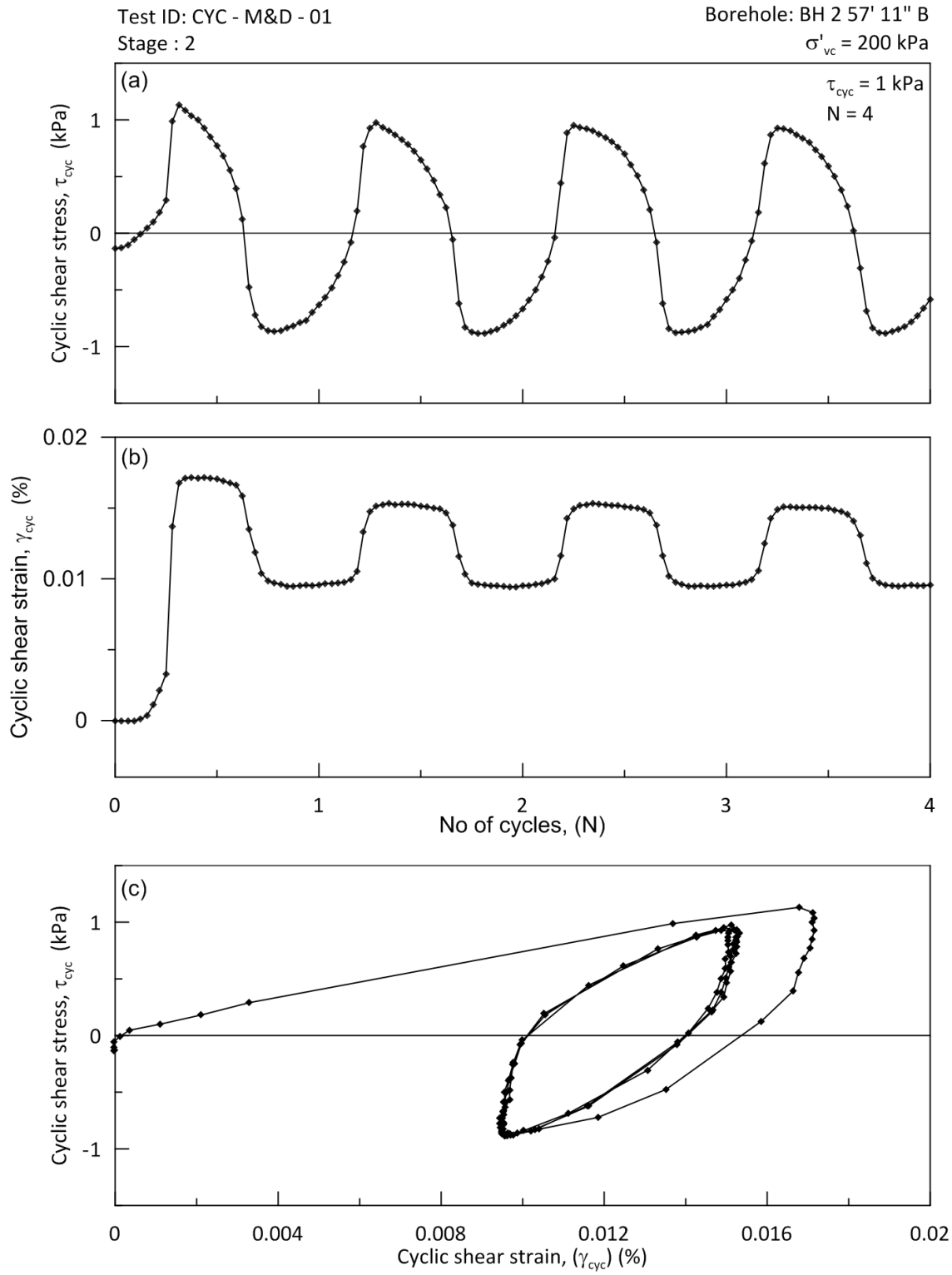

Fig 5.5 Inaccuracies in stress controlled shearing at small stress amplitudes (a) τ_{cyc} with N (b) γ_{cyc} with N (c) Cyclic stress – strain behaviour

5.3.2 Strain controlled quasi cyclic tests

The problems and inaccuracies encountered in the stress controlled quasi-cyclic loading made the investigation to be carried out only in the strain controlled mode. Hence, it was decided to carry out single stage strain controlled quasi-cyclic shearing at various strain amplitudes using the modifications made to the data acquisition programme. Stress–strain loops obtained from this testing were used to obtain Modulus and damping values at different cyclic shear strain amplitudes (γ_c)

Given the noise levels in the data acquisition system, it was decided to start strain controlled quasi–cyclic loading at a cyclic strain amplitude (γ_c) of 0.01%. A mechanically driven gear-motor is used to apply the strain controlled loading, and the play between the gear-teeth causes some errors at this strain level, but they are not very significant if the measured strain is at least 0.01%. Following the first 0.01% strain amplitude cycle, cycles with various γ_c amplitudes were applied to the same specimen.

Tests were carried out on samples obtained from the two depths mentioned earlier. Specimens from samples obtained at a depth of 16.7m (BH 2 55'10") were sheared under four different σ'_{vc} levels, 100, 200, 400 and 800 kPa. Similarly 3 specimens from a depth of 3m (BH 2 11' 07" B) were sheared under σ'_{vc}values of 50, 100 and 200 kPa. These variations in depth and σ'_{vc} levels are helpful in determining modulus and damping curves under different conditions. Also, they would provide some insights into the effects of consolidation stress level on the modulus and damping values of Ottawa area Leda clay.

Fig 5.1 shows the characteristics of a typical quasi-cyclic strain controlled shearing. This figure illustrates behaviour of a specimen obtained from a depth of 17 m and sheared to

multiple small strain amplitude cycles under σ'_{vc} of 400 kPa. Stress strain behaviour shown in Fig 5.1 (a) is similar to a cyclic stress strain response of clay. Shearing progressively increases the pore pressure, and as a result, stress path moves towards the origin as shown in Fig 5.1 (c).

Each of these small strain cycles obtained from these tests were considered to calculate secant modulus (G_s) (or "Modulus") and damping ratio (ξ) at their respective γ_c amplitudes. Specimens obtained from 17 m depth were sheared under γ_c amplitudes of 0.01%, 0.02%, 0.05%, 0.1%, 0.2%, 0.5%, 1%, 2% and 5% as shown in Fig 5.1 (a). Similarly specimens obtained from 3m depth were sheared from γ_c amplitude of 0.01% to 2%. All these strain amplitudes have practical significances. They provide data to construct modulus reduction and damping curves. Very small strain amplitudes are important to determine dynamic properties such as G_{max}. Modulus and damping values at large strain amplitudes have practical relevance as soil reaches failure at such large strain amplitudes during cyclic loading events.

Fig 5.6 shows the individual loops obtained from strain–controlled quasi-cyclic shearing carried out under σ'_{vc} of 400 kPa on specimen obtained from 17 m depth (test illustrated in Fig 5.1). A gradual softening and modulus degradation can be observed as the slope of these loops reduces with γ_c. Also area of the loops becomes fairly large with increasing $\gamma_c$due to the larger damping at higher γ_c amplitudes. Similar loops were obtained from all single stage quasi-cyclic strain controlled tests at different σ'_{vc} levels for specimens obtained from both depths.

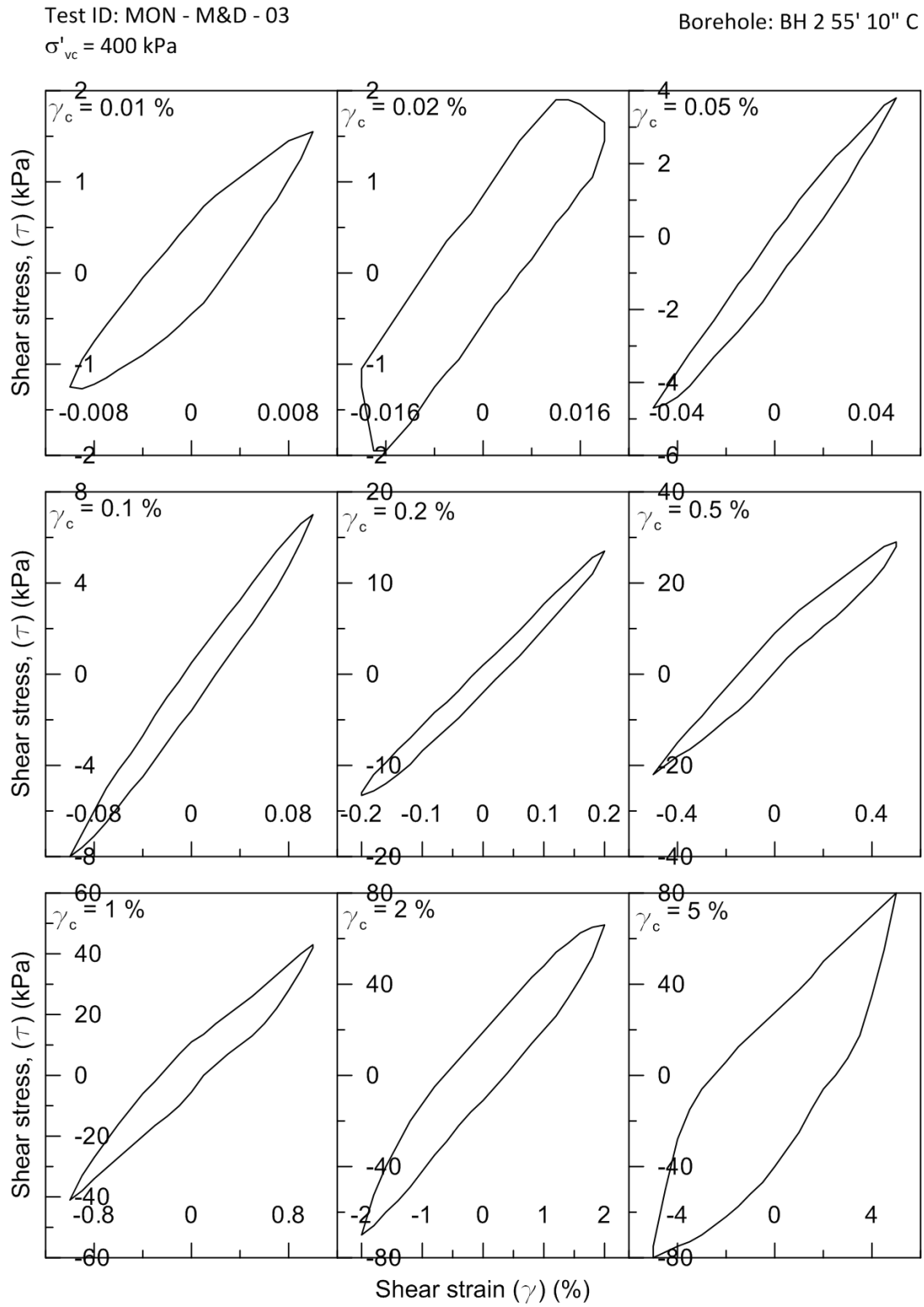

Fig 5.6 Stress-strain loops obtained from strain controlled quasi-cyclic shearing

5.3.2.1 Calculation of modulus

Secant Modulus (G_s) values were obtained from the slope of these loops and calculated modulus values for σ'_{vc} of 400 kPa at 17m depth from Fig 5.6 are shown in Fig. 5.7 (a). In addition, a 'Modulus reduction curve' can be obtained by normalizing G_s by maximum modulus value (G_{max}) which is measured at very small γ_c amplitudes. However, the smallest modulus value measured in this clay using strain controlled loading is at γ_c of 0.01%. Thus, each Modulus value (G_s), was normalized by G_s at γ_c of 0.01% [$G_s/(G_s)_{\gamma_c=0.01\%}$], in order to get normalized modulus curves. Technically, this is not the modulus reduction curve since the initial elastic shear modulus is not known. The value measured at 0.01% strain level would be somewhat lower than the elastic modulus (which is expected to be measured at a strain level of about 0.001% or less). Yet, these normalized modulus curves provide a way to compare similar curves from different depths and different σ'_{vc} levels. Normalized modulus curve for σ'_{vc} = 400 kPa is shown in Fig. 5.7 (b). In addition, once the actual elastic modulus is known, these curves can be corrected to yield the actual modulus reduction curves that can be used in ground response analysis.

5.3.2.2 Calculation of damping ratio

Similarly, Damping ratio (ξ) values were calculated based on the total area enclosed by the loop. However, the membrane and ram has a friction in the loading which provides some additional area to this loop. Thus, the area or energy released as a result of friction in the system and membrane was found by shearing the system without soil specimen. The energy released due to this friction at different γ_c amplitudes is shown in Fig 5.8.

This area was deducted from the total area of these loops in order to calculate the actual energy released by soil during the calculation of ξ.

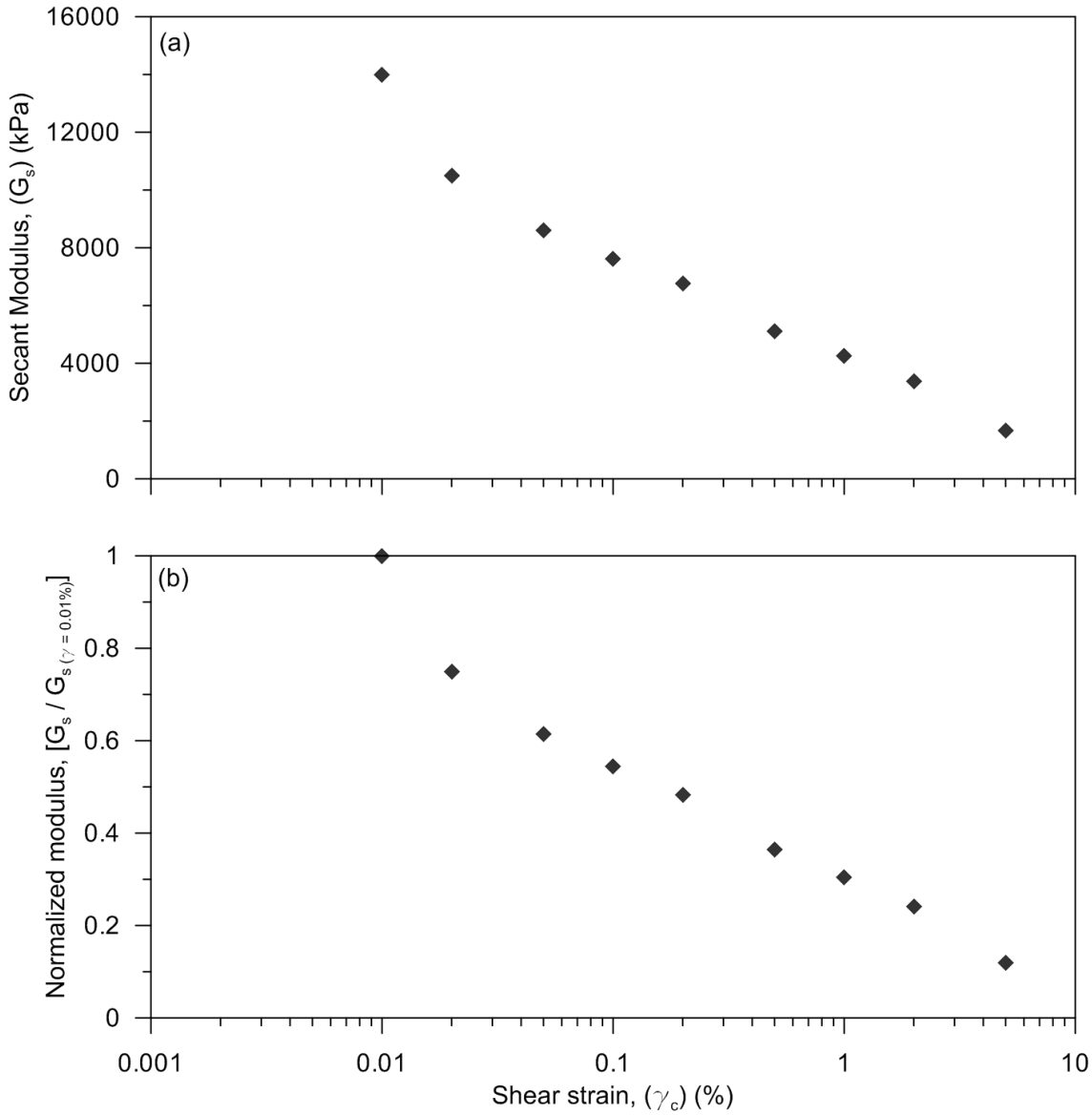

Fig 5.7 a) Modulus (b) Normalized modulus values at z = 17m under σ'_{vc} = 400 kPa

Furthermore, the mechanical components of stepper motor of the CU simple shear device lead to a small lag or delay when reversing the loading. As a result of this lag-time at the instances of stress reversal, and possibly due to the change in the electrical signals that trigger the direction change, the data becomes comparatively 'noisy' (at either extreme of

the stress-strain loops). This creates small inconsistencies in measured shear stress and strain during small strain cycles. Effect of these discrepancies is negligible in medium and large strain cycles (i.e. $\gamma_c \geq 0.1\%$), as they deal with relatively large stresses and strains. But these effects can be a distraction at very small strain values (i.e. $\gamma_c < 0.1\%$) and therefore, damping values at γ_c = 0.01, 0.02 and 0.05% were calculated by fitting a relatively smooth loop through the average readings instead of considering every data point. It is noted that the maximum deformation for a 0.01% loop is about 0.002mm, and more than 10-data points were recorded within that level of deformation. The measurement resolution in the newer-system is sufficient to record such small deformations while the motor is moving monotonically in one direction, but it has been noted that the noise level is significantly higher at the instant of stress reversal. Noise levels fall back to normal-levels once the gear-drive engages in the other direction and starts to move. Thus, a few of the outlying points at the edges of the loop, which were clearly noise induced and not due to soil response (e.g., readings that show an increase in strain while the motor has reversed and the loading should lead to a decrease in shear strain) were omitted while constructing the loop.

As a result, the damping values reported in this research programme for the small strain amplitudes of γ_c = 0.01, 0.02 and 0.05% are only approximate. Furthermore, Vucetic et al (1998) showed the importance of viscous creep in clays which gives roundness to the clay at small strain values. Thus, a more accurate device such as resonant column can be employed to obtain modulus values accurately at very small γ_c values. The damping ratio (ξ) values found at different γ_c values from respective loops shown in Fig 5.6 for

specimen sheared under σ'_{vc} of 400 kPa are shown in Fig 5.9. Similarly, damping ratio values under all the other conditions were also obtained.

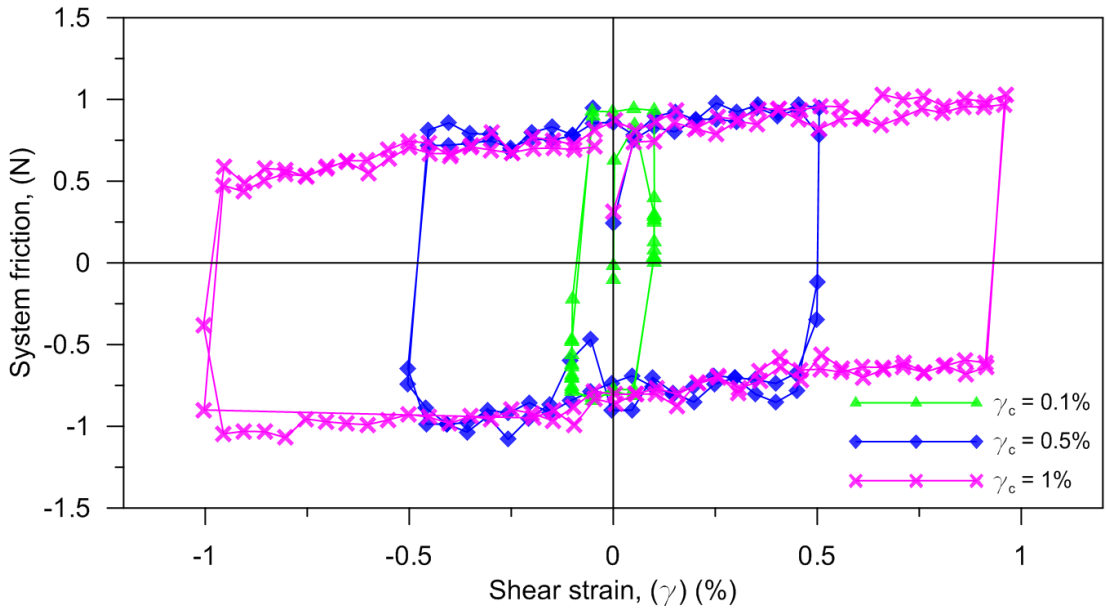

Fig 5.8 Friction measured in the system at different quasi-cyclic strain amplitudes

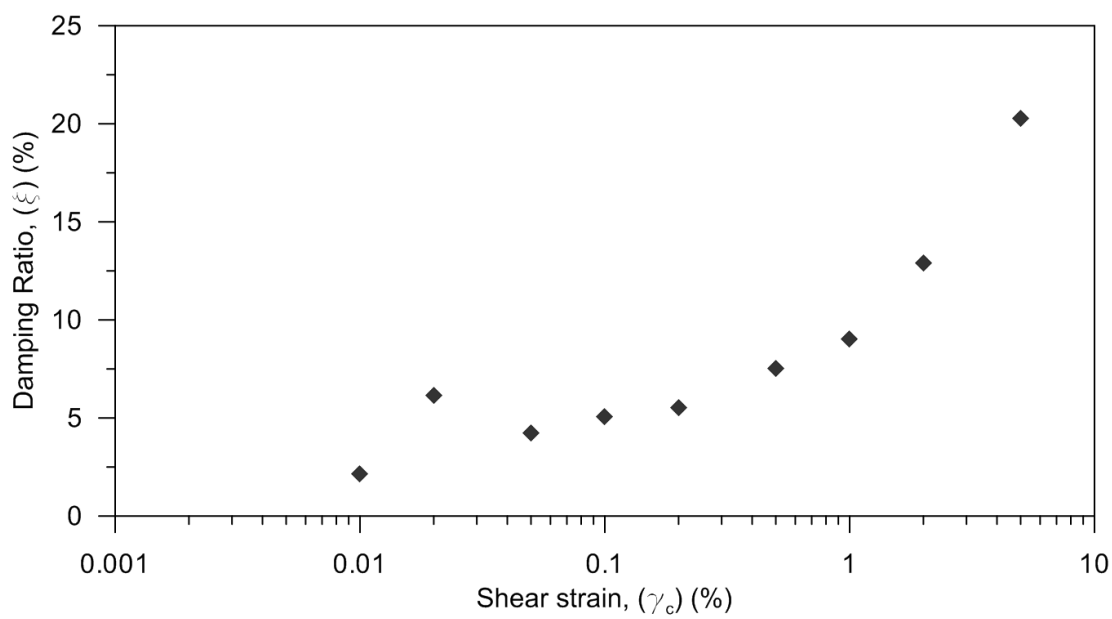

Fig 5.9 Damping ratio values at z = 17m under σ'_{vc} = 400 kPa

5.4 Modulus and damping

5.4.1 Normalized modulus and damping ratio

The modulus (G_s), normalized modulus [$G_s/(G_s)_{\gamma_c=0.01\%}$] and damping ratio ($\xi$) curves established under all considered σ'_{vc} (σ'_{vc}= 100, 200, 400 & 800 kPa) levels for Ottawa area Leda clay at 17 m depth are shown in Figures 5.10 and 5.11. Modulus values (G_s) decrease with increasing γ_c at each stress level. Also, modulus values (G_s) at each γ_c (or Modulus curve) increases with vertical effective stress level (σ'_{vc}). But the trends of these curves are similar as seen in Fig 5.10 (a). The difference between modulus values at $\sigma'_{vc}$of 100 and 200 kPa are higher than the difference between any other successive σ'_{vc} levels tested at this depth. However, when these G_s values are normalized as mentioned before, the resulting curves of different σ'_{vc} levels converge within a fairly narrow band as shown in Fig 5.10 (b). Even though, they are not the exact modulus reduction curves, this converging behaviour agrees with the findings of Vucetic and Dobry (1991) among others in that modulus reduction curves of clays do not vary significantly with σ'_{vc} level.

These modulus and normalized modulus curves indicate that modulus values of Ottawa area Leda clay at 17 m depth starts to reduce significantly at γ_c range between 0.01% and 0.05%. This modulus reduction continues further, but with a milder slope with γ_c at all tested σ'_{vc} levels. Normalized modulus values under different σ'_{vc} levels are approximately similar up to about γ_c of 0.5%. However, at larger γ_c amplitudes there is some scatter in normalized modulus curves especially at large σ'_{vc} values (i.e. σ'_{vc} = 400 and 800 kPa). This behaviour shows that modulus degradation under higher σ'_{vc} levels is smaller than that of modulus degradation under smaller σ'_{vc} levels at large amplitudes of

γ_c. Also, the effects of the loading history on soil behaviour will become more prominent as the strains increase (closer to 1% shear strain is considered 'large' in this context).

Damping ratio curves show relatively considerable scatter especially at large γ_c amplitudes. The trend of these curves clearly illustrates that restructured Leda clay due to normal consolidation stresses under natural state shows significant lower damping at larger strain amplitudes than intact or over consolidated clays. However, damping values almost follow a single line or falls within a very narrow band below γ_c of 0.5%. Moreover, the damping values at small strain levels (γ_c = 0.01, 0.02 and 0.05%) are not very accurate due to the inconsistencies and approximations discussed earlier; however, they fit considerably well with the pattern of damping curves noted in the literature for clays.

Similar curves for Leda clay obtained at shallower (3m) depth are shown in Figures 5.12 and 5.13. The general trend of modulus and normalized modulus curves are similar to that of curves obtained at a depth of 17m. However, the difference between modulus values at subsequent σ'_{vc} values is bit lower in clays obtained at 3 m depth. Moreover, normalized modulus values falls within a very narrow band (more or less a single line). This could be partly due to the fact that this clay was not tested at higher σ'_{vc} levels; since it is located at a shallower depth, behaviour at higher σ'_{vc} levels is not of practical interest. Furthermore, damping ratio curves show some scatter, but the general trend is similar to the previous data and of typical clays.

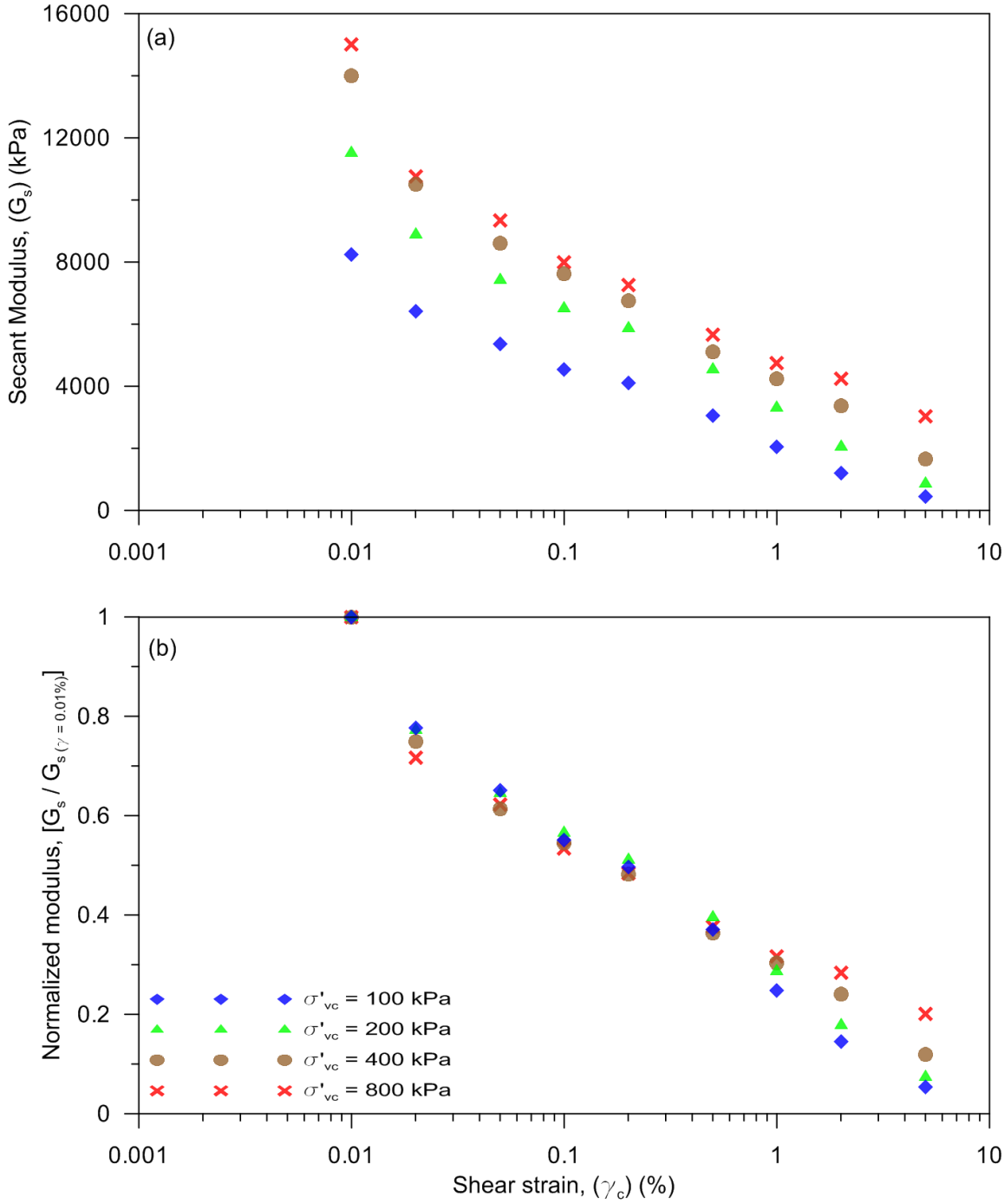

Fig 5.10 (a) Modulus (b) Normalized modulus values at z = 17m under different σ'_{vc} levels

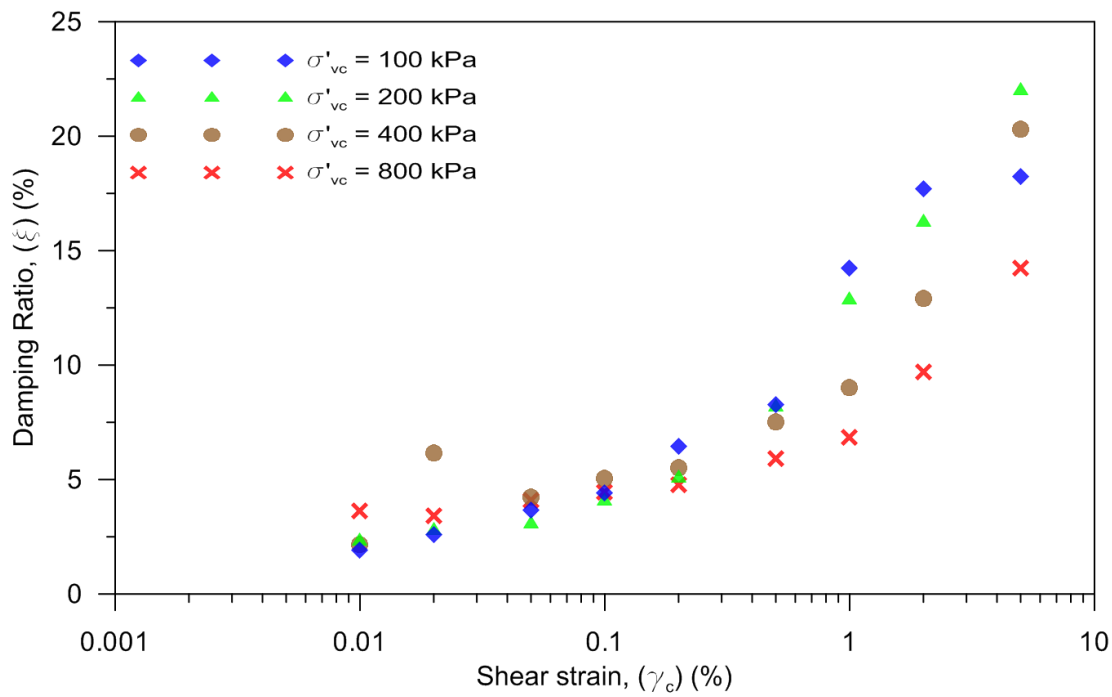

Fig 5.11 Damping Ratio at z = 17m under different σ'_{vc} levels

5.4.2 Variation of modulus and damping with depth

The approximate variation of modulus and damping values of Ottawa area Leda clay can be examined by comparing these values obtained at 3m and 17m depths. Modulus and damping values of Leda clay were obtained at these two depths under σ'_{vc} values of 100 and 200 kPa on common. Hence these values are compared at these equivalent stress levels in Figures 5.14 and 5.15.

Under σ'_{vc} of 100 kPa, there is no significant variation in G_s values at equal γ_c amplitudes except at very small strain values. At smaller strain amplitudes modulus values are bit higher in clay from shallower depth. Moreover, modulus curve of clay from 3m depth locates bit lower than that of clay from 17 m depth under σ'_{vc} of 200 kPa especially in medium γ_c values. The plasticity index (PI) of Leda clay at 3m is approximately 30 and PI of clay at 17m depth is about 12, and this is expected to cause some variations in the

modulus and damping characteristics. Also, it is contrary to the general trend in clays that modulus increases with PI.

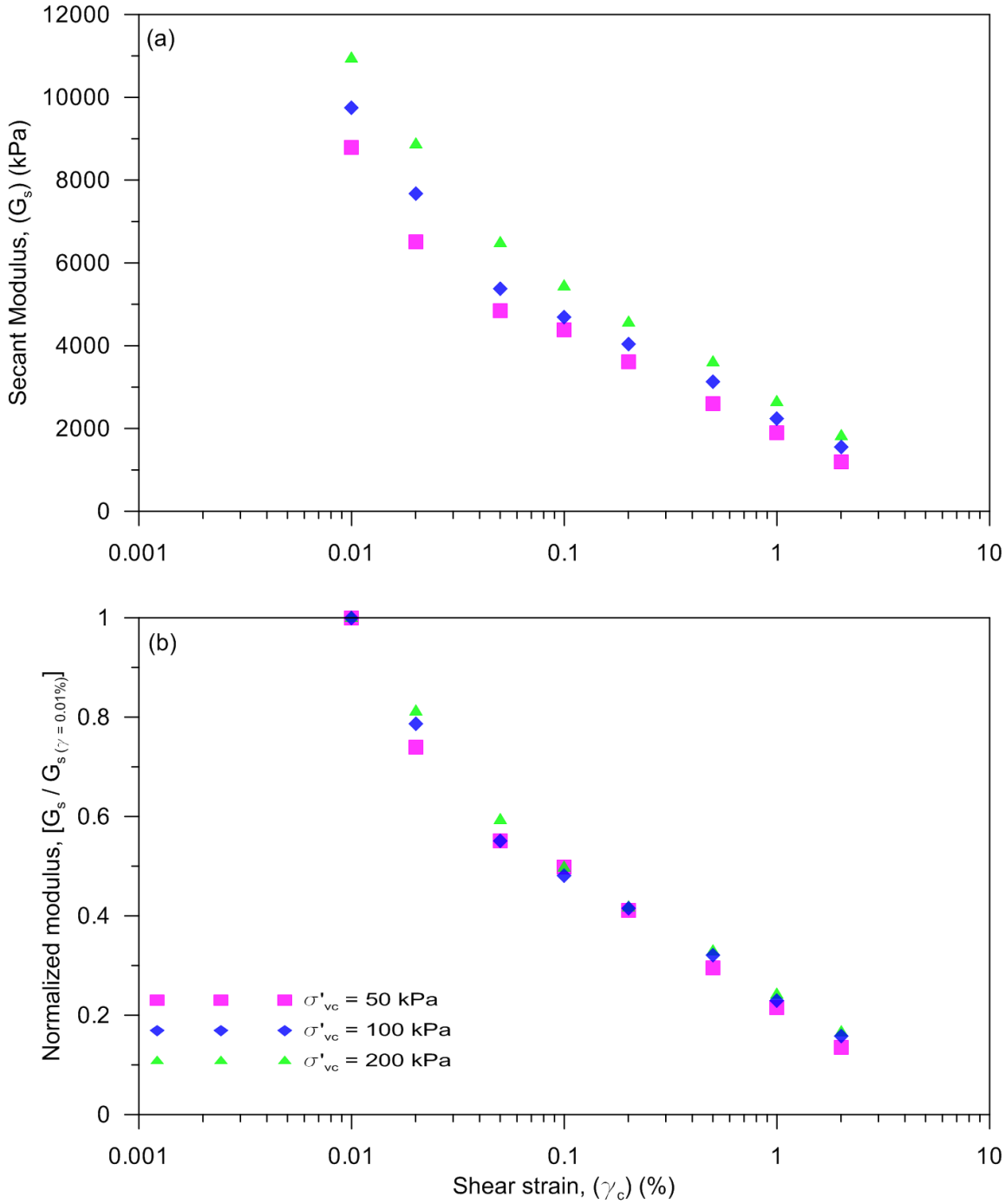

Fig 5.12 (a) Modulus (b) Normalized modulus values at z = 3m under different σ'_{vc} levels

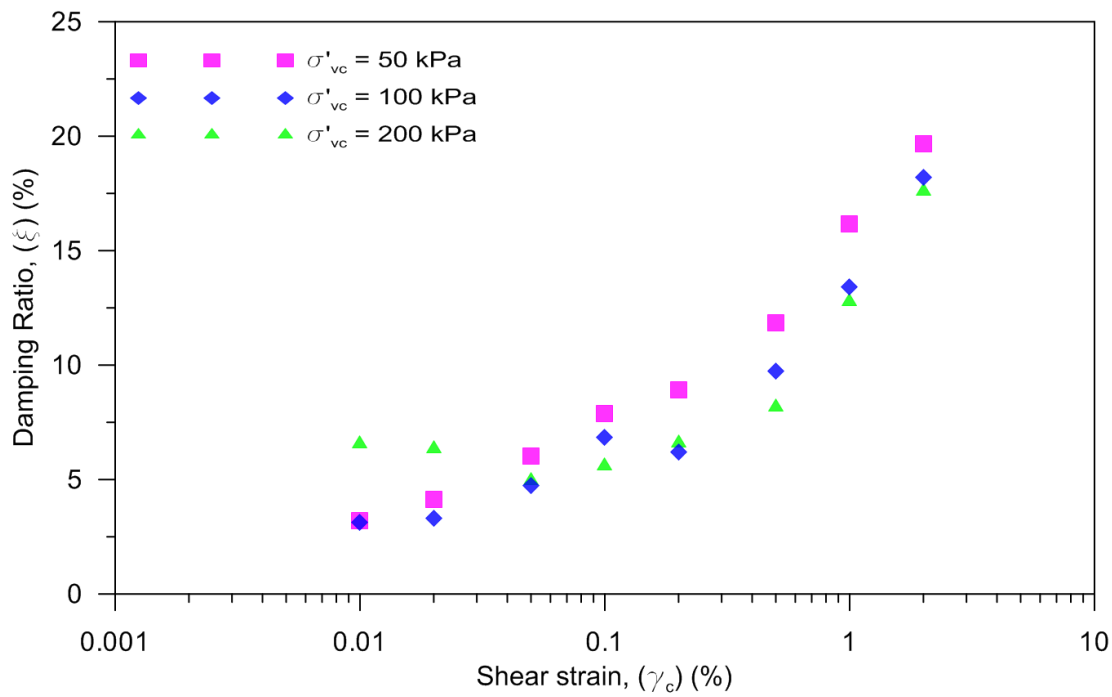

Fig 5.13 Damping Ratio at z = 3m under different σ'_{vc} levels

But the tested specimens had different OCR values and that would be partly responsible for the measured differences. Furthermore, bulk density of clays from 17m and 3m depths after consolidation under σ'_{vc} of 200 kPa are approximately 1825 kg/m^3 and 1550 kg/m^3 respectively. The differences in these bulk density values could also be partly responsible for this behaviour. Moreover, clay size particles at 3m depth (75%) are considerably higher than that at 17 m depth (35%), hence the effects in modulus and damping based on PI may have been minimized due to the severe variations in the gradation as well. However, effects of the amount of clay size particles on modulus and damping is not clearly documented in literature.

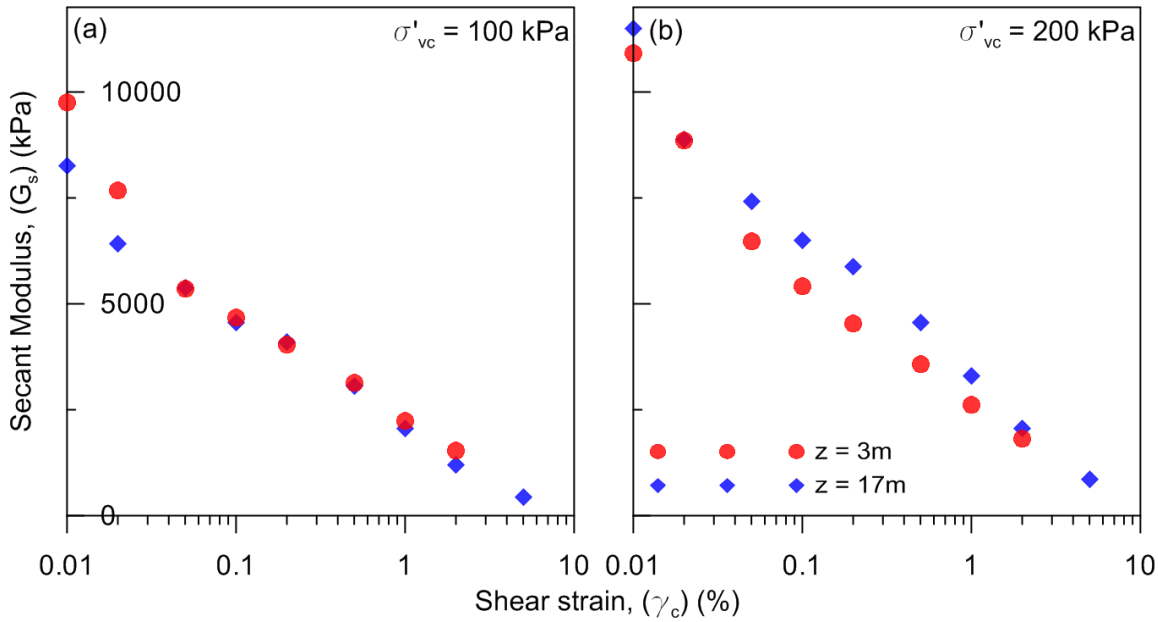

Fig 5.14 Comparison of modulus values at various depths

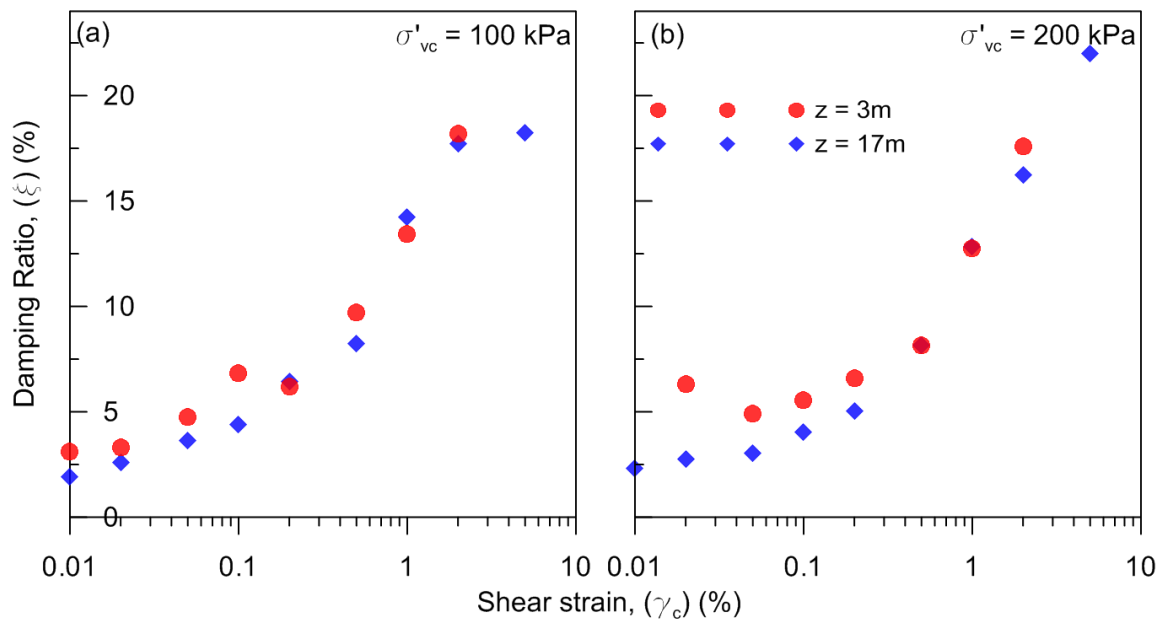

Fig 5.15 Comparison of damping ratio at various depths

Damping curves also do not show significant difference with depth as can be seen from Fig 4.15. Damping curves of Leda clay from 3 m depth locates bit higher than that of clay obtained from 17m at both σ'_{vc} levels. Hence, these observations suggest that sensitive

clay may not necessarily follow the modulus and damping curves used for typical clays based on PI values.

5.4.3 Effects of frequency

The effects of frequency of loading on the modulus and damping values of Leda clay can be investigated by comparing the modulus and damping values obtained from strain controlled and stress controlled quasi cyclic shearing. The strain controlled shearing is very slow (5% / hr) and stress controlled loading is applied at a very high frequency in the simple shear device. Hence, the comparison of modulus and damping values from these two modes of shearing would provide the effects of frequency on modulus and damping curves.

Modulus values observed from both test modes at medium γ_c amplitudes under σ'_{vc} of 200 kPa on samples from both depths are compared in Fig 5.16. In these curves the values obtained from both modes agree well and falls on a same curve. Similar trend was observed on samples tested under other σ'_{vc} levels as well. Thus, it can be concluded that frequency of loading do not affect modulus values of Leda clay significantly. Similar comparisons on damping values were not made in this study due to the inaccuracies in stress controlled cyclic loading mode discussed earlier.

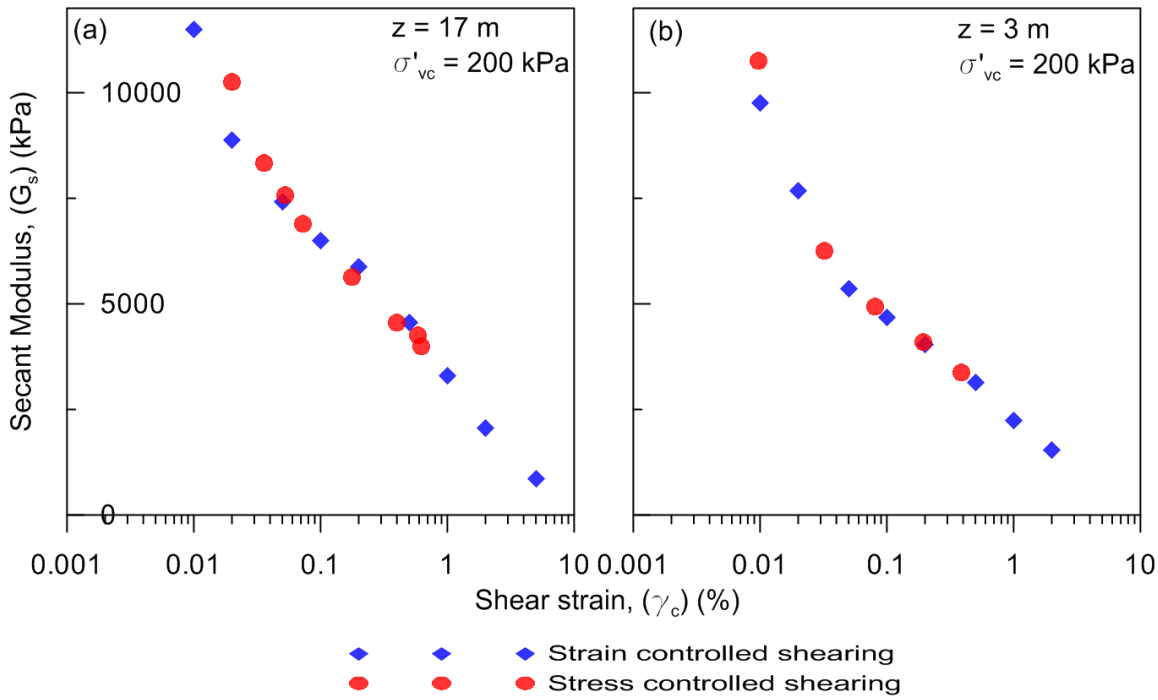

Fig 5.16 Comparison of modulus values from strain and stress controlled loading at (a) z = 3m (b) z = 17m

5.5 Modulus reduction and damping curves for Ottawa Leda clay

Modulus and damping values at depths of 17m and 3m were obtained at various γ_c magnitudes starting from 0.01% as discussed in the previous section. However, modulus reduction or stiffness degradation typically starts well below this strain amplitude. Hence, modulus $(G_{max})$and damping data at much smaller γ_c levels are necessary to propose a complete modulus reduction curve. This is generally achieved by resonant column or bender element tests. Obtaining data below γ_c of 0.01% using stress controlled shearing was not possible in this simple shear device due to the problems discussed earlier. Hence, some approximations were made in an attempt to propose approximate modulus reduction and damping ratio curves for Ottawa area Leda clay.

5.5.1 Approach

Modulus reduction curves proposed by Vucetic and Dobry (1991) for clays were considered to estimate the ratio of G_{max} to $G_{\gamma=0.01\%}$ and propose modulus reduction curves that can be used in ground response analysis. The trend of normalized modulus curves obtained at these depths (Fig 5.10 & 5.12) showed a pattern that they follow modulus reduction curves proposed for a PI of 15 at smaller γ_c amplitudes. At the same time they tend to follow curves proposed for a PI of 50 or more at larger γ_c levels (i.e. γ_c > 0.5%). At medium strain levels they tends to move from the curve of PI = 30 to PI = 50. Based on the observed trend, and Vucetic and Dobry (1991) data, the modulus value obtained at $\gamma_c$of 0.01% is estimated to be approximately 85% of the maximum shear modulus (G_{max}) at all σ'_{vc} levels [i.e. $(G_s)_{\gamma=0.01\%} \approx 0.85 \times G_{max}$]. This approximation plots this point closer to the curve proposed by Vucetic and Dobry for PI value of 15. The Vucetic and Dobry curve at this point also shows a modulus reduction value closer to 0.85 and modulus degradation starts at γ_c = 0.0005% for such case.

Moreover, Vucetic (1994) observed that the threshold strain for linear behaviour (γ_{tl}) (i.e. for no modulus reduction) is approximately 1.5 log cycles smaller than the threshold shear strain for volume change or pore pressure development (γ_{tv}). Approximate γ_{tv} value can be obtained by considering the pore pressure variation (Δu) with strain amplitude (γ_c) of the soil during cyclic shearing. Fig. 5.17 shows the variation of normalized pore pressure with γ_c for all the strain controlled quasi cyclic tests conducted in this research study. Even though they are not complete cyclic shear tests, from this figure, it can be inferred that approximate γ_{tv} of the tested Leda clay falls between γ_c of

0.007% to 0.008%. Hence, the assumption of γ_{tl}at 0.0005% for this Leda clay is reasonable.

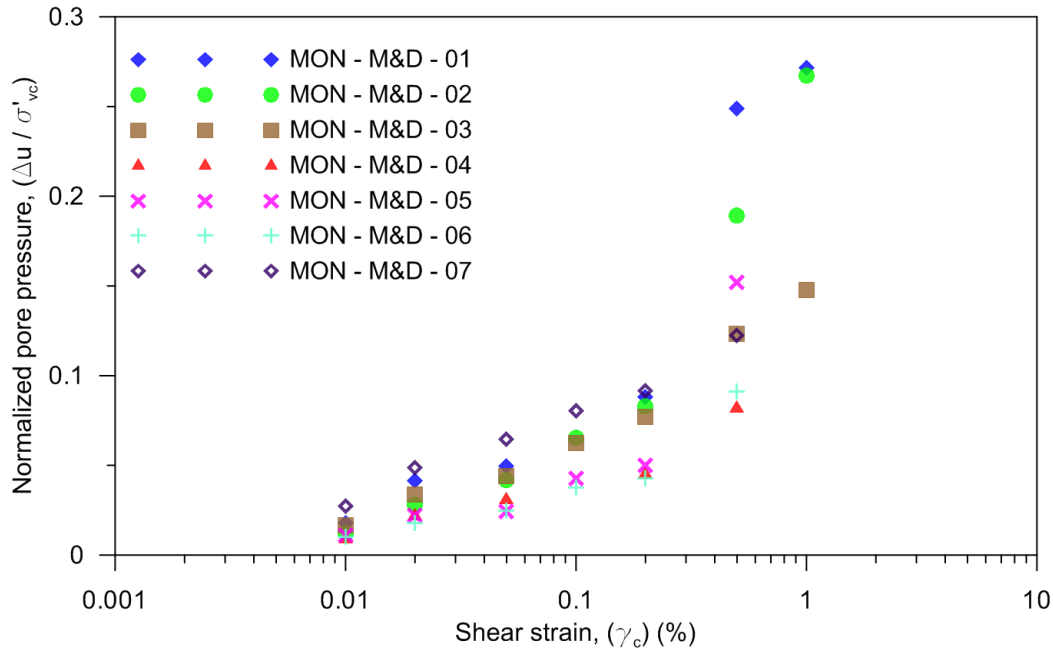

Fig 5.17 Variation of normalized pore pressure with γ_c in quasi-cyclic tests

5.5.2 Modulus reduction and damping curves under different conditions

Based on these simplifications, modulus reduction curves are proposed from the normalized modulus curves obtained at depths of 17m and 3m. The modulus values below γ_c of 0.01% are proposed based on the trend of these curves and the trend of modulus reduction curves proposed by Vucetic and Dobry (1991) as discussed. The proposed curves at 17m and 3m depths are shown in Figures 5.18 (a) and 5.19 (a) respectively. Best fit lines are also shown through these data points in these figures.

Similarly, damping ratio curves for these depths are proposed in Figures 5.18 (b) and 5.19 (b). Damping ratio data for γ_c amplitudes below 0.01% were obtained by extending the trend of damping curves obtained from test results. The best fit lines are also shown

in this figure. Moreover, modulus and damping curves proposed by Vucetic and Dobry are shown in the back ground in order to compare the trend of the proposed curves.

The Modulus reduction curves proposed from the data obtained at a depth of 17m (Fig 5.18 (a)) suggests that this curve tends to follow the curve for PI value of 15 at smaller strain levels (i.e. γ_c < 0.02%). At medium γ_c levels it spans from PI = 15 curve to PI = 30 curve. Furthermore, it tends to follow the curve of PI = 50 at larger strain amplitudes (i.e. γ_c > 1%). Moreover, a clear trend of structured Leda clay in its natural state can be observed at these larger strain amplitudes. Clays consolidated under σ'_{vc} of 400 and 800 kPa (i.e. in the normally consolidated region) show lesser degradation at large γ_c amplitudes. Hence, they follow the curves proposed for even higher PI values. The trend of normally consolidated or structured clay is proposed in broken lines.

The modulus reduction curve proposed for 3m depth (Fig 5.19 (a)) based on experimental data also follows a similar trend; however, initially it locates closer to curve proposed for PI = 30 material at very small γ_c amplitudes. Then, it shows steep modulus degradation between γ_c amplitudes of 0.01% and 0.05%. Moreover, no significant deviations in modulus reduction are observed at this depth. Possibly the σ'_{vc} levels considered (50, 100 and 200 kPa) may not have exceeded the pre consolidation stress enough to restructure these clays.

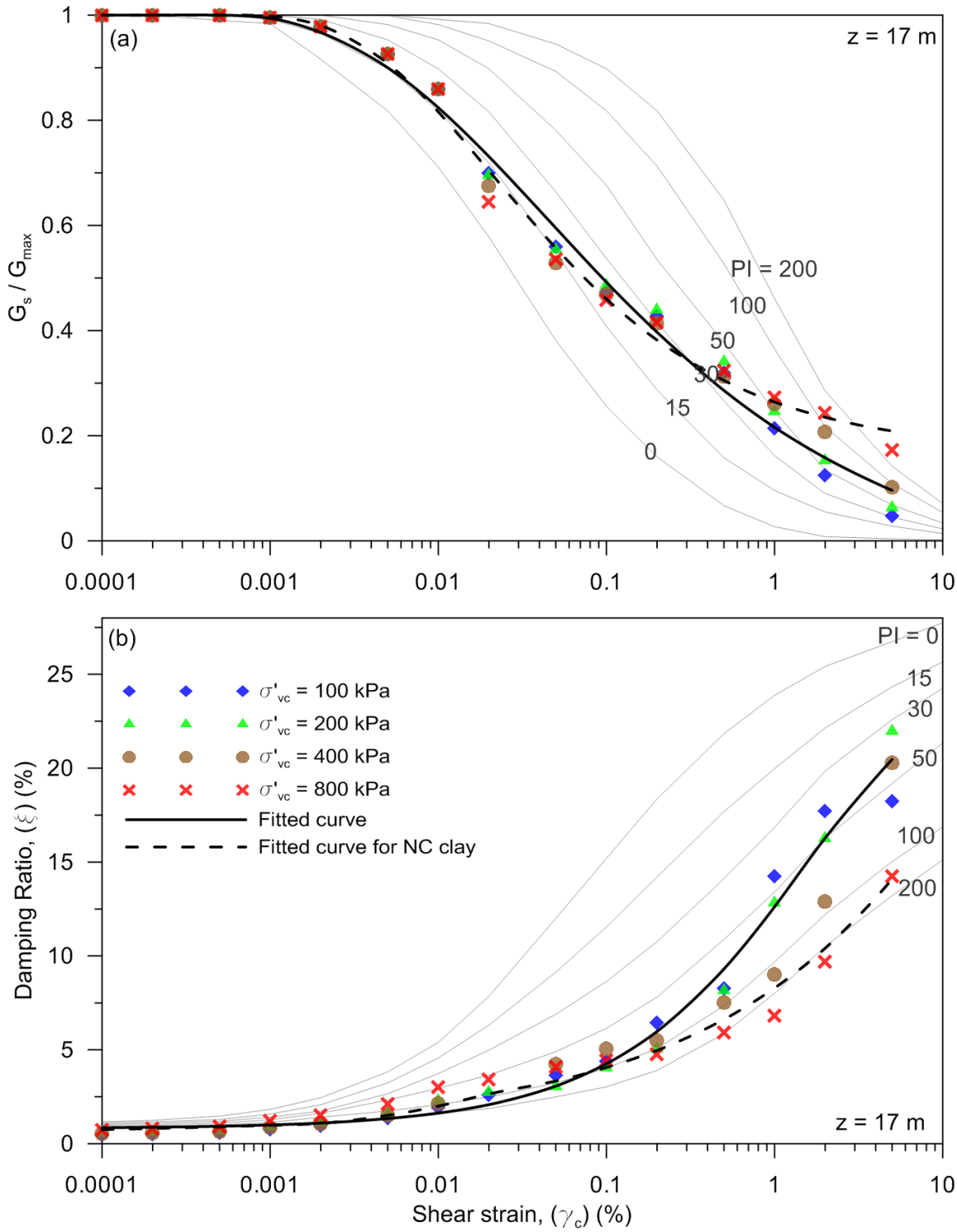

Fig 5.18 (a) Modulus reduction (b) Damping ratio curves proposed at z = 17m for Ottawa area Leda clay

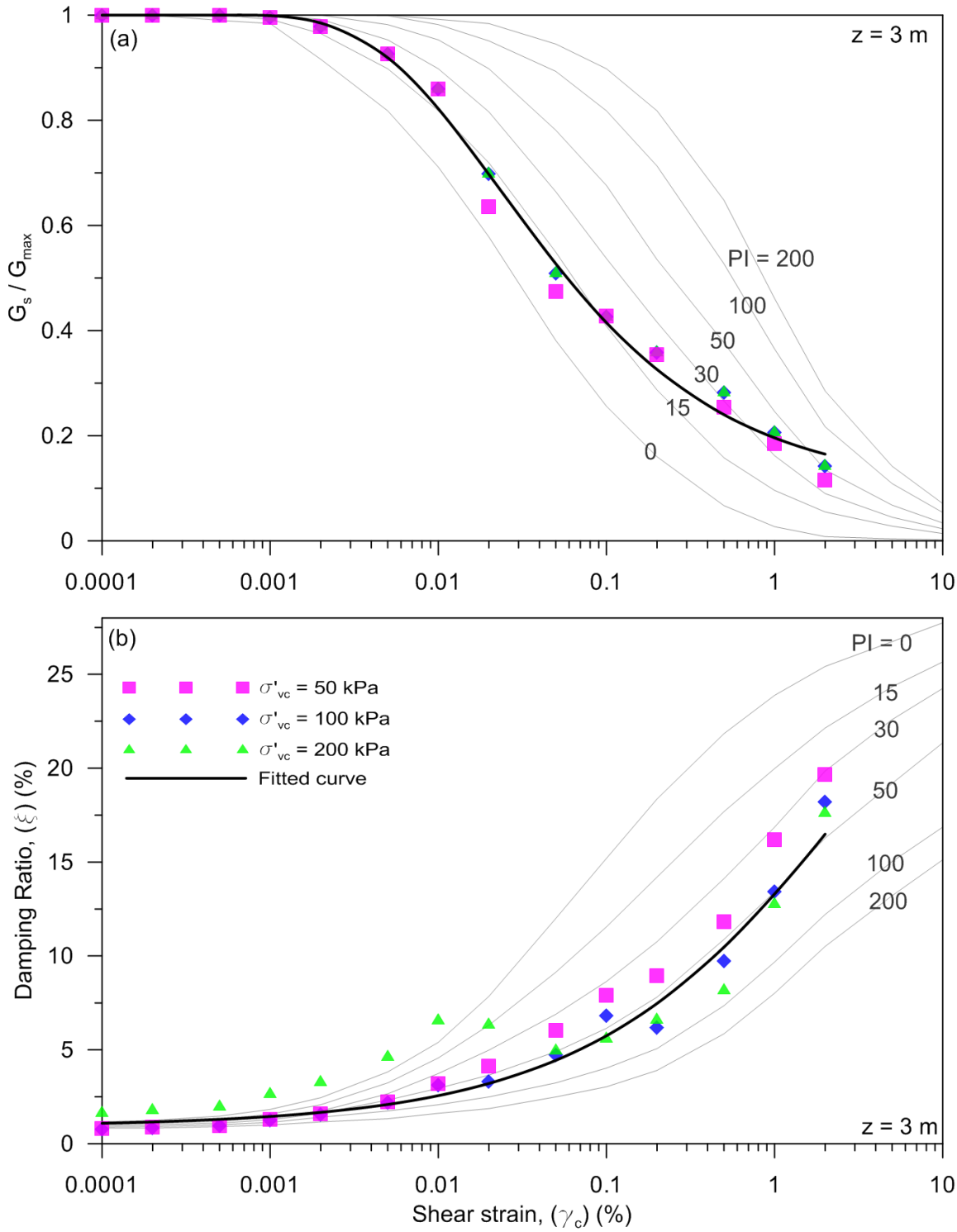

Fig 5.19 (a) Modulus reduction (b) Damping ratio curves proposed at z = 3m for Ottawa area Leda clay

The damping curve obtained for the depth of 17m (Fig 5.18 (b)) tends to follow the curve proposed for a PI of 100 at smaller strain values (i.e. γ_c < 0.1%). However, it should be noted that damping values reported for smaller strain amplitudes are not very accurate. The subsequent trend shows another clear difference between structured and intact Leda clay. The over consolidated or intact clay shows larger damping values at medium and large strain levels. It tends to move from the curve of PI = 100 to PI = 50 and towards PI = 30. However, the normally consolidated clay shows much lower damping values and tends to follow the curves proposed for PI of 100 and 200.

Damping curve proposed for 3m depth (Fig. 5.19 (b)) more or less follows the damping curve for PI = 50 until higher γ_c amplitude of 0.5%. Then it shows some increase in damping and deviates from the curve of PI = 50. At the same time highly or moderately over consolidated Leda clay such as the one consolidated under σ'_{vc} of 50 kPa shows larger damping values and falls in between the curves of PI between 30 and 50.

It is very common in current practice to use modulus reduction and damping curves from the literature based on other clays, but with a high P.I. value. Vucetic and Dobry (1991) curves corresponding to PI values of 100 or 200 are typically used. These results clearly indicate modulus reduction and damping curves at artificially high PI values are not exactly representative of Leda clay. Material specific modulus reduction curves do approach the high PI curves only at large strain values. The behaviour at small strain values is in fact comparable to the curves with similar PI values. Similarly, the nature of the damping curve is also affected by the strain level, and using a damping curve from the literature with a constant high PI material might not be appropriate.

5.5.3 Modulus reduction and damping curves for engineering analysis

The observations made on the above data at depths of 17m and 3m show that the modulus reduction and damping curves do not vary significantly with depth and falls within a fairly narrow band. In addition they do not show any significant variations with respect to PI values or follow any specific curves proposed by Vucetic and Dobry based on PI values. Moreover, these observations agree with the suggestions of Vucetic and Dobry (1991) for sensitive clays that the behaviour of these clays could be different from that of less or non sensitive clays.

While it is recognized that this study did not investigate complete variations and factors affecting these variations in modulus and damping of Leda clay, an approximate single curve based on this data might enable enhanced ground response analysis in Leda clays. Also, it can account for the fact that modulus reduction and damping curves in Leda clay is shaped somewhat differently compared to the curves proposed in the literature. Therefore one set of common modulus and damping curves for Ottawa area Leda clay are proposed based on the curves suggested for the depths of 17m and 3m. These proposed curves are shown in Fig 5.20.

The modulus reduction curve (Fig 5.20 (a)) follows more or less the curve proposed for clays with PI = 15 by Vucetic and Dobry until γ_c of 0.02%. Thereafter it gradually moves towards curves corresponding to higher PI values. The measured values are similar to a PI = 30% material at $\gamma_c \approx 0.5\%$ and PI = 50% at $\gamma_c \approx 2\%$. The actual PI values of the clays tested are from 12 to 30. Hence, the above observations show that sensitive Leda clays follows modulus reduction curves closer to their PI value at smaller γ_c levels. Then they deviate from these curves and follow the curves proposed for higher PI values. In

current practice, designers typically use Modulus and damping curves proposed for higher PI values in site response analysis that deals with sensitive clay (e.g. Nastev, 2012). However, the above observations suggest that this consideration is truly appropriate only at large γ_c levels.

In addition, restructured Leda clay in its natural condition shows even lesser modulus degradation at larger γ_c levels and follows the curves proposed for much higher PI values by Vucetic and Dobry. Hence a line suggesting this behaviour is also shown in the modulus reduction curve plot (Fig 5.20 (a)). Also, this curve would be useful in designing behaviour of Leda clay at stresses in the normally consolidated region.

The proposed damping ratio curve (Fig 5.20 (b) locates between PI = 50 and 100 curve until medium γ_c levels. However, damping values at larger γ_c levels are higher and the curve moves closer to PI = 50 curve at $\gamma_c \approx 2\%$ and move further. Hence the intact or over consolidated Leda clays show a damping behaviour opposite to that of modulus reduction behaviour. They tend to follow damping curves proposed for higher PI values at smaller γ_c amplitudes and then they tend to move towards the curves proposed for PI values similar to their own. Moreover, clays consolidated to normally consolidated states tend to show much lower damping ratios at larger γ_c amplitudes. Hence, a curve is suggested to illustrate the behaviour of structured Leda clay as in the case of modulus reduction curves. This curve follows curves proposed for much higher PI values (100 or 200) by Vucetic and Dobry (1991).

Moreover, the modulus reduction and damping values reported for Leda clay by Rasmuseen (2012) under σ'_{vc} of 106 kPa using simple shear device agrees with the curves proposed here. However, the values proposed using resonant column tests by Rasmuseen

(2012) varies from the curves proposed at small strain values in this study. The variation in loading mode and effects of frequency could be some possible reasons for these deviations. More data from various loading modes and from Leda clays of different locations would help to construct more accurate modulus reduction and damping curves for Eastern Canadian Leda clays. Moreover, they would be used to investigate the level of variation with respect to location and loading mode etc.

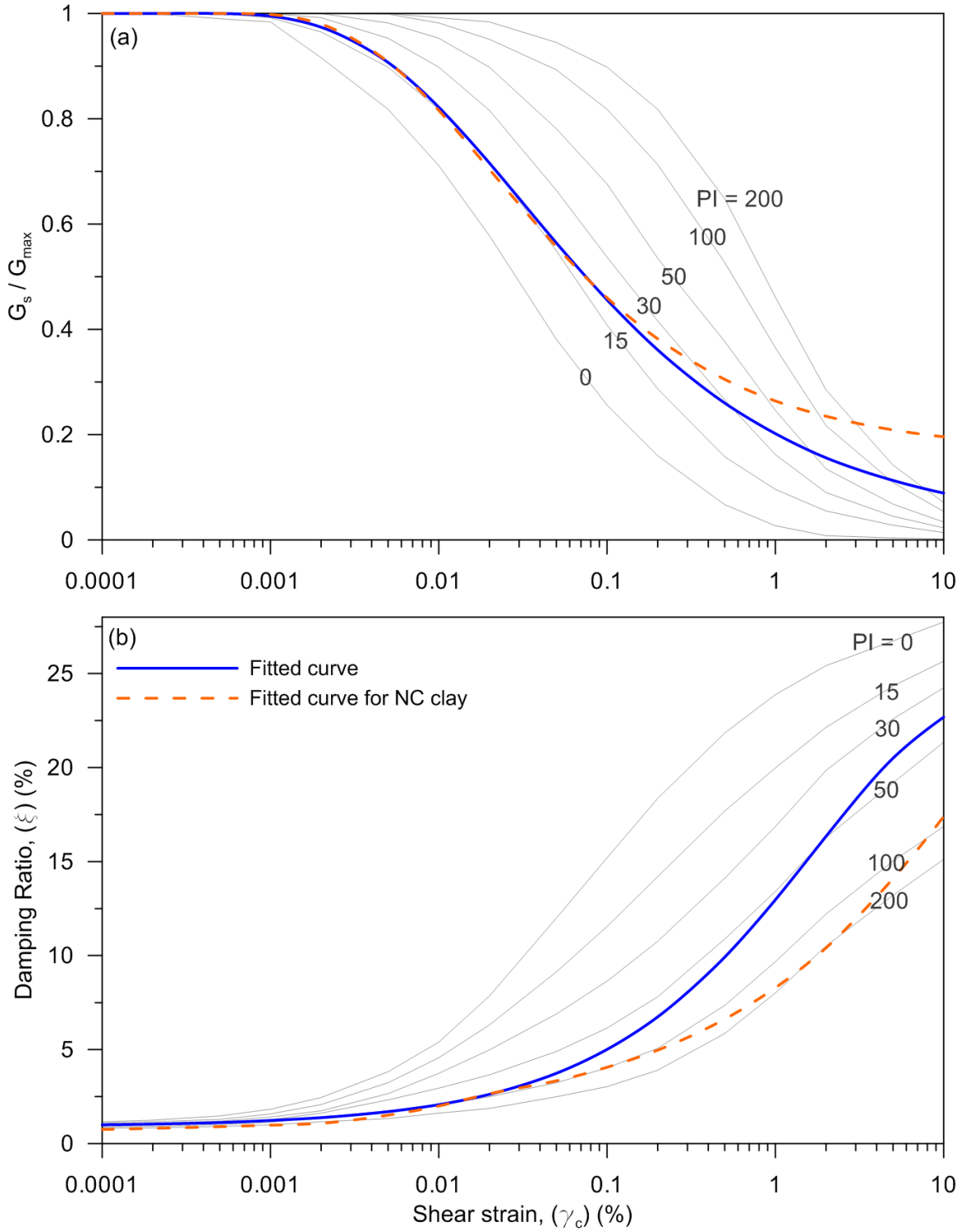

Fig 5.20 (a) Modulus and (b) Damping ratio curve for Ottawa Leda clay

5.6 Earthquake ground response analysis

Soft clays are known to amplify ground shaking produced by earthquakes. Many building codes around the world, including the National Building Code of Canada (NBCC), underwent considerable changes in their approach to site amplification following the damaging 1985 Mexico city earthquake, which caused extensive damage to structures founded on soft soil deposits but minimal or no damage to nearby structures that were on firm soil. A 'SHAKE' type site response analysis is currently the most common means of estimating site specific response during an earthquake. The modulus reduction and damping curves are key input to such an analysis, in addition to the soil profile and ground motion time history.

An earthquake ground response analysis was carried out for the Victoria Memorial Museum (currently called the Museum of Nature) site in Ottawa. The undisturbed samples tested in this research programme were obtained from this site and thus, an analysis using the modulus reduction and damping curves generated in this study would represent a true site specific analysis. Soil profile at the site was obtained from the data available in the literature, and input time history records consistent with NBCC shaking levels of 2005 and 2010 were generated using the approach prescribed by the SimQuake (Gasparini & Vanmarke, 1976) program. The response of the site for a number of input ground motion records was obtained using the 'ProSHAKE' software (Proshake, 2001).

Two sets of analyses were carried out for each set of input motions (i.e. NBCC 2005 & NBCC 2010 compatible). Firstly, the modulus and damping curves proposed from the tests results of this research study were used to represent the behaviour of the clay layers. For the second series, typical Vucetic and Dobry (1991) modulus reduction and damping

curves were used for clay layers (similar to the analysis typically conducted by practicing engineers). This was intended to enable a comparison of the ground response spectra from both these analysis and determine the differences in using conventional modulus and damping curves from literature and material specific curves.

5.6.1 Input parameters

A ground response analysis using 'ProSHAKE' software requires various input parameters. They are site profile including details of layers, geotechnical properties of each layer such as unit weight, maximum shear modulus (G_{max}) or shear wave velocity (v_s) and modulus reduction and damping curves. Furthermore, time histories of input motions should be assigned to the bedrock to represent the shaking expected from earthquakes.

The soil profile of this site is shown in Fig 5.21. Bedrock at the site is at a depth of about 36m. Available soil profile data suggests that the loose to dense sandy fill top material has a thickness of about 2m to 3m. For the ground response analysis it has been considered to be 2.5m thick top soil. High plasticity silty clay and clayey silt layer of about 20.5m thick is immediately below it. This is followed by 9m of transition zone that consists of clayey silt and sandy silt. This layer is immediately followed by a 4m thick glacial sand/till down to the bedrock. The soil profile was divided into 45 layers including the bedrock for the analysis.

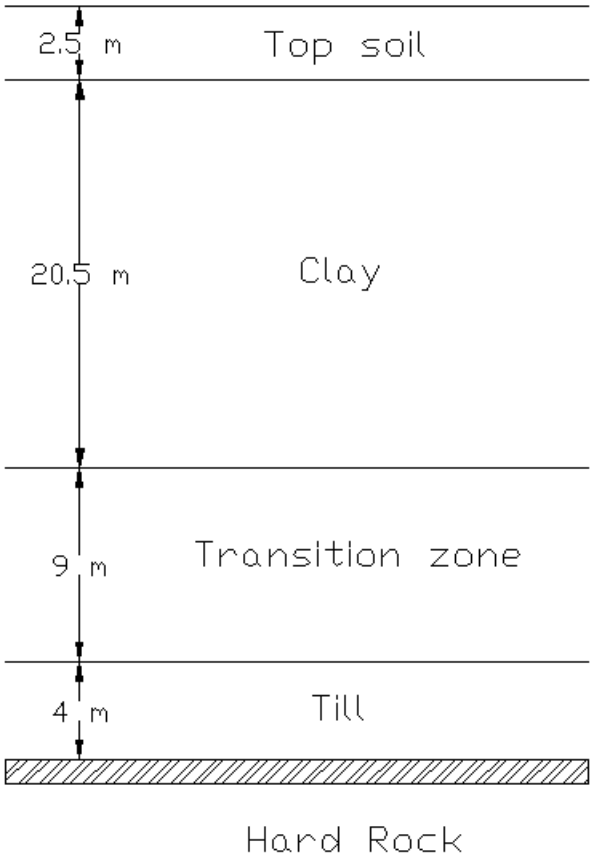

Fig 5.21 Soil profile of Victoria Memorial Museum site

The Leda clay layer that lies in the region of 2.5m to 23m depth is the focused zone in the context of this research. Data available in the literature indicate that clay from 3m to 15m depth has higher clay content. And clay from 15 to 19m depth has a lower clay fraction and lower liquid limit. Clayey silt is found between 19 and 23 m depth. For the site specific analysis (Analysis 1 (2005) and Analysis 1 (2010)) material specific curves obtained from this research program were used for this clay layer. Modulus and damping curves obtained from z = 3 m under σ'_{vc} of 50 kPa (M&D OTT z = 3 Sig V 50) was used between 2.5 and 5 m depth to accommodate the lower in-situ effective vertical stresses. For the region of 5 to 10 m, curves suggested for 3m depth (M&D OTT z = 3) in Fig 5.19 were used. The final average curves proposed in Fig 5.20 (M&D OTT avg) were used

between 10 and 15 m depths. Modulus reduction and damping curves proposed for 17 m depth (M&D OTT z = 17) shown in Fig 5.18 were used for the clay layer lies below 15 m depth. For the conventional analysis (Analysis 2 (2005) and Analysis 2 (2010)) Vucetic and Dobry (1991) curves with PI values slightly higher than the actual PI values of the material were assigned for the clay layers concerned. The modulus reduction and damping curves used for the clay layers in both set of analyses are shown and compared in Fig 5.22.

Other than the variation in the modulus reduction and damping curves in the clay all the input properties (including the modulus reduction and damping curves for the top soil, transition zone, till and bedrock) are the same in both analyses. Table 5.1 summarizes the soil input parameters used in these analyses.

Two sets of twelve input motions were applied at the Bed rock layer (Layer 45) for analyses based on NBCC (2005) and NBCC (2010). These time histories were generated to match the NBCC (2005) and NBCC (2010) input spectra on Class A bedrock respectively. They were named as 2005-EQ 1 to 2005-EQ 12 and 2010-EQ 1 to 2010-EQ 12. The 'Simquake' software was used to generate these input motions. Time history of a typical input motion (2005-EQ 1) and its response spectra are shown in Fig 5.23. The mean response spectrum of all NBCC 2005 compatible motions is also shown in this figure. Similarly Fig 5.24 shows time history of a typical motion (2010-EQ 2) compatible with NBCC 2010 and response spectra. It can be noted that the peak acceleration (PHA) values for NBCC 2005 and 2010 compatible input motions are 0.325g and 0.245g respectively.

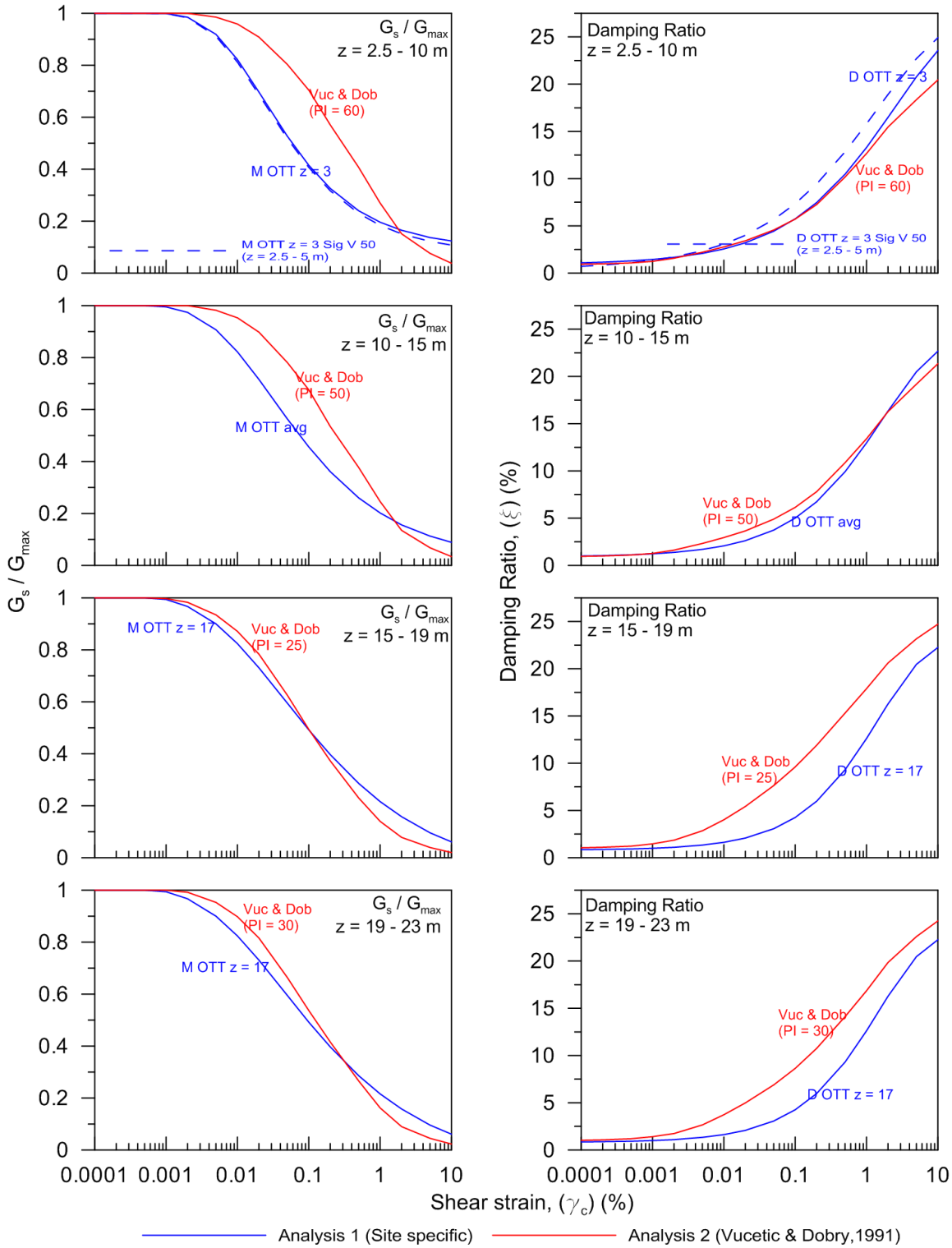

Fig 5.22 Modulus reduction and damping curves used for Analyses 1 and 2

Table 5.1 Details of input parameters for ground response analyses

Depth (m)	Material Name	Unit weight (kN/m3)	Shear wave vel (v_s)	Analysis 1		Analysis 2	
				Mod reduction curve	Damping curve	Mod reduction curve	Damping curve
0 – 2.5	Granular Fill	16	150	Sand (Seed & Idriss) - Upper	Sand (Seed & Idriss) - Lower	Sand (Seed & Idriss) - Upper	Sand (Seed & Idriss) - Lower
2.5 – 5	Top Clay	16	105 – 148.5	M OTT z = 3 Sig V 50	D OTT z = 3 Sig V 50	Vucetic – Dobry PI: 50-60	Vucetic – Dobry PI: 50-60
5 – 10	Top Clay			M OTT z = 3	D OTT z = 3		
10 – 15	Clay	16	155 - 182	M OTT avg	D OTT avg	Vucetic – Dobry PI: 30	Vucetic – Dobry PI: 30
15 – 19	Clay	16	188.5 - 215	M OTT z = 17	D OTT z = 17	Vucetic – Dobry PI: 25	Vucetic – Dobry PI: 25
19 – 23	Clayey Silt	16	220 - 240			Vucetic – Dobry PI: 25 -30	Vucetic – Dobry PI: 25 - 30
23 - 32	Transition Layer	16	240	Sand (Seed & Idriss) - Upper	Sand (Seed & Idriss) - Lower	Sand (Seed & Idriss) - Upper	Sand (Seed & Idriss) - Lower
32 -36	Till	20	250	Gravel (Seed et al)	Gravel (Seed et al)	Gravel (Seed et al)	Gravel (Seed et al)
36 - ∞	Bed Rock	25	1500	Linear (ξ = 1%)	Linear (ξ = 1%)	Linear (ξ = 1%)	Linear (ξ = 1%)

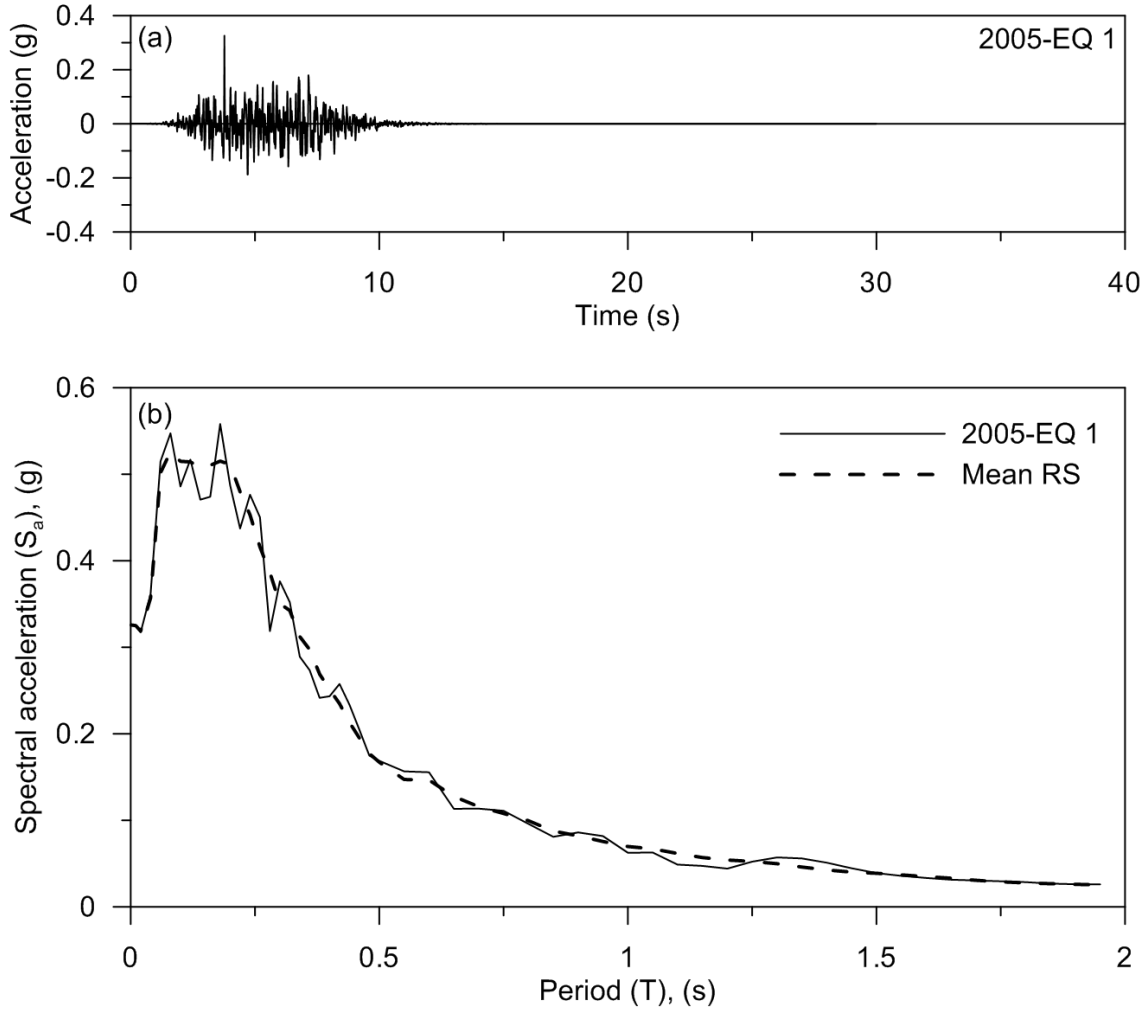

Fig 5.23 Time history and response spectra of a typical input motion (2005-EQ 1) compatible with NBCC (2005)

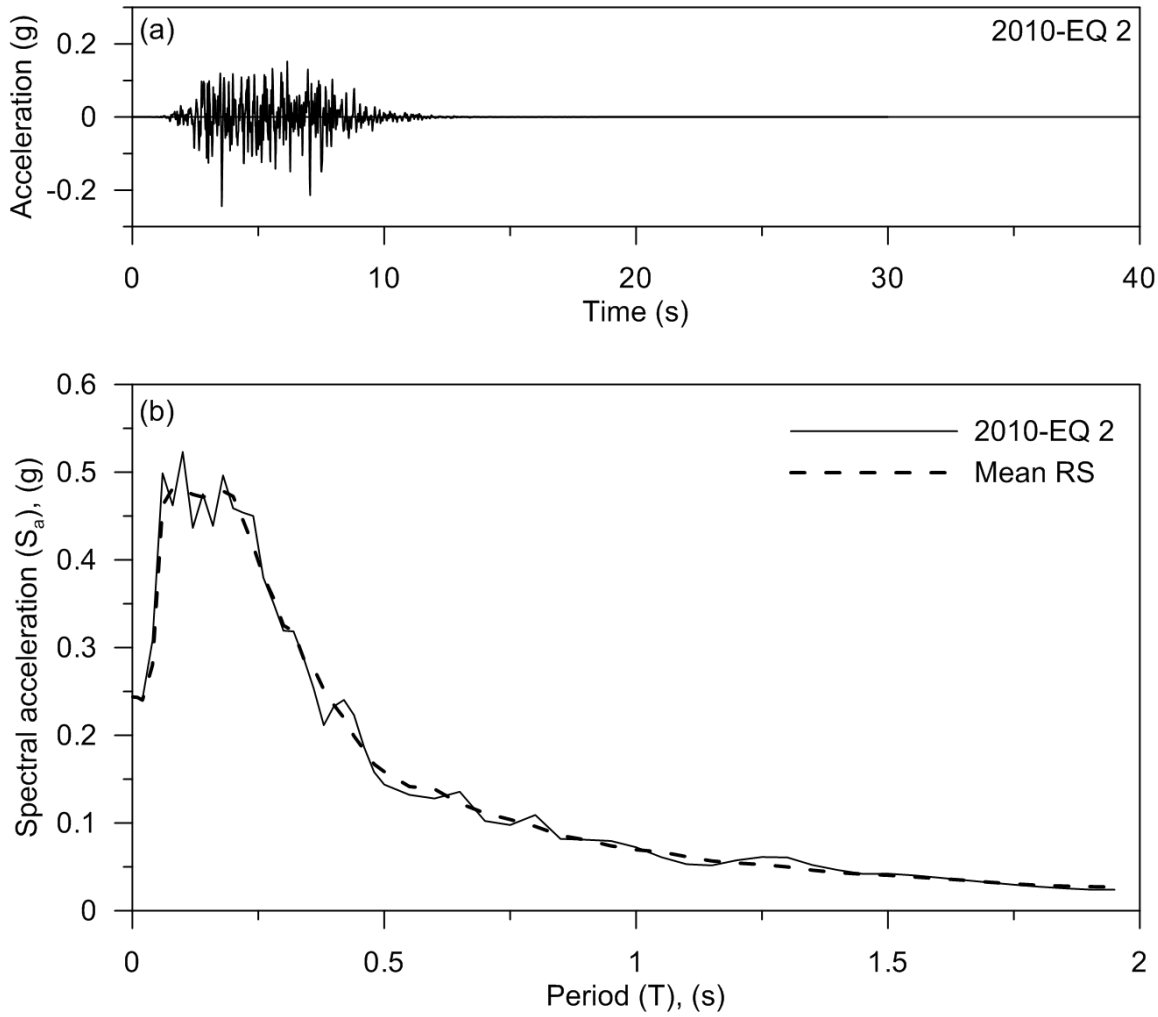

Fig 5.24 Time history and response spectra of a typical input motion (2010-EQ 2) compatible with NBCC (2010)

5.6.2 Ground response (Output)

Time history of the ground response was obtained from both site specific (Analysis 1) and conventional (Analysis 2) analyses for both set of analyses (NBCC 2005 and 2010 compatible). Peak ground acceleration values obtained for each input motion from NBCC 2005 and NBCC 2010 compatible analyses are shown in Tables 5.2 and 5.3 respectively. Typical ground response time histories obtained from site specific and conventional analyses for input motions 2005-EQ 1 and 2010-EQ 2 are shown in Figures 5.25 and 5.26. These observations illustrate that ground response pattern and peak acceleration

values for a given input motion is different between site specific and conventional analyses.

Table 5.2 Peak ground acceleration values for different input motions compatible with NBCC (2005)

	Peak horizontal acceleration (PHA) of ground (g)											
Input motion	EQ 1	EQ 2	EQ 3	EQ 4	EQ 5	EQ 6	EQ 7	EQ 8	EQ 9	EQ 10	EQ 11	EQ 12
Analysis 1 (Site specific)	0.239	0.335	0.368	0.316	0.282	0.249	0.281	0.25	0.311	0.215	0.245	0.283
Analysis 2 (Conventional)	0.239	0.264	0.351	0.303	0.251	0.283	0.233	0.284	0.253	0.292	0.321	0.25

Table 5.3 Peak ground acceleration values for different input motions compatible with NBCC (2010)

	Peak horizontal acceleration (PHA) of ground (g)											
Input motion	EQ 1	EQ 2	EQ 3	EQ 4	EQ 5	EQ 6	EQ 7	EQ 8	EQ 9	EQ 10	EQ 11	EQ 12
Analysis 1 (Site specific)	0.222	0.326	0.337	0.282	0.269	0.224	0.278	0.239	0.273	0.216	0.21	0.274
Analysis 2 (Conventional)	0.209	0.255	0.314	0.255	0.239	0.249	0.255	0.234	0.236	0.265	0.279	0.219

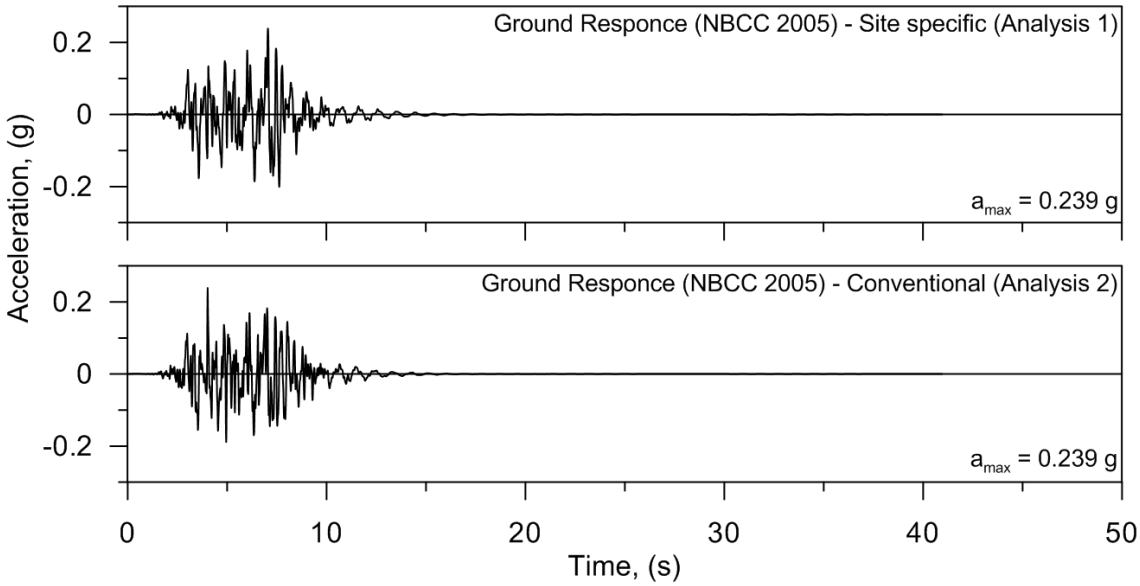

Fig 5.25 Time history of ground response for input motion 2005-EQ 1

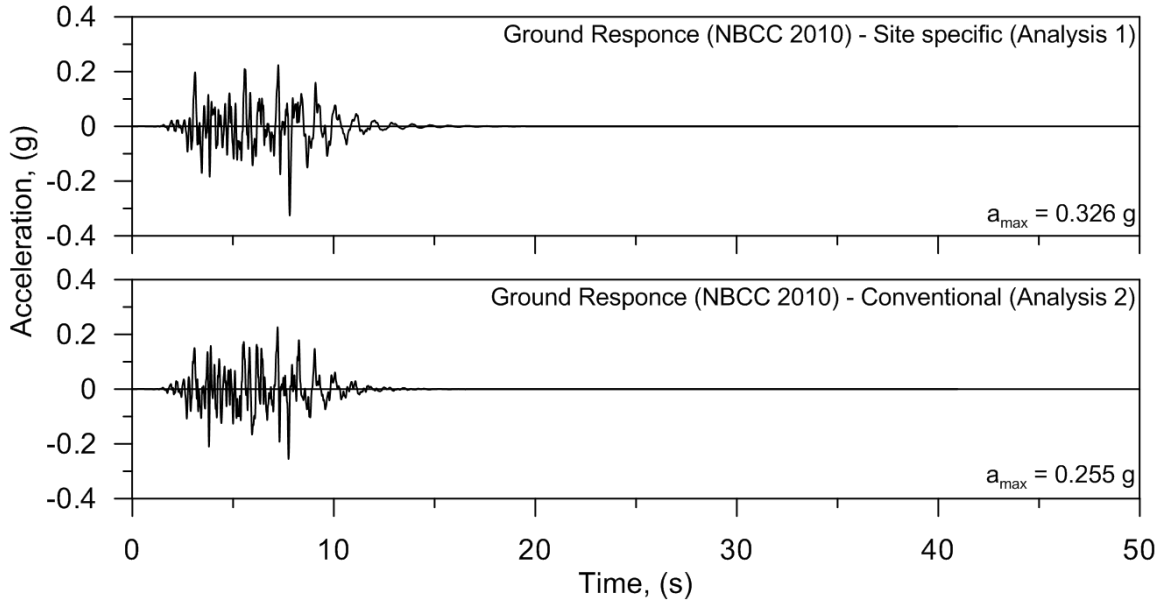

Fig 5.26 Time history of ground response for input motion 2010-EQ 2

Spectral acceleration values were obtained at different periods (T) based on a damping ratio of 5% (a typical value for structural design). Figures 5.27 (a) and 5.27 (b) show ground response spectra obtained from both analyses for the above input motions (2005-EQ 1 and 2010-EQ 2). For input motion 2005-EQ 1, analysis carried out based on material specific curves (Analysis 1) shows the highest spectral acceleration at approximately T = 0.25s (Fig 5.26 (a)), while peak spectral acceleration from conventional analysis occurs at period (T) lower than this. Similar response was obtained for the input motion 2010-EQ 2 as well (Fig 5.26 (b)). Moreover, in both occasions peak spectral values from these two analyses do not occur at equal periods, peak spectral values from site specific analysis lags slightly in terms of period than that from conventional analysis. Also, site specific analysis produces somewhat higher amplification at larger period levels.

Similarly response spectra were obtained for all the input motions in the two sets of analyses. The individual output spectra are of limited value due to their dependence on

the input time history. Hence, mean response spectra obtained for NBCC 2005 and 2010 compatible motions based on the ground response for all individual input motions is shown in Fig 5.28. Moreover, the ratios of this mean spectral acceleration values from analyses 1 and 2 are also calculated in terms of period and are shown in Fig 5.29.

These results show that a ground response analysis performed using conventional modulus reduction and damping curves proposed for typical clays may be different from the actual behaviour of sensitive Leda clays. The peak spectral acceleration values corresponding to individual earthquakes vary by as much as 50% and the analysis using site specific parameters generally shows higher amplification at large period values. Even though the peak value of the mean spectral ground acceleration does not vary much, there is clearly a small shift in the response as a function of site period. Moreover, the variation of the ratio of mean spectral values shown in Fig 5.29, illustrates that sites with some specific period levels (e.g. around T = 0.4s) may show significant variations in amplifications when using conventional modulus reduction and damping curves. Hence, these observations illustrate the importance of using material specific parameters in calculating accurate amplifications. At the same time, behaviour in a specific analysis (e.g. site specific) does not show any significant characteristic variations between NBCC 2005 and NBCC 2010 (Figures 5.28 and 5.29) as expected.

Additional, more comprehensive studies are required to better understand the effects approximating the soil properties on the results of site response studies. These results suggest that the use of material specific modulus and damping curves are necessary to obtain reliable estimates of resonance frequencies.

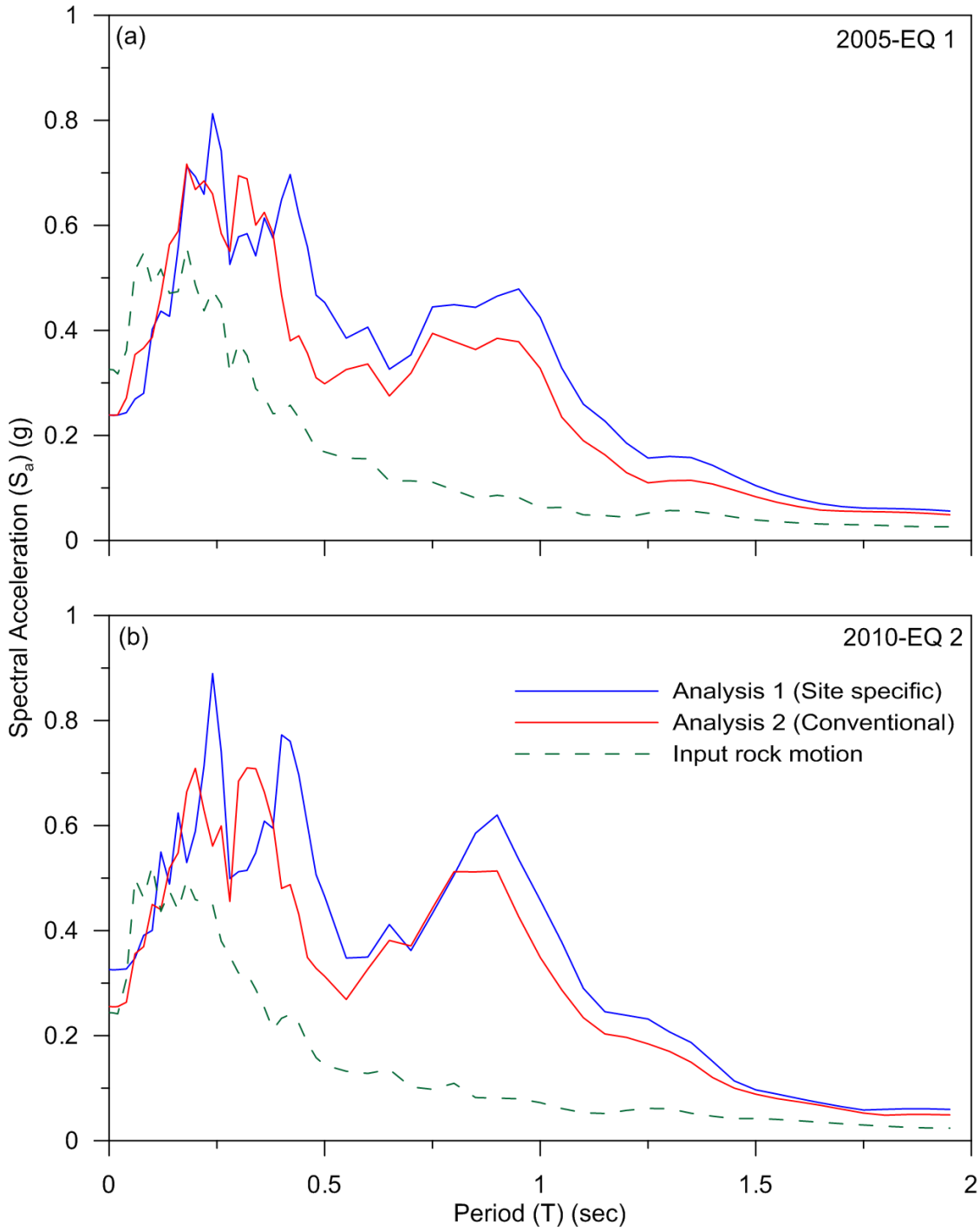

Fig 5.27 Ground response spectra for input motions (a) 2005-EQ 1 (b) 2010-EQ 2

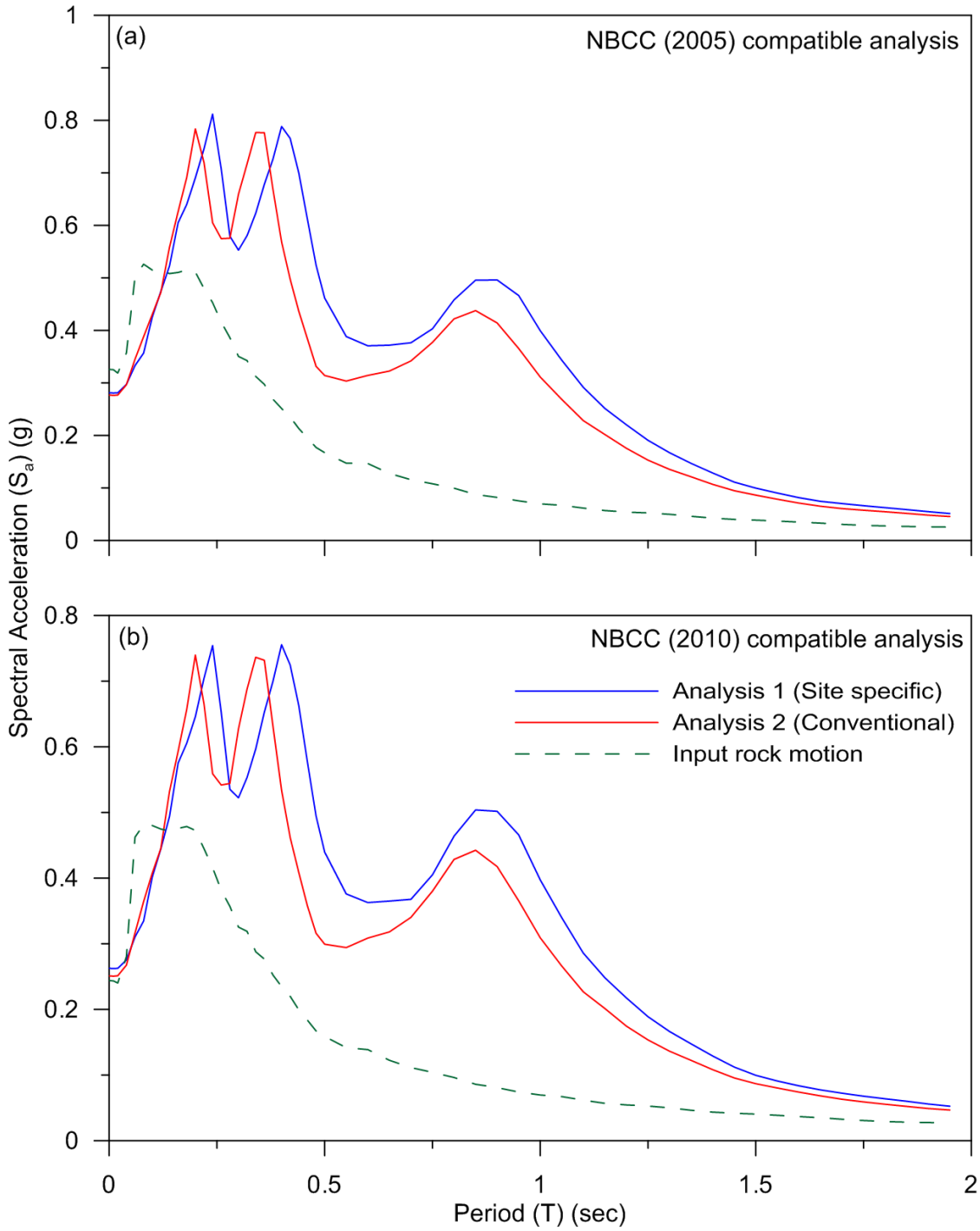

Fig 5.28 Mean ground response spectra for (a) NBCC 2005 (b) NBCC 2010 motions

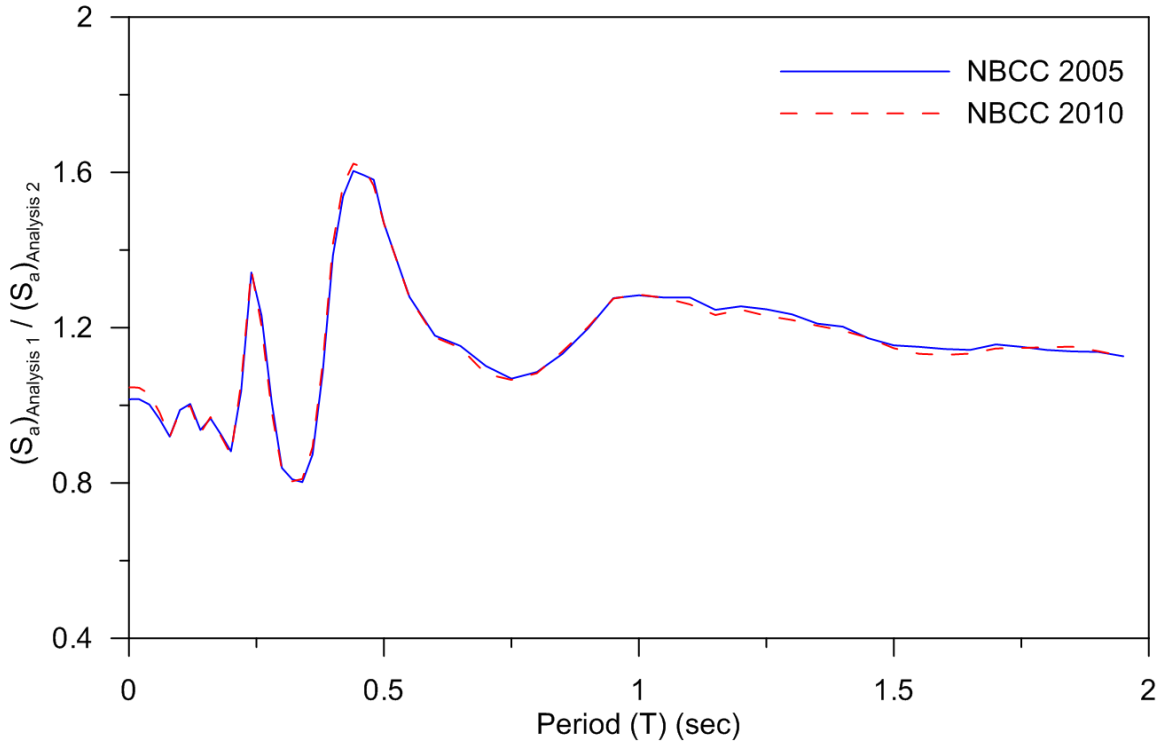

Fig 5.29 Ratio of mean spectral values from Site specific and conventional analysis

6 SUMMARY AND CONCLUSIONS

6.1 Summary

A knowledge of the monotonic and cyclic shear deformation characteristics, and strength reduction due to such loading events in sensitive clays (such as the Champlain Sea clay of Eastern Canada) is important in geotechnical engineering practice. In addition, material specific modulus reduction and damping curves are necessary to carry out confident dynamic soil response investigations in sensitive clays. An experimental study was carried out to investigate the above properties and characteristics of Champlain sea clay under simple shear conditions.

The existing simple shear device that was capable of measuring monotonic and cyclic shear characteristics at large strain levels was modified to enable the measurement of soil behaviour at small strain amplitudes. More sensitive horizontal and vertical LVDTs were included with appropriate mounting hardware to significantly improve the measurement resolution at very small strain levels without sacrificing the ability to measure large strains. Furthermore, existing data acquisition programme was modified to facilitate both strain and stress controlled quasi cyclic shearing with varying amplitudes continuously. These improvements enabled the CU simple shear device to measure dynamic soil properties such as modulus and damping at small strain levels with reasonable accuracy.

The modified CU simple shear device was used to conduct both small and large strain tests on undisturbed clay samples obtained from a site in Ottawa. Small strain tests yielded modulus reduction and damping characteristics of Leda clay, and the large strain tests were used to investigate the monotonic and cyclic shear strength characteristics of Leda clay. A pair of modulus reduction and damping curves that can be used in site

specific numerical analysis to assess the response of Leda clay during earthquakes are proposed, and the nature of progressive strength degradation in Leda clay was studied.

6.2 Conclusions

The experimental investigation on undisturbed Leda clay samples lead to the following conclusions. It is noted that tests were conducted on undisturbed samples obtained from a single site in Ottawa and caution is warranted before using these at other sites. Design decisions should not solely rely on these test results alone until additional studies on Leda clay samples from different sites are conducted to establish the range of variability, and dependence on various factors that might change from site to site. Nonetheless, the modulus reduction and damping curves proposed herein are believed to be an improvement over using '*text-book'* curves from the literature, or those built into geotechnical software.

6.2.1 Shear strength behaviour

Leda clay specimens from two depths show typical strain softening behaviour during monotonic shearing. Furthermore, normally consolidated Leda clay shows an essentially unique normalized stress–strain behaviour. On the other hand, over consolidated clay shows increasing normalized strength with OCR. Hence, the monotonic behaviour of tested Leda clay closely matches the typical strength behaviour of clays in qualitative terms. Moreover, friction angle at peak strength of NC clay and friction angle at residual state do not show significant variation at the two depths considered ($\varphi'_p \approx 21° \; and \; \varphi'_r \approx 26°$).

Cyclic strength of Leda clay typically reduces like a hyperbolic curve with number of cycles in logarithmic scale under different vertical effective consolidation stress levels

(σ'_{vc}) considered. Cyclic strength clearly reduces with σ'_{vc} in intact or over consolidated clays (i.e. σ'_{vc} of 100 & 200 kPa) in terms of CSR and percentage of static undrained strength (S_u). Also, clay at shallower depth with higher clay content show more cyclic resistance than that at deeper depth (17m). However, clay consolidated to 400 kPa shows larger cyclic resistance in terms of percentage of S_u than that of Leda clay consolidated to 200 kPa. This might represent the effects of soil fabric changes as the sensitive clay collapses at consolidation stress levels that exceed the pre consolidation pressure. Additional studies are required to establish the dependence of cyclic resistance on effective consolidation stress.

Cyclic simple shear tests indicate that cyclic shear stress amplitude (τ_{cyc}) of at least 50 to 60% of the undrained strength of clay is necessary to 'liquefy' the Leda clay from this site within 50 cycles. Similarly cyclic loading events with CSR less than 0.2 would not liquefy the soil within 15 cycles (i.e. equivalent to M 7.5 earthquake). Liquefied Leda clay shows stress–strain behaviour similar to that of an OC clay during post cyclic shearing. It does not show a region where deformation occurs at essentially zero stress strength, which is characteristic of liquefied sands. Furthermore, it regains stiffness during the course of shearing and shows an average post cyclic strength of 75% of its undrained strength (S_u). The post cyclic strength shows some dependency on the magnitude of pore pressure developed during cyclic loading.

Limited pre-straining (a few small strain-amplitude quasi–cyclic shearing well below the strain corresponding to the peak strength) does not alter the subsequent large strain shear behaviour significantly. However, pre-loading beyond the peak-strength state causes significant strength reductions during subsequent shearing. A gradual reduction in shear

strength was noted with each large strain cycle, and this suggests that the strength-loss due to remoulding is progressive.

6.2.2 Modulus and damping characteristics

This research programme established material specific modulus and damping characteristics of sensitive Leda clay using quasi–cyclic strain controlled tests. These tests show that reliable data can be obtained by appropriately using the strength reduction characteristics of Leda clay. Modulus and damping values at multiple small strain amplitudes can be obtained by performing quasi cyclic stress or strain controlled tests of varying magnitudes to a single specimen; thus avoiding the uncertainties related to natural heterogeneity of samples from different in-situ locations.

An attempt was made to obtain modulus and damping data at very small γ_c ($\approx 0.001\%$) amplitudes under multiple σ'_{vc} levels using a stress–controlled quasi-cyclic testing process. But, this did not yield quality data in this programme due to mechanical issues in the simple shear device. While the data acquisition system has sufficient resolution to measure displacements that are smaller than $\gamma_c \approx 0.001\%$, the device is not sufficiently accurate to enable readings at small strain level. The smallest strain level that could be measured with reasonable confidence and repeatability in this simple shear device was determined to be about $\gamma_c \approx 0.01\%$. Both, stress and strain controlled quasi–cyclic tests provided similar modulus reduction data in simple shear conditions.

Modulus and damping values of Ottawa area Leda clay at γ_c amplitudes above 0.01% are presented at two depths under different σ'_{vc} levels. Basic characteristics of the modulus and damping behaviour of Leda clay are similar to other soils. However, normalized modulus curves and damping ratio curves fall within a very narrow band regardless of the

σ'_{vc} levels as in the case of typical clay. Leda clays consolidated to very high stress levels and possibly restructured in their natural state show some deviations from this behaviour, especially at large γ_c amplitudes.

Normalized modulus and damping curves obtained at two depths do not show a significant correlation with PI values of Leda clay (the range of PI values encountered are fairly small at about 15-30%). More importantly, the material specific curves do not follow any specific curve proposed in the literature (Vucetic and Dobry, 1991) for clays based on their PI values. Vucetic and Dobry have mentioned that sensitive clays do not behave similar to typical non sensitive clays, but yet the practice in the industry has been to use Vucetic and Dobry curves at a high PI value.

The modulus and damping values obtained were rescaled to account for the degradation in material properties from elastic range ($\gamma_c < 0.001\%$) to the smallest strains that were measurable in this device ($\gamma_c \approx 0.01\%$) in order to propose complete material specific modulus reduction and damping curves for Ottawa area Leda clay. Modulus reduction curves tend to follow curves proposed for lower PI values at smaller γ_c amplitudes, and gradually move towards the curves of higher PI values at larger γ_c amplitudes. However, damping ratio curves moves from higher PI curves to lower PI curves with increasing γ_c amplitudes. Damping ratios reported at very small γ_c amplitudes are not very accurate due to friction and other uncertainties of the simple shear device as discussed.

A demonstration earthquake site response analysis was performed for a site in Ottawa using the modulus reduction and damping curves suggested from this study. The site response from this analysis is compared with the analysis performed using Vucetic & Dobry (1991) modulus reduction and damping curves. Analysis using site specific

parameters generally shows higher amplification at large period values and shows a shift in response in terms of site Period (T).

6.3 Recommendations for Future work

The results obtained from this research study provide some basic insights about shear strength and dynamics properties of Leda clays of Ottawa area. However, more comprehensive data are important to draw firm conclusions on the behaviour of Leda clays and the factors influence their behaviour. This is a difficult task due to the fact that essentially identical undisturbed samples are required to properly evaluate the effect of a given factor. But, such studies are critical to improve our understanding of the behaviour of sensitive clays that underlie many urban population centres. Strength tests using triaxial and torsional hollow cylinder devices would provide additional information regarding the influence of loading mode, anisotropy and principal stress rotation on monotonic and cyclic strength behaviour of Leda clay

The suitability of simple shear device to obtain stress strain behaviour accurately at very small stress - strain levels can be investigated by resolving the mechanical issues in the current device or using some more sophisticated and accurate loading. Furthermore, successful implementation of the quasi cyclic test methods can be tested in other testing devices as well by following similar procedures and modifications presented here. Moreover, more data on modulus and damping on Eastern Canadian Leda clay from various locations and from different loading modes (i.e. different testing equipment) would help to construct more accurate material specific modulus reduction and damping curves for Champlain Sea clay of Eastern Canada.

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APPENDIX - A

(Details of Monotonic Test Results)

Table A. 1 Details of Monotonic tests on Ottawa Leda clay

Test ID	Borehole	Consolidation stress (σ'_{vc}) / (kPa)	Properties at peak state				Properties at γ = 20% (Residual)	
			S_p / (kPa)	S_p / σ'_{vc}	γ / (%)	σ'_v / (kPa)	S_r / (kPa)	S_r / σ'_{vc}
MON - 01	BH 2 – 50’ 10” C	100	34.5	0.35		66	0.35	0.27
MON - 01 A	BH 2 – 50’ 10” C	100	34.5	0.35		66	0.35	0.28
MON - 02	BH 2 – 50’ 10” C	200	52.5	0.26	3.5	135	0.26	0.15
MON - 03	BH 2 – 50’ 10” C	400	88	0.22	4.3	234	0.22	0.12
MON - 04	BH 2 – 50’ 10” C	800	171.5	0.21	6.5	445	0.21	0.13
MON - 05	BH 1 – 50’ 08” B	50	28	0.56		40	0.56	0.21
MON - 06	BH 2 – 11’ 07” B	100	39	0.39	4	77	0.39	0.27
MON - 07	BH 2 – 11’ 07” B	400	94.5	0.24	5	245	0.24	0.18
MON - 08	BH 2 – 55’ 11” C	100	34.7	0.35		68	0.35	0.27
MON - 09	BH 2 – 55’ 20” B	200	58.5	0.29	5.5	131	0.29	0.21
MON - 10	BH 2 – 57’ 11” A	400	95	0.24	6.1	238	0.24	0.17

APPENDIX - B

(Details of Cyclic Test Results)

Table B. 1 Details of cyclic tests on Ottawa Leda clay

Test ID	Borehole	σ'_{vc} / (kPa)	Cyclic					Post cyclic (Monotonic)				
			τ_{cyc} / (kPa)	τ_{cyc}/S_u (%)	CSR	N	Max γ / (%)	S_p / (kPa)	S_p/σ'_{vc}	S_r/σ'_{vc}	Initial PWP $\Delta u/\sigma'_{vc}$	S_p/S_u (%)
CYC - 01	BH 2 55' 11" B	100	24	69	0.235	53	3.8	29	0.29	0.23	0.89	84
CYC - 02	BH 2 55' 11" B	100	27.5	79	0.272	19	3.8	28	0.28	0.21	0.88	81
CYC - 03	BH 2 55' 11" B	100	31	89	0.304	10	3.8	30.5	0.31	0.22	0.76	88
CYC - 04	BH 2 55' 11" B	100	34	98	0.333	4	4	26	0.26	0.18	0.69	75
CYC - 05	BH1 55' 02" B	200	30	51	0.149	37	3.95	32	0.16	0.10	0.90	55
CYC - 06	BH1 55' 02" B	200	35	60	0.175	22	3.8	39	0.20	0.12	0.88	67
CYC - 07	BH1 55' 02" B	200	41	70	0.204	9	4	40.5	0.20	0.14	0.88	69
CYC - 08	BH1 55' 02" B	200	47	80	0.236	5	3.8	44	0.22	0.13	0.75	75
CYC - 09	BH1 57'11" A	400	57	60	0.143	60	4	70	0.18	0.11	0.80	74
CYC - 10	BH1 57'11" A	400	67	71	0.167	23	3.8	83	0.21	0.13	0.68	87
CYC - 11	BH1 57'11" A	400	71	75	0.180	6	4.2	76	0.19	0.10	0.63	80
CYC - 12	BH1 57'11" A	400	76	80	0.192	5	4.05	83	0.21	0.12	0.55	87
CYC - 13	BH2 11' 07" C	100	26	67	0.259	65[2]	3.1[3]	30	0.30	0.18	0.62	77
CYC - 14	BH2 11' 07" C	100	30	77	0.286	38	11.3[3]	24	0.24	0.24	0.86	62
CYC - 15	BH2 11' 07" C	100	33	85	0.327	27	4.5	28	0.28	0.22	0.76	72
CYC - 16	BH2 11' 07" C	100	38	97	0.376	5	7	22	0.22	0.19	0.82	56

[2] Estimated value
[3] Shear strain values due to error in defining stop criterion in the data acquisition program