BEHAVIOUR OF SENSITIVE CLAY UNDER CYCLIC LOADING

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ABSTRACT

Behaviour of Sensitive Clay Under Cyclic Loading

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Sensitive clay is a kind of natural soil known to develop occasional flow slides. Understanding its behaviour becomes a demanding research field due to the rapid development of infrastructures, housing projects, and large-scale land reclamations. The economical implication of constructing foundations built on sensitive clay becomes evident while considering the wide occurrence of this clay in the Eastern Canada and other parts of the world. In order to understand its special features, research studies in the literature are mostly devoted to the exploration of the fundamental properties of sensitive clay, the evaluation of different testing techniques, the appropriate methods of testing, and case studies. Research on the strength of sensitive clay has been lagging, especially when subjected cyclic loading.

Due to the complex behaviour of sensitive clay, developing a unique model, which incorporates many parameters, is practically impossible. This thesis represents a study of parameters, which are believed to influence the strength properties of sensitive clays. The experimental data available at Concordia University and in the literature are used extensively to evaluate the role of each parameter. Based on the results of these analyses, a step-by-step design approach has been developed and tested for designing foundations on sensitive clay. This procedure can also be used to examine the condition of an existing foundation built on sensitive clay.

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LIST OF SYMBOLS

C_c	Compression index
C_u	Undisturbed undrained shear strength
$(C_u)_r$	Remoulded shear strength
C_r, C_α	Consolidation parameters
C_L	Undrained shear strength of remoulded soil at liquid limit
C_P	Undrained shear strength at plastic limit
C_v	Coefficient of consolidation
F.S	Factor of safety
I_L	Liquidity index
I_P	Plasticity index
k	constant describing the variation of sensitivity with liquidity index
LL	Liquid limit
N	Number of cycle
OCR	Over consolidation ratio
PL	Plasticity index
p_c	Effective man principal stress after consolidation
q_{allow}	Allowable deviator stress
q_{cyc}	Cyclic deviator stress
$q_{cyc}/2C_u$	Cyclic strength ratio
q_{max}	Maximum deviator stress
q_s	Static deviator stress

q_s/p_c	Initial static deviator stress ratio
R	Ratio of C_p to C_L
$R_{(N=20)}$	Anisotropic cyclic strength ratio to cause failure at 20 cycles
R_{ISO}	Isotropic cyclic strength
RR(N)	Index of the possibility of cyclic failure
R_f	Cyclic shear strength
R_L	Intrinsic cyclic strength
S_t	Sensitivity
u^+	Pore water pressure generated due to cyclic loading
w	Water content
w_L	Liquid limit
α	Aging parameter
σ_p	Preconsolidation pressure
$\sigma_s/2\sigma'_c$	Initial drained shear stress ratio
σ_{3c}	Confining pressure
ε_p	Peak axial strain
η	Effective stress ratio
η_f	Effective stress ratio at failure
η_p	Effective stress ratio at the peak of the cyclic stress in each cycle
η_s	Effective stress ratio for initially consolidated condition
η^*	Relative effective stress ratio
κ	Cyclic strength after first cycle

CHAPTER 1

Introduction

1.1 Background Information

Landslides, liquefaction, down slopes and failure of structures under cyclic (repeated) loading are all problematic phenomena that have challenged engineers for a very long time. In the literature, researchers have come to the conclusion that the presence of a special clay type is responsible for most of these disasters. This type of clay is called *sensitive clay*. This clay is a kind of natural soil with unusual characteristics. It displays a considerable decrease in shear strength after disturbance (remoulding). The difference between the firmness in the undisturbed state and that in the remoulded state is considered the characteristic property of sensitive clay. This is due to the flow slides of this clay, which has tendency to fail. This causes the clay to change from strong brittle soil into a viscous liquid. Furthermore, high-rise buildings, towers, tall flexible structures, etc. founded on sensitive clay usually suffer reduction in safety factor during their life span.

For satisfactory performance of any foundation, there are two conditions to be satisfied; the foundation should not undergo excessive settlement and the foundation should be safe against overall shear failure in the soil that supports it. Foundations, built on sensitive clay and subjected to cyclic loading, may experience both extensive settlement and a reduction in the safety factor against shear failure.

Sensitive clay is found in many parts of the world, but is concentrated in the Eastern Canada, the north-eastern areas of the United States of America, the coastal regions of India, and the Central and Southern coastal districts of Norway and Sweden. Sensitive clays, such as Leda clay, Drammen clay, Champlain clay, St. Jean-Vianney clay, Hong-Kong marine clay and Norwegian marine clay, can be found in different geographical locations and with different levels of sensitivity.

1.2 Problem Definition

In general, the term sensitivity is referred to the loss in undrained shear strength as a consequence of disturbance (remoulding) of the undisturbed material. Terzaghi (1944) introduced the quantitative measure of sensitivity (S_t) as a ratio of the peak undisturbed shear strength (C_u) compared to the remoulded shear strength $(C_u)_r$ with the same water content. That is,

$$S_t = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}} = \frac{C_u}{(C_u)_r} \qquad \text{(1.1)}$$

Sensitivity is based on the actual measurements of the undrained shear strength of the clay. Accordingly, sensitivity is controlled by the properties of the clay in its remoulded state (Bentley, 1979). Figure 1.1 shows the stress-strain curves of a sample of post-glacial clay from the Thames estuary in its undisturbed and remoulded state. The water content was identical in both tests (Skempton et al., 1952). For this sample, the undisturbed shear strength is found 36.5 kN/m^2 and the remoulded shear strength is 4.8 kN/m^2. Consequently, for this clay material the sensitivity $S_t = 36.5 / 4.8 = 7.6$. The remoulding loss and the percentage of remoulding loss are defined by the expression $(C_u - (C_u)_r)$ and $\left[\frac{(C_u - (C_u)_r)}{C_u}\right]*100$ respectively.

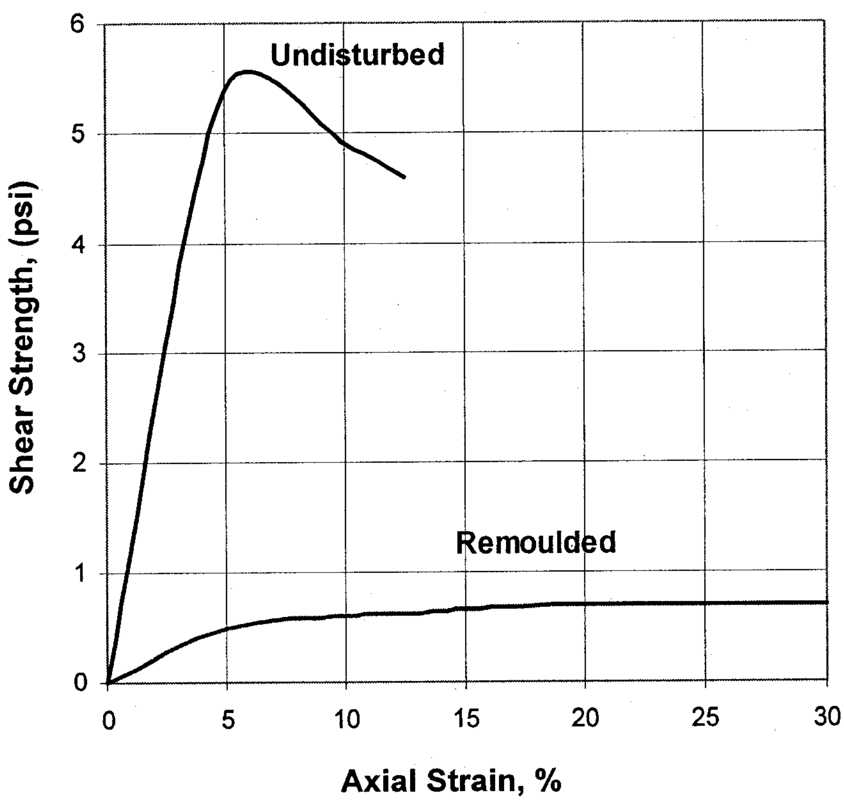

Figure 1.1: Stress Strain Curves for Typical Sensitive Clay (After Skempton et al., 1952) (1 psi = 6.89475 kPa)

1.3 Classification of Sensitive Clay

Several classifications of sensitive clay have been proposed by Skempton et al. (1952), Rosenqvist (1953), and Shannon et al. (1964), as shown in Table 1.1. One difficulty with this sensitivity scale is that it is only concerned with extreme values of the soil strength. It is important to note that clays become quick when the remoulded strength is relatively low.

Table 1.1 Classifications of sensitive clays by different researchers

Skemption and Northey 1952		**Rosenqvist 1953**		**Shannon and Wilson 1964**	
S_t	**Classification**	S_t	**Classification**	S_t	**Classification**
~1	Insensitive	~1	Insensitive	<3	Low
1-2	Low Sensitive	1-2	Slightly Sensitive	3-5	Low to Medium
2-4	Medium Sensitive	2-4	Medium Sensitive	5-7	Medium
4-8	Sensitive	4-8	Very Sensitive	7-11	Medium to High
>8	Extra Sensitive	8-16	Slightly Quick	11-14	High
>16	Quick	16-32	Medium Quick	20-40	Very High
		32-64	Very Quick	>40	Extremely High
		>64	Extra Quick		

1.4 General Problems associated with Sensitive Clay

Sensitive clay is recognized as one type of difficult soils from the Foundation Engineering point of view, as it requires special treatment during the different stages from the site investigation stage to the post-construction stage of the foundation. For example, during sample collection, investigators should observe if samples display a marked tendency to slip out of the sampler tube before reaching the surface. This clay challenges foundation engineers with specific problems concerning sample collection, stability, settlement and prediction of soil response behaviour at the quick condition.

Landslides constitute a major problem due to their disastrous affects on human life and property damage. In the eastern Canada, most flow slides have occurred in the valleys of the St. Lawrence low lands and Ottawa Rivers, and on the southern and the eastern shores of the James Bay area. These regions were formed by the isostatic uplift of the clay and silt deposits in Champlain seawater. In the northeastern United States of America, flow slides have occurred in the varved clay (mostly normally consolidated and sensitive clay) deposited in glacial lakes. In Norway and Sweden, most slides occurred in fjords and valleys of the coastal areas, where marine clays were deposited. Investigation indicates that flow slides are due to the low friction angle, the high cohesion, and the low in-situ stresses of sensitive clays (Eden, 1956; Bilodeau, 1956).

Sensitive clay, especially quick clay, constitutes some of the most troublesome soil conditions. Not only they are extremely soft and very compressible, but also their high level of sensitivity makes it difficult to carry out excavation.

Total and differential settlements are severe problems of foundations on quick clays. The relatively low but heavy masonry buildings that were constructed between 50 and 100 years ago experienced settlements which, in many cases, exceed 0.5 m. In the case of the old town hall, built in 1870 in Drammen, constructed of red bricks with heavy walls and two low towers, the net foundation load varies between 53.4 kN/m^2 (6 ton/m^2) under the main building and 160.2 kN/m^2 (18 ton/m^2) under the towers. The differential settlements are of the order of 40 cm and the building has obviously suffered considerable damage (Bjerrum, 1967).

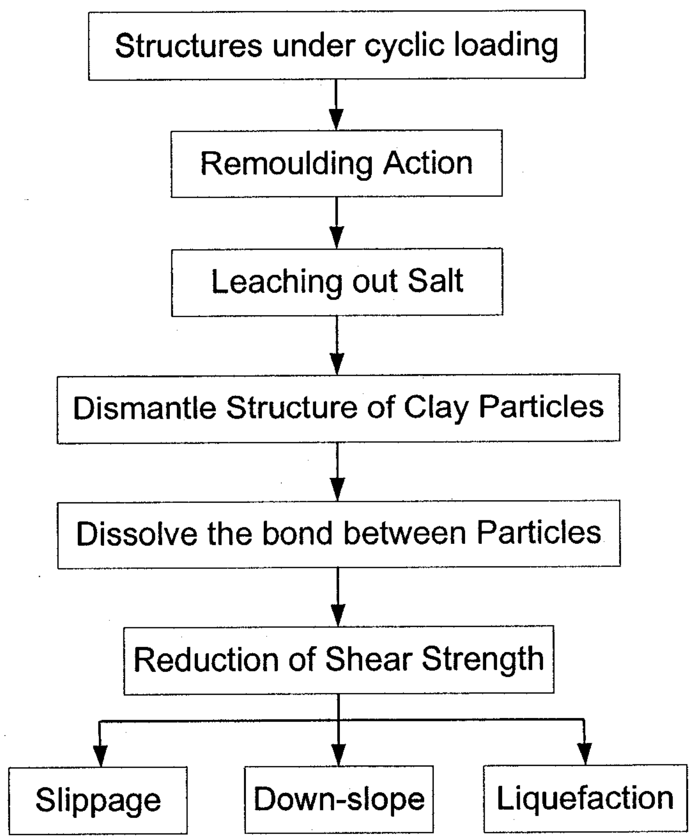

Figure 1.2: Consequences of Cyclic Loading

Cyclic loading causes a remolding action that allows the available water in the soil to dissolve away the salts. This results in reduction of particle interaction and causes a loss in the shear strength of the soil through slippage down-slope or liquefaction.

Structures founded on sensitive clays usually suffer reduction in the safety factor during their life span. Impacts of cyclic loading on sensitive clay are presented in Figure 1.2 in the form of a flow diagram.

1.5 Objectives

The objectives of the present study are to:

1. Conduct a state-of-the-art literature review on sensitive clay under cyclic loading.
2. Document the test results on sensitive clay performed at Concordia University.
3. Document the test results available in the literature.
4. Develop a generalized model relating the cyclic strength to the governing parameters.
5. Develop a procedure as a basic tool for the design and the monitoring of the condition of foundation on sensitive clay.

CHAPTER 2

Literature Review

2.1 General

In 1944, Terzaghi introduced the quantitative measure of sensitive clay as the sensitivity number (S_t). From that time, sensitive clay has been drawing the attention of researchers seeking a better understanding of the sensitivity phenomenon, improvement in determining the design parameters, and establishing inter-relationship between different parameters and the sensitivity number. Recently, the study of the sensitivity phenomenon has become a demanding research field due to the rapid development of land reclamation, infrastructure and housing projects. In addition, cyclic loading acting on foundations built on sensitive clay is causing more damage than other disasters associated with sensitive clay.

The review of published literature is presented in two parts; the first part focuses on reporting the behaviour of sensitive clay and the second part focuses on the performance of sensitive clay subjected to cyclic loading.

2.2 Sensitive Clay

In 1952, Skempton et al. made an effort for exploring the special feature of sensitive clay. They also searched for the causes that may induce sensitivity to some types of clay after remoulding, while remoulding is considered not to effect other types of clay. Due to the limited field data and laboratory results, they concluded that thixotropy might be responsible for causing clay low or the medium sensitive only. Leaching of the original salt concentration in the pore water could develop high sensitive clay. However, Skempton et al. (1952) suggested not assuming the leaching action as the sole cause of the high sensitivity. They identified that micro-structural stability of the clay is the product of sensitivity and the action of leaching is developing a meta-stable structure. Furthermore, they did not observe any significance of mineralogy or particle size distribution in the sensitivity problem for this specific sensitive clay.

Bjerrum contributed to the knowledge about sensitive clay, especially Norwegian Marine clay, based on his experimental observations. In 1954, he reported that leaching causes an essential change in the fundamental properties of clay. Correlating the ratio of the shear strength to the effective overburden pressure, c/p with the plasticity index, he established the concept that shear strength decreases due to the reduction in salt concentrations and the high sensitivity of marine clay is a result of leaching out of salt. Experimental results indicated that, remoulded strength is more sensitive to leaching process than undisturbed shear strength.

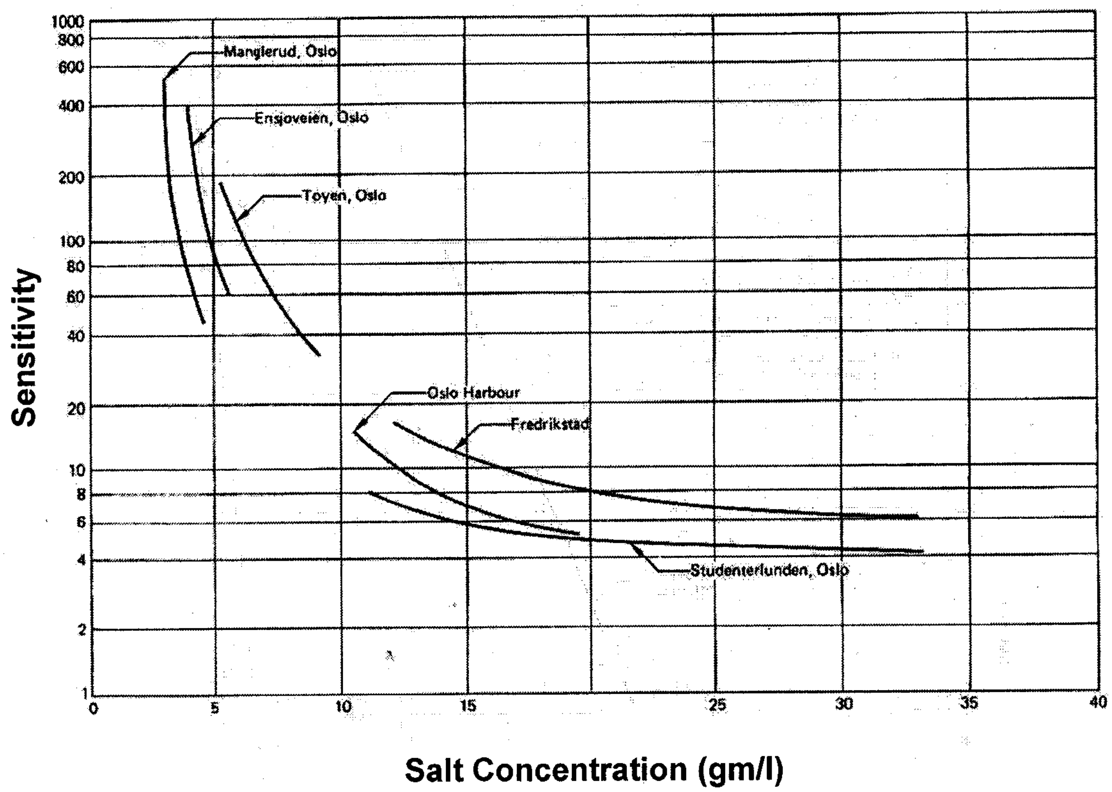

Figure 2.1: Relation between Sensitivity and Salt Concentration for Norwegian Clay deposits (Bjerrum, 1954)

Bjerrum (1954) produced a chart of sensitivity against different salt concentration for sensitive clay of different regions, as shown in Figure 2.1. From this study, it becomes known that the reduction in shear strength does not occur in a linear fashion with respect to the decrease in salt concentration over the last part of the leaching process. He confirmed these findings by analysing a number of slides, which occurred without external causes.

In 1967, Bjerrum reviewed various geological processes that could lead to changes of geotechnical properties (specially compressibility) of Norwegian Marine clays. He illustrated the impact of all the processes causing sensitivity by a collection of settlement records of buildings in Drammen, Norway. Use of compensated foundations on quick clay in Drammen could reduce settlements to 5 cm or less, whereas it might be as high as 90 cm for other types of foundation. Delayed consolidations lead to the development of a reverse resistance on the settlements against compression under additional loads. However, the effect of reverse resistance is well pronounced during the initial period after completion of the buildings. In some places in Norway, the base-exchange process follows the leaching process. This process changes the quick clay to the clay with reduced sensitivity. That means, after the clay has become quick, it will gradually lead to the development of additional strength.

Penner (1965) reported that the sensitivity of Leda clay obtained from the Ottawa area is relatively independent of the salt content in pore water. Sensitivity values varied widely in the low salt content range (less than 2 g/l). He introduced a relationship between the electro-kinetic potential and the sensitivity of the clay. It was indicated that both the sensitivity and the electro-kinetic potential could depend on the nature of the electrolyte. The increase of the electro-kinetic potential of Leda clay was found consistent with the increase in the level of sensitivity. This study showed that sensitivity is more dependent on the average surface area than on gradation of soil particle, at the same electro-kinetic potential. Chemical analyses of the pore water showed that large variation in sensitivity, particularly within one profile, are function of the nature of the pore water electrolyte in the low salt content range. Higher sensitivity depends on the valence of cation contents in pore water.

There is a gap between the prediction and the observation of structural performance of sensitive clay. Some progress has been achieved in the context of the improvement of sampling equipment and testing technique. At least two types of special samplers are presently available for sensitive clay. The Norwegian Geotechnical Institute (Bjerrum, 1954) has designed a thin walled piston sampler to collect undisturbed samples from a depth as great as 25 meters, even for the highly sensitive clay. The agreement, between the shear strength determined by the insitu vane test and by the unconfined compression test, is considered a criterion of undisturbed sampling.

Cylindrical block sampler was the outcome of research by the Geotechnical team of University of Sherbrooke (Lefebvre et al., 1979). This block sampler allows the carving of a clay block at a depth from the surface. Procedure of sampling is almost similar to the conventional block sampling. Conventional block sampling in open trenches has usually been restricted to shallow depths. However, in this special cylindrical block sampling technique, the borehole remains kept full of water or Bentonite mud and accordingly, the tendency of bottom heave is drastically reduced. Samples can be obtained from much greater depth by cylindrical block samplers as compared to those obtained in an open trench.

Analysis of experimental data by Eden et al. (1961) for assessing the sensitivity measurement methods showed that no one method is to be preferred for the determination of the sensitivity. The field vane method was found the most economical one. However, it underestimated the sensitivity at the low liquidity index and overestimated it when the liquidity index is high. The fall-cone test was evaluated the most convenient and economical of the laboratory methods, however, the interpretation of the results from this test remains doubtful.

Numerous researchers had paid their attention to develop techniques to determine reliable shear strength of sensitive clay in the laboratory. The shear strengths produced by the Vane tests agree well with the results produced by the Cone tests and the Quick shear tests up to a certain depth, whereas below this depth they are substantially lower than the Vane strengths. The remoulded strengths of some clay are so low that *unconfined compression* test specimens cannot be formed. As a criterion for the reliability of shear strength determination by unconfined compression test, the strain at failure is limited. Accordingly, the Vane Shear test is often used both in the field and in the laboratory (Mitchell, 1969).

Eide et al. (1972) and Dascal et al. (1972) showed that the vane shear test over-estimates the shear strength in highly plastic clay. In an effort to resolve this problem, a triaxial-vane apparatus was constructed at the Division of Building Research, National Research Council of Canada, Ottawa. Law (1979) described this apparatus and published a series of test results on soft marine clay. He also studied the effect of pressure on vane shear strength. The test results showed that the shear strength produced by the vane apparatus is relatively insensitive to any change in the vertical pressure. However, vane strength increases steadily with the increase of the all-around or horizontal pressure. He explained this phenomenon through an analysis incorporating the concept of strength anisotropy. As a finding of this study, he concluded that the apparent vane strength requires correction for strength gain under sustained load.

Bentley (1979) made an assessment on two remoulded samples of sensitive clay from Gloucester ($S_t \sim 20$) and St. Jean Vianney ($S_t > 200$) using viscometric techniques. He explained that Gloucester material, with significant clay mineral content, has Bingham plastic characteristics and this property will contribute to the remoulded strength and accordingly will not increase its sensitivity. On the other hand, St.-Jean

Vianney material with lower clay mineral content (~ 8% by wt.) has been subjected to a more efficient leaching process. Bentley classified the destructive landslide at St. Jean Vianney in 1971 as a settlement flow with no thixotropic component. From this study it is realized that viscometric analysis may provide a suitable basis for the fundamental study of the behaviour of sensitive clay, the soil characterization and the design of electrochemical ground treatment.

Chagnon et al. (1979) participated in a field trip where the objective was to illustrate various engineering geological problems and their solution with the help of field examples. In that attempt, they conducted a number of field and laboratory investigations regarding the physical properties of sensitive clay. In one landslide zone of quick clay type or sensitive clay of St. Urbain, they suggested installing a screen of vertical drains (geo-drains). This solution was to prevent the slide from moving further back towards and inhabited area. This solution ensures that water pressure will not exceed the hydrostatic level in any case.

Lacasse et al. (1981) conducted pressuremeter expansion tests on two Norwegian clays. Based on these results, it was found that this test method significantly over-predicts the undrained shear strength. Test result is highly dependent on initial pressure. This test produced undrained strength values twice of those obtained from the cone and the vane tests in the laboratory. Moreover, the pressure-meter predicted the insitu Young's moduli significantly higher than the laboratory values.

Silvestri (2003) performed an extensive study on the Cambridge Self Boring Pressure Meter (SBPM) in the sensitive Champlain clay of Quebec. He reported that SBPM deduced undrained shear strengths are 39% (on average) higher than the results obtained from the vane test. He further indicated that strain-rate and progressive failure effects in SBPM should be considered when the vane and the SBPM test results are compared. However, the effect of finite length of the pressuremeter on the pressure-expansion curves and on the deduced stress-strain relationships is still undiscovered.

Hamouche et al. (2000) attempted to assess the performance of wedge-pile system in sensitive clay. They performed this investigation in three sensitive Champlain clay deposits with different overconsolidation ratios. The piles were tested in tension and the test results showed that the average increase in pullout capacity is relatively small (about 30%) in such deposits. As there are some technical difficulties associated with the installation of wedge piles, their use in sensitive clay cannot be recommended.

Torrance (1999) investigated the rheological response of the remolded Leda clay from the South Nation River landslide (1971) site using a coaxial viscometer. He presented a wide range of results representing physical, chemical and mineralogical factors, which influence (control) the remoulded behaviour of these materials. Using a coaxial viscometer, he assessed the effects of the water content, salinity, ion saturation, oxides and hydrous oxides of iron and aluminum, primary silicates, grain-size distribution, and pH change on the remoulded behaviour of the low activity post-glacial marine clay. This study indicated that there is no apparent relationship between the flow type and the tendency to exhibit quick clay behaviour.

The various factors affecting the mechanical behaviour of cemented silty clay were considered in the research carried out by Loiselle et al. (1971). According to their observation, though cementing agent slightly could change the stiffness of the soil structure, it did not show any influence on the shear strength at failure. It was then concluded that cementing bonds most likely were broken at very small strains and therefore, those cannot contribute any substantial increase of the shear strength except under very small consolidation pressure. For this reason, it becomes difficult to study the influence of bonds between particles by any test at the time of rupture. This was also noted for both the triaxial compression and the simple shear tests.

Moum et al. (1971) explained the observed changes in the geotechnical behaviour of sensitive clay due to the geochemical changes and described three different variations of leached marine clay: weathered crust, quick clay and 'desensitized' clay. Weathered crust forms on the surface of all the marine clay deposits. It gives the surface with higher undisturbed and remoulded shear strength (as measured by the Cone test) than the original marine clay. "*Fe*" and "*Al*" acting as cementing agents are responsible for this increased strength (Moum, 1967). When leaching causes pore water salt contents less than 1 g/l, quick clay condition is formed. It is frequently observed that the remoulded shear strength has been reduced to such an extent that the remoulded material behaves as a liquid. There is an additional restriction with regards to the content of specific ions; namely, quick clay was found only where "*Mg*" concentration in the pore water was less than 20 mg/l. The term 'desensitized' has been coined to describe clay that is believed to have been quick at some time in the past but now has reduced sensitivity. Where leaching has occurred from below, quick clay layer lies between the weathered curst and a bottom zone of desensitized clay. This condition is observed at various sites as for instance at the site of the much-studied Ullensaker slide of 1953.

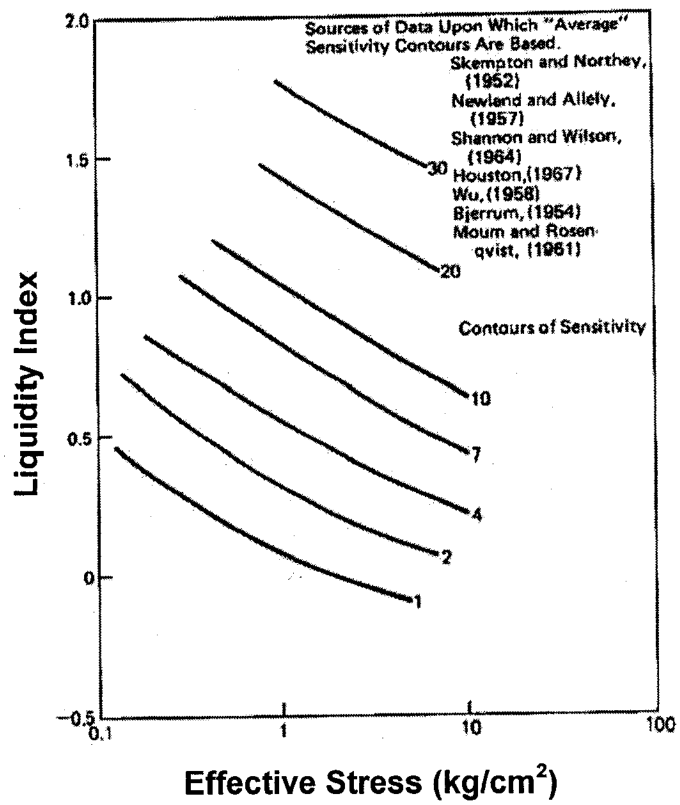

Figure 2.2: Relationship between Sensitivity, Liquidity Index and Effective Stress (Mitchell, 1976)

Mitchell (1976) presented a general relationship between the sensitivity, the liquidity index and the effective stress for sensitive clays. This relation as shown in Figure 2.2 was established by utilizing the tests data from published literature. Although this was developed for normally consolidated clays, it can be used for moderately over consolidated clays if the preconsolidation pressure is used instead of the present effective stress in Figure 2.2. The sensitivity and the strength can be estimated using this relationship if undisturbed samples or in-situ test data are unavailable. Therefore, it was considered as a guide for extrapolating a small amount of data into a larger pattern.

Liquidity index provides the basis for establishment of a relationship between the sensitivity and the water content. In 1954, Bjerrum showed that the sensitivity of a given clay type usually correlates uniquely with liquidity index for Norwegian Marine Clays. Wood (1990) utilized the concept of Bjerrum (1954) in expressing sensitivity and liquidity index in the form of:

$$S_t = \exp(kI_L) \quad \text{...(2.1)}$$

Where, S_t = Sensitivity Number,

I_L = Liquidity Index,

k = a constant describing the variation in sensitivity with the liquidity index.

A value of $k \approx 2$ provides a reasonable fit for Norwegian Marine Clays. This equation implies a clay has sensitivity of 7.4 at its liquid limit ($w = w_L$, $I_L = 1$). A value $k = 0$ implies that the soil is insensitive. Insensitive behaviour is usually observed for clays at or below their plastic limit. So, the above relation should be used only for $I_L > 0$. It should be noted that the sensitivity number (S_t) increases due to an increase of liquidity index (I_L) up to a limit beyond which the clay will be in the liquid state. A value for the constant k = 4 or higher indicates that the clay is highly sensitive, while a value of k =1 or less implies that the clay is low sensitive.

Wood (1990) produced an expression for estimating the undisturbed strength of sensitive clay. This equation is best used for related soils with known values of k and R. The equation is given below:

$$C_u = c_L \mathrm{R}\, exp\left[\left(k - \ln R\right) I_L\right] \text{ for } I_L > 0 \quad \text{...(2.2)}$$

Where, C_u = undisturbed shear strength,

c_L = undrained shear strength of remoulded soil at liquid limit,

c_p = undrained shear strength of remoulded soil at plastic limit,

$R = \frac{c_p}{c_L}$,

I_L =Liquidity Index,

and k = constant describing the variation in sensitivity with liquidity index.

In 1988, Locat et al. studied the rheological behaviour of sensitive clay. They proposed a relationship for the evaluation of the remoulded shear strength (C_{ur}) as the following:

$$C_{ur} = \left(\frac{19.8}{I_L}\right)^{2.44} \quad \text{.....(2.3)}$$

This relationship would be valid for a liquidity index (I_L) between 1.5 and 6.0. Equation (2.3) is valid for the computation of the sensitivity of soils provided the remoulded undrained shear strength is lower than 73 Pa.

Yin (1999) studied the properties and the behavioural parameters (i.e., consolidation parameters) of Hong Kong Marine Deposit (HKMD). He proposed some general correlations, as a function of physical parameter, for estimating consolidation parameters C_c, C_r and C_α for HKMD. These correlations are as follows:

$$C_c = 0.0138 I_p + 0.00732 \quad \text{.....(2.4)}$$

$$C_r = 0.00219 I_p - 0.0104 \quad \text{.....(2.5)}$$

$$C_\alpha = 0.000369 I_p - 0.00055 \quad \text{.....(2.6)}$$

Where, the plasticity index (I_p) is in percentage (%).

2.3 Sensitive Clay under Cyclic Loading

Sangrey et al. (1969) reported the results of undrained shear tests conducted under strain-controlled repeated loading of low cyclic frequencies (about 2.5 cycle/day). The soil samples were normally consolidated and moderately sensitive clay (S_t <10). The objective of these tests was to support the concept of a 'critical level of repeated deviator stress'. Below this level, the soil would reach an elastic equilibrium and above which the failure would occur. Continued increase of the pore water pressure and continued distortional strain were due to cyclic loading phenomena and caused failure of samples.

Mitchell et al. (1977) studied an Ottawa area Champlain Sea Clay through laboratory investigation. The sensitivity of this clay was in the range of 15 to 25. Triaxial test specimens were 5 cm in diameter and 10 cm in height. Undrained static triaxial tests were carried out using a backpressure of 50kN/m^2 and at a nominal axial strain rate of 0.5%/hr. During testing, the effective consolidation pressures ranged from 5-150 kN/m^2. A load-controlled equipment was specially designed for cyclic loading tests. Confining stress was applied by air pressure and undrained cyclic loading was applied at desired frequency through the air space provided at the top of triaxial cell. Tests were performed on isotropically consolidated samples. Loading frequencies of 2 and 15 cycles/min, typical of some long duration field loadings, were employed during cyclic tests. All tests were conducted at effective confining pressures less than $\sigma'_p = 170$ kN/m^2. This condition simulates the normal working stress range for foundations and this type of soil in laboratory. They observed a significant difference in the behaviour expected between weathered and un-weathered clay. Weathered clay was found less resistant to deformation and failure under cyclic loading. They observed that undrained cyclic loading of sensitive clay caused continued increase in the excess pore water pressure. In this case, the rate of increase in the pore water pressure is a function of the initial state of stress, the cyclic stress level and the magnitude of the cyclic stress increment. Accompanied distortional strain at failure is larger for cyclic loading test than those for static loading test. This is considered as a result of the progressive destruction of the soil structure. However, the distortional (or axial) strain and the development of the pore water pressure were not uniquely related rather complex. Number of stress cycles required to cause failure is a function of the magnitude of the initial effective confining stress and the magnitude of the cyclic deviator stress. They observed that usually short duration cyclic loadings (less than 100 cycles) do not cause foundation failure or excessive deformation of sensitive clay, unless static strength is exceeded. To avoid excessive deformation and possible failure, they suggested to apply a factor of safety, not less than 2, to the maximum cyclic shear stress induced in a sensitive soil by foundation elements subjected to repeated loadings.

Eekelen et al. (1978) utilized the existing test results for Drammen clay to develop a model for the behaviour of clay under static and cyclic loading. The static part of the model was an adaptive version of ‘modified Cam Clay’ model. They described the behaviour of Drammen clay under cyclic loading in terms of a single fatigue parameter, i.e., the pore water pressure, u^+, generated by cyclic loading. The cyclic loading model consisted of three formulae:

1. The Pore Pressure relation

$$\frac{du^+}{dN} = A\exp(W/B) \quad \text{for } W > C$$

$$\frac{du^+}{dN} = 0 \qquad \text{for } W \leq C \quad \text{.....(2.7)}$$

Where W = invariant stress amplitude

For Drammen clay, A = 4.54 x 10^5 t/m^2, B = 0.073 and C = 0.20.

2. Stress Strain relation

$$E_c = \alpha \frac{S_c}{\sqrt{(1-S_1)(1-S_2)}} \quad \text{with } \alpha = 0.4\% \quad \text{.....(2.8)}$$

Where, E_c = cyclic strain amplitude measure

S_c = cyclic shear stress level,

S_1 and S_2 are normalized stress levels, α is a parameter of the model.

3. The strength reduction by cyclic loading

$$J_{fc} = b\left\{p_u\left(1-\frac{u^+}{p_c}\right)^{k/\lambda} + a\right\} \quad \text{.....(2.9)}$$

Where, J_{fc} = cyclic stress amplitude at failure,

P_c = consolidation pressure (at start of test),

u^+ = pore pressure generated by cyclic loading,

p_u = undrained virgin pressure,

k/λ is a parameter of the model,

b is a function of the basic parameter M in static loading test, load angle at failure and another parameter of the model, and

a = attraction

If cyclic loading is in action for a short duration (for example, earthquake), the excess pore pressure can dissipate during the calm period. Therefore, the clay will regain much of its original strength and stiffness. However, only a small net effect remains and in the long term significant permanent effects may be induced due to the occurrence of cyclic loading. This model by Eekelen et al. (1978) predicts that normally consolidated or lightly over consolidated clay becomes stronger and more resistant to cyclic loading. On the other hand, a heavily overconsolidated clay becomes weaker and less resistant to cyclic loading. Some experimental evidence reported by Andersen (1976) reinforced this idea.

Lee (1979) performed an extensive research regarding the cyclic strength characteristics of very sensitive clay from a zone of known seismic activity. The physical-chemical aspects of this clay at the Outardes site and of the Leda clay series were similar. The samples were collected from two nearby sites, referred to as soil A (S_t = 380) and soil B (S_t = 35). Both soils were very sensitive. Though soil A was stronger than soil B, soil A converted to a thin fluid upon remoulding. On the other hand, after complete remoulding the weaker clay (B) attained thicker consistency like a dough. He studied experimentally the static and the cyclic (both symmetrical and unsymmetrical loading) behaviour using triaxial apparatus. Some miniature vane and pocket penetrometer tests were also conducted on these samples for assessing the undrained static strength. Triaxial test specimens were 1.4-inch in diameter and 3-inch in length. Total strength approach was employed in the interpretation of all strength data. Loading in cyclic triaxial tests was applied as sinusoidal loading with frequency 1 Hz. Vane tests underestimated the undrained strength, which is 30% lower than that obtained from unconfined compression tests. These sensitive clays were found slightly stronger in compression than in extension, at the same consolidation stress. These sensitive soils exhibited a different mode of failure. In all tests, the specimens reached failure state by shearing along one or more well defined planes at small cyclic strain amplitudes (4-6% for soil A and 2-3% for soil B). The cyclic strength data indicated that at the low effective consolidation pressure, σ_{3c}, there was an increase in cyclic strength due to a slight increase in σ_{3c}. At high σ_{3c} above the preconsolidation pressure, the cyclic strength was found to be significantly larger. Static test results also followed this trend. Moreover, the cyclic strength was found lower for symmetrical loading than that obtained for unsymmetrical loading with respect to the isotropic condition. Anisoptropic test data were observed anomalously high when compared directly with the isotropic test data. However, the pattern of anisotropic test data reflected that of unsymmetrical loading test results.

Raymond et al. (1979) carried out a research work, which was designed to study the behaviour of sensitive, weak, saturated, or almost saturated Leda clay as a sub-grade soil under repeated loading condition. The clay was collected from Casselman, Ontario, South East of Ottawa, and is the part of the Champlain Sea deposits in Quebec, Ontario, and New York State. The investigation was carried out as this sensitive clay forms the part of the sub-grade soils in the Windsor-Quebec City corridor, which would be considered for a new medium or high speed (either 200 km/hr or 300 km/hr maximum) railroad track. Block samples were obtained from a flat agricultural area. Though they also recognized the fact that a sample might have failed before dissipation of excess pore water pressure, they performed conventional drained triaxial tests. In repeated load tests samples were consolidated at a confining pressure of 35 kPa, which simulated a typical sub-grade stress. Each axial difference was repeatedly applied at a frequency of 1 Hz until sample failed or at least 10^5 cycles were reached. Permanent and resilient deformation characteristics resulting from a constant level of repeated load were investigated. It was observed that under repeated loading there is a threshold stress. This threshold value separates the behaviour of the soil into two different modes. In this specific case, the threshold stress was observed 54% (approximately) of the static shear strength. Below this level, permanent deformations increased continuously with the increase of number of stress cycles even at low stresses. Above this level, without warning sample could fail after relatively lower number of cycling. Raymond et al. (1979) noted that a small change in the stress difference factor must have a major impact on the subgrade life, particularly if loaded near the critical or threshold value. They suggested the necessity for visco-plastic and (or) non-linear elastic analyses for the design purposes involving this material when subjected to variable repeated loading.

Houston et al. (1980) performed an experimental investigation with the objective of quantifying the undrained response of seven seafloor soils to various combinations of static and cyclic shear loads. All the samples were consolidated anisotropically before loaded cyclically. The principal effective consolidation stress of about 29.4 kN/m^2 was chosen as it represents the seafloor condition of about 6 m depth. In some cyclic triaxial tests, after consolidation, an initial static bias was applied to the specimen (undrained) and then the cyclic stress at a frequency of 2 Hz was superimposed. The static bias is also called a static component of loading. For the other cyclic specimens, tests were performed under "pure stress reversal" condition. Only the bay mud exhibits a moderate sensitivity of about 8 and all of the other soils have low sensitivities of about 3 or less. Any generalized relationship between the sensitivity and the cyclic resistance had not been established and could not be quantified. Data showed that bay mud has a very high resistance as compared to all other soils utilized in this investigation though it has the highest sensitivity and the highest plasticity index. They concluded that the determination of a precise value of CLRL (Critical Level of Repeated Loading – a level below which failure will never occur regardless of the number of cycles) for a specific field application might be a difficult and complex process. Where partial drainage condition has to be accounted for, an effective stress approach was found necessary. They showed quantitatively that a steady increase in the normalized cyclic resistance depends on the increase in plasticity. They predicted the importance of degree of bonding between particles when considering cyclic resistance.

Lefebvre et al. (1987) utilized the results of a series of monotonic and cyclic triaxial tests to study the influence of the strain rate and the load cycles on the undrained shear strength of three undisturbed sensitive clays from Eastern-Canada. Triaxial tests were conducted both on isotropically and anisotropically consolidated samples. Application of back pressure about 100 kPa improved de-aeration and saturation of the triaxial system. Some cyclic triaxial tests were also conducted on de-structured specimens at two different vertical consolidation pressures (1.8 and 1.1 times σ'_p). Symmetrical cyclic deviator was applied under stress-controlled conditions at a frequency of 0.1 Hz with a double entry pneumatic cylinder. They noted that shear strength is dependent on the strain rate for both structured clays (naturally over consolidated) and de-structured clay (normally consolidated). The relation is observed linear for at least five-log cycle for strain rate. Cyclic strength of clay at a given frequency could be approximately evaluated from a monotonic test result by applying a correction for the strain rate effect to obtain the monotonic strength at the given frequency. Then the approximated cyclic strength requires a further correction by applying a degradation function to obtain the mobilized cyclic strength for a given number of cycles.

Hyodo et al. (1994) conducted a series of undrained cyclic and monotonic triaxial compression and extension tests on highly plastic marine clay. Various combinations of initial static and subsequent cyclic shear stress on isotropically and anisotropically consolidated specimens were applied during testing. They gave some definitions for the terms like effective stress ratio (η), cyclic shear strength (R_f), index of the possibility of cyclic failure (RR (N)) and relative effective stress ratio (η^*). From this study, a unique relationship was recognized between peak axial strain, ϵ_p and mobilized effective stress ratio measured for each stress cycle during cyclic loading, despite various initial static and subsequent cyclic shear stresses. The relationship was as follows:

$$\varepsilon_p = \frac{a_1 \eta_p}{1 - \eta_p / \eta_{ult}} \quad \text{...(2.10)}$$

The best-fit curve for this relationship was a hyperbola.

They proposed a unified definition of cyclic shear strength for both reversal and non-reversal stress conditions. Firstly, the cyclic failure was defined as the peak accumulated axial strain ε_p = 10%. However, for the purpose of development of the empirical model, cyclic strength was redefined as:

$$R_f = \left\{ \frac{q_{cyc} + q_s}{p_c} \right\}_f = \kappa N^{\beta} \quad \text{...(2.11)}$$

Where, q_{cyc} = Cyclic deviator stress,

q_s = Static deviator stress,

p_c = Effective mean principal stress after consolidation,

κ = Cyclic strength for the first cycle

$= 1.0 + 1.5 \dfrac{q_s}{p_c}$

$\dfrac{q_s}{p_c}$ = Initial static deviator stress ratio,

and β = -0.088.

A unique correlation between the new parameters RR(N) and η^* was presented. The first parameter was defined as an index of the possibility of cyclic failure, RR(N) = R/R_f. It is the ratio of the peak cyclic deviator stress, R = $q_s + q_{cyc}$, to the cyclic shear strength R_f = $\left\{ \dfrac{q_{cyc} + q_s}{p_c} \right\}_f$ in a given number of cycles. The second parameter was defined as relative effective stress ratio,

$$\eta^* = \frac{\eta_p - \eta_s}{\eta_f - \eta_s} \quad \text{...(2.12)}$$

Where, η_p = Effective stress ratio at the peak of the cyclic stress in each cycle,

η_s = Effective stress ratio for initially consolidated condition,

η_f = Effective stress ratio at failure.

These two parameters were uniquely correlated by the following equation:

$$\eta^{*} = \frac{R/R_f}{\left\{a_2 - \left(a_2 - 1\right)R/R_f\right\}} \quad \text{...(2.13)}$$

Where, a_2 was determined experimentally as 6.5. This equation was independent of the magnitude of initial static deviator stress and intensity of cyclic load. A unique non-linear relationship was found between the post cyclic volumetric strain and the cyclic induced pore pressure.

In 1999, Hyodo et al. studied the cyclic behaviour of undisturbed and remoulded marine clays obtained from coastal locations of Japan. Triaxial tests were conducted on these samples. In this study, the influences of plasticity index, aging factor and over consolidation ratio (OCR) on the undrained cyclic strength of clays were separately examined. It was observed that if the clay is normally consolidated, both the remoulded and the undisturbed samples have the same cyclic strength and cyclic strength increases with plasticity index. A unique relationship between the cyclic strength ratio and the plasticity index was given for normally consolidated marine clay (remoulded and undisturbed). This relationship is called as intrinsic cyclic strength that is as follows:

$$R_1 = 0.0007 I_p + 0.25 \quad \text{...(2.14)}$$

This relation is valid for remoulded and undisturbed clays with no aging and sedimentation effects. But for undisturbed samples with a residual yield stress ratio, $\sigma'_{vy} > \sigma'_c > \sigma'_v$, effect of aging was observed on the intrinsic cyclic strength R_1. The deviation of the aged cyclic strength, R_L from the intrinsic cyclic strength ratio R_1 was defined as R_2 (= $R_L - R_1$). This deviation is greater for samples with a greater initial yield stress ratio. For soils with initial yield stress ratio, $\sigma'_{vy} > \sigma'_c > \sigma'_{v0}$, an aging parameter α was introduced,

$$\alpha = \frac{\sigma'_{vy} - \sigma'_c}{\sigma'_{vy} - \sigma'_{v0}} \quad \text{.............(2.15)}$$

The value of α ranges from 0 to 1. This parameter was applied for calculating the normalized functions of aging correction factor R_2.

$$R_2 = 0.044\left(\frac{\sigma'_{vy}}{\sigma'_{v0}}\right)\alpha \quad \text{.............(2.16)}$$

Furthermore, the analysis of the test results showed that the deviation (R_L – R_1 –R_2) increases with increasing the overconsolidation ratio. Hyodo et al. (1999) proposed a new factor for including the effect of the overconsolidation into intrinsic cyclic strength as follows:

$$R_3 = R_L - R_1 - R_2 = 0.17\left(\frac{\sigma'_{v0}}{\sigma'_c} - 1\right) \quad \text{.............(2.17)}$$

In fact, they proposed .to determine the cyclic strength as an intrinsic function of the plasticity index, I_p and then modify it for including vertical yield stress ratio $\frac{\sigma'_{vy}}{\sigma'_{v0}}$ or 'aging' effect and geological OCR. For isotropically consolidated remoulded or undisturbed clays ($\sigma'_c > \sigma'_{vy}$):

$$R_L = R_1 \quad \text{.............(2.18)}$$

In the case of undisturbed aged clay, where $\sigma'_{v0} < \sigma'_c < \sigma'_{vy}$, then:

$$R_L = R_1 + R_2 \quad \text{.............(2.19)}$$

In the case of overconsolidated undisturbed clay:

$$R_L = R_1 + R_2 + R_3 \quad \text{......(2.20)}$$

In the case of over consolidated remoulded clay:

$$R_L = R_1 + R_3 \quad \text{......(2.21)}$$

The cyclic strength of anisotropically consolidated clays was also investigated. In this case, cyclic strength was found as a function of plasticity of the clay and the drained shear stress ratio. Investigation showed that anisotropic cyclic shear strength ratio (to cause failure at 20 cycles) normalized by isotropic cyclic strength is a function of initial drained shear stress ratio. This function depends on the plasticity index but independent of the stress history. The proposed function is as follows:

$$\frac{R_{(N=20)}}{R_{ISO}} = a\left(\frac{\sigma_s}{2\sigma'_c}\right)^2 + 1 \quad \text{......(2.22)}$$

Where, $a = -0.032 I_p + 0.725$,

I_p = plasticity index,

$\dfrac{\sigma_s}{2\sigma'_c}$ = initial drained shear stress ratio.

Javed (2002) analyzed the results of the conventional consolidation test and studied the deviator stress versus axial strain relationship under cyclic loading of undrained and drained condition. The effect of cyclic loading on the pore water pressure measurements was examined. He pointed out that the higher the value of sensitivity the faster the clay looses its strength under cyclic loading. Undrained condition is the most critical phase for sensitive clay as compared to the drained condition. Specially, he observed that under cyclic loading condition it is possible to reach equilibrium state.

2.4 Discussion

By reviewing the literature on sensitive clays, numerous research studies explored the properties of sensitive clay in order to get a clear conception about their unusual characteristics. Before the definition of sensitivity number given by Terzaghi in 1944, foundation engineers were unable to identify this type of clay, and accordingly, the problems associated with it. At that time, it was only known that in some regions, clays are responsible for most of the liquefaction, the catastrophic disasters, and also cause the difficulties during soil exploration and construction processes. Terzaghi gave the idea of the definition 'Sensitive Clay' among all clay soils and identified the characteristic feature of this group. Sensitivity number, S_t, is a mechanical parameter that provides a useful basis for the clay to indicate the level of its sensitivity. In the literature, researchers give several classifications for sensitive clay. Before the 50's, studies on this field were based on relatively small-scale experiments and this research area was young. For example, Rosenqvist (1953) attempted to strengthen sensitive clay artificially in order to increase its stability against disturbance. However, no decrease in the problems related to sensitivity was achieved. In short, the reasons behind this unusual behaviour of sensitive clay remained undiscovered at that time.

Skempton et al. (1952) and Bjerrum (1954, 1967) looked for natural processes that could induce the sensitivity property in clays. According to their explanation, thixotropy alone could account for the low or the medium sensitivity, but it was unable to account for the cause of the high sensitivity. According to them, mineralogy or particle size distributions do not have any major significant influence on sensitivity. Based on the studies on Norwegian Marine clays, researchers identified the action of leaching out of salt from pore water as the factor responsible for causing the high sensitivity. The process of leaching enhances the development of meta-stable structure, which is the property of sensitive clay. It has become known that leaching causes a reduction in the remoulded shear strength as compared to the undisturbed shear strength. However, the reduction in shear strength does not occur in linear proportion with the decrease in salt concentration. According to the research findings by Moum (1971), in the case of sensitive clay formation there is an additional restriction with regard to the content of specific ions along with the leaching process. As an example, leaching out of salt provided Mg concentration in the pore water is less than 20mg/l can form quick clay. The leaching process is followed by base-exchange process. This explains the reason that clay formed by leaching will gradually attain additional strength. Based on the viscometric assessment conducted by Bentley (1979) it can be noted that sensitive clay with lower clay mineral content would be subjected to more efficient leaching than that with significant clay mineral content. On the contrary, in the case of Leda clay from the Ottawa area, Penner (1965) observed that the sensitivity is relatively independent of the salt content in the pore water.

In addition to geotechnical investigation on sensitive clay, different researchers also investigated physical, chemical, and mineralogical factors influencing the behaviour of sensitive clays, and studied various aspects as well as natural processes affecting their mechanical behaviour. As reported by Loiselle et al. (1971), it was found that presence of cementing agents cannot increase the stiffness of soil structure significantly and shear strength at failure.

Some researchers paid attention to developing better sampling and testing techniques for the assessment of sensitive clay behaviour. Conventional samplers and testing procedures failed to reflect the actual situation. This explains the discrepancies between the prediction that was based on test results of the collected samples, and the performance of the structures built on sensitive clay.

Practically, the Self-boring pressuremeter (SBPM) test has the potential of measuring the entire stress-strain curve and insitu horizontal stress. Lacasse (1981), indicated that this technique overestimates the undrained shear strength of the clay due to the disturbance caused by probe insertion. Furthermore, the produced undrained shear strength values were twice of those obtained from the cone and the vane tests in laboratory. On the other hand, for the Champlain clay of the Eastern Canada, the undrained shear strength deduced from the SBPM test results which were found on average 39% higher than the corresponding the vane deduced values (Silvestri, 2003). Silvestri indicated that, it is not the SBPM test that overestimates the undrained shear strength, but rather the vane test that underestimates it for sensitive clay. Furthermore, in the case of comparing the vane and the SBPM test results, the strain rate and the progressive failure effects are identified as important factors.

Based on the above discussion, it becomes clear that the estimation of any special parameter, such as the strength parameters, the consolidation parameters, and the sensitivity for sensitive clay, is not an easy task. For this reason, many attempts were made for establishing a general relationship so that those parameters could be evaluated from the index properties determined in the laboratory. From the present literature review, some equations for determining the sensitivity, the undisturbed strength, the remoulded strength and the consolidation parameters can be found. A general relationship between sensitivity, effective stress and liquidity index was given by Mitchell (1976). However, it should be noted that all these relationships are only valid within certain limits.

In fact, from the very beginning, the objectives of most of these studies were to understand the sensitivity phenomena, improve its sampling and testing techniques, and assess their properties and behaviour. The problem related to sensitivity in sample collection has been eliminated by developing new sample tools; problems in excavation can be reduced by strengthening the soil artificially beforehand and after all settlement of simple four-five stored buildings can be limited within tolerable limits by adopting the principal of compensated foundation. The problems with sensitive clay under cyclic loading are the most critical one, which presents new difficulties in the design stage. Some sensible research works were carried out on cyclic loading in soft and saturated clay region but only a few are dealing with marine and sensitive clay.

An empirical cyclic loading model in terms of a single fatigue parameter i.e., pore water pressure generated by cyclic loading (Eekelen et al., 1978), and a semi empirical model for cyclic strength (Hyodo et al, 1994) are available. Furthermore, there is another approach in which cyclic strength has been defined by an intrinsic function of plasticity index and this strength has to be corrected to include the effect of aging and overconsolidation ratio. This concept for evaluating cyclic strength was proposed by Hyodo et al. (1999).

Cyclic strain of sensitive clay can be evaluated from monotonic tests provided that strain are corrected for strain rate effect and mobilized degradation due to number of cycles (Lefebvre et al., 1987). In the literature, the existence of two critical values regarding cyclic loading on sensitive clay was found; these are the threshold stress defined by Raymond et al. (1979) and the critical level of repeated loading (CLRL) given by Houston et al. (1980). In the case of Leda clay, threshold stress was found as 0.54 times the static failure strength, approximately. On the other hand, CLRL has not been modeled yet.

CHAPTER 3

Analysis of Experimental Data and Design Procedure

3.1 General

The experimental data available in the literature and the data of the tests performed at Concordia University are analyzed in this chapter. Moreover, the geological, the chemical, the physical, the structural and the rheological aspects of sensitive clay are critically reviewed. The interdependency of different parameters is explored to attempt to develop a generalized model incorporating all the parameters influencing the mechanical behaviour of sensitive clay. Finally, a step-by-step design procedure is developed for the design of foundations on sensitive clay under cyclic loading condition.

3.2 Experimental Investigation

The results of Laboratory tests conducted at Concordia University were made available to me during this study. The objective of these tests was to examine the behaviour of sensitive clay under cyclic loading. Samples of sensitive clay were collected from the city of Rigaud, Quebec. This site represents the heart of the Champlain Clay deposit in Eastern Canada and is known for its sensitive clay deposits. The soil was post-glacial Champlain Marine Clay with varying properties including inter-beds or lenses of silt or sands. The clay was described as brittle and sensitive, massive and blocky, with horizontal layers ranging in thickness from millimetre to a few centimetres. The samples were obtained as blocks of 30 cm x 30 cm x 30 cm from a depth of 4 meters under the expert supervision of the company 'Geocon'.

Disturbed samples were used for the physical and the index properties tests. Moreover, the undisturbed samples were collected with care, avoiding all sorts of possible occurrence of vibration. These samples were utilized for unconfined compression, conventional consolidation and both static and cyclic triaxial compression tests. During the testing program, the samples were kept totally covered with the aluminium foil and then with the wax coating to avoid the loss of water content, and maintained at a specific temperature.

The clay was classified according to the Unified Soil Classification System (USCS) as a highly plastic clay soil (CH). The natural water content was (w_c) =71%, liquid limit (LL) = 69%, plastic limit (PL) = 44%, plasticity index (I_p) = 25.6% and the specific gravity (G) = 2.74.

The unconfined compression tests were conducted on undisturbed and remoulded samples to evaluate the sensitivity number (S_t) of this Champlain clay. Soil specimens were prepared in a cylindrical shape with a ratio of 2 (height): 1 (diameter). Dimensions of the diameter and the height were 3.81 cm and 7.62 cm, respectively. The loading was rapid enough to ensure that the water content of the soil remained constant during testing.

Simultaneous measurements of the load and of the vertical displacement of the tested sample were taken. The failure load or, if the sample did not fail outright, the load required to produce 20% strain was expressed as the failure load. These values were used to calculate the undrained undisturbed shear strength of the clay. Samples were then remoulded to form another cylindrical specimen without changing its water content. In the preparation of the remoulded sample, maintaining uniformity and avoiding any trapped air were minimized. Remoulding was performed rapidly with fingers over a thin plastic sheet and repacked with the thumb in about 6.4 mm layers in a 3.81 cm diameter tube against the end of a dolly, which was progressively moved back down the tube. Removal of the sample from the tube was facilitated by a film of oil on the surface of the tube, and a paper disc at the end of the dolly. The ends were trimmed with the use of wire saw and frame. At the end of each test, the water content was again measured. If any change was noted, a correction was made from a plot of water content against the logarithm of the undrained shear strength of the remoulded soil. The unconfined compression testing of the remoulded sample gave the remoulded shear strength of the sensitive clay. Sensitivity numbers were then calculated and found in the range of 6.5 to 9.5. This Champlain clay has been identified as medium to high sensitive. Upon remoulding these samples, the clay became like thick dough.

The conventional consolidation tests were performed on selected samples. Figure 3.1 presents a schematic diagram of consolidometer used in conducting these tests. An undisturbed soil specimen was carefully trimmed and placed into the confining ring of the consolidometer. Dimensions of the ring were 6.355 cm in diameter, 1.94 cm in height and 31.72 cm^2 in cross-sectional area. The ring was relatively rigid so that minimal lateral deformation could take place. Porous stones were placed on the top and the bottom of the ring to allow drainage during the consolidation process. Porous stones were made of porous brass. The top porous stone had a diameter approximately 0.5 mm smaller than the ring to allow direct loading of the specimens. The ratio of the diameter to the height of the specimen was about 3.3. In this testing program, the choice of specimen size was fixed by the size of the ring available in the laboratory. Smaller diameter specimens have more trimming disturbances, while larger diameter specimens have greater side friction. Javed (2002) has reported that the coefficient of consolidation (C_v) was in the range of 0.101 mm^2/sec at 277.32 kPa (loading) to 1.509 mm^2/sec at 138.66 kPa (unloading), which are the typical values for soil of low permeability. Furthermore, the compression index (C_c) was equal to 0.00498, which supports that the clay is fairly compressible and the typical of brittle sensitive clay.

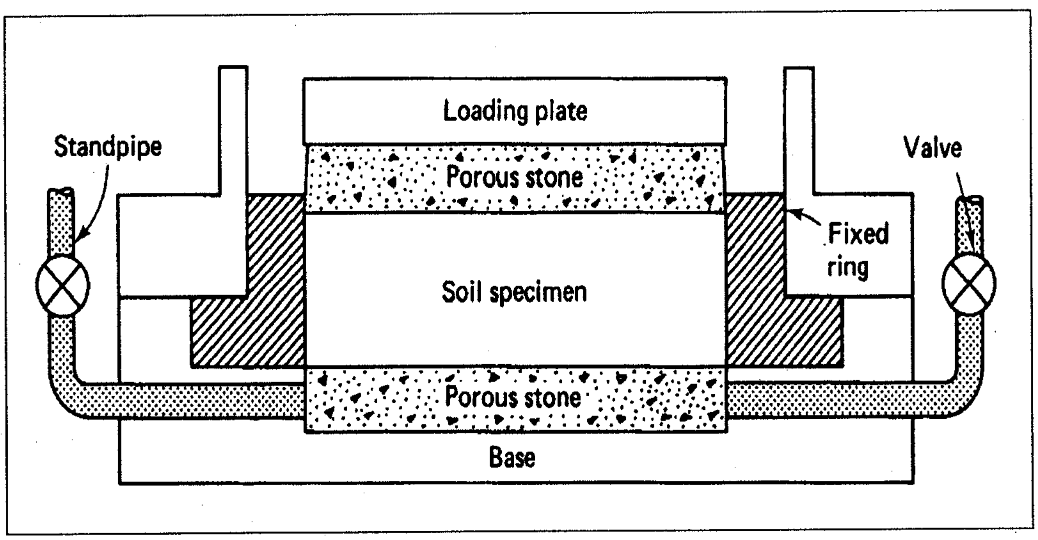

Figure 3.1: Schematic of a Fixed-ring (Odeometer) Consolidation Test Apparatus

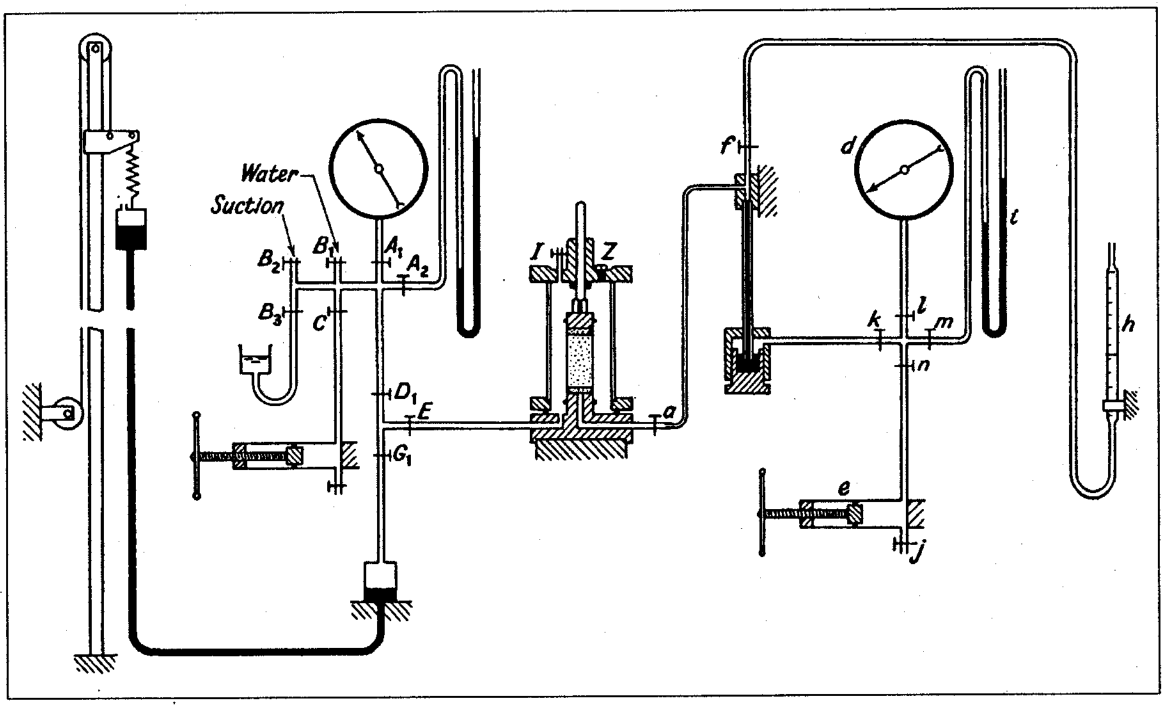

Figure 3.2: Triaxial Test Apparatus (Bishop et al., 1962)

The static undrained triaxial compression tests were performed on samples having a cross-sectional area of 0.0009 m^2. Cylindrical sample had the same dimensions of those used for the unconfined compression testing. Standard consolidated undrained triaxial compression tests were performed on these soil specimens. A schematic diagram of a similar triaxial testing apparatus is presented in Figure 3.2. In this Figure, two pressure gages are found; the pressure gage, *d*, is used for to measure the pore-water pressure while the other gage records the cell pressure. The air valve, I, is used for releasing the trapped air. The plug, Z, introduces the oil into the cell. Except h and I, the other letter designations represent the location of valves used for various purposes. Closing of valves A_1 and A_2 allows most of the water from the cell to flow down the suction line. Valve *a* is the connection between the pore-water pressure device and the cell. Valves B_1, D_1 and E remain open when water is allowed to enter the cell. Opening of the valve B_3 enables the cell pressure to reduce to atmospheric pressure. After closing of valve I, valve C is opened to raise the cell pressure forcing water from control cylinder on the pressure side into the cell. When mercury control is connected to the circuit, valve C is then closed. Valve f is closed for achieving the undrained condition after consolidation. Closing valve G_1 is the connection between the mercury control and the cell.

The soil specimen was first consolidated under the all-round pressure in the triaxial cell before failure was reached by increasing the major principal stress. Pore water pressure measurements were taken occasionally. The specimen was consolidated under isotropic consolidation pressure simulating the level ground condition.

Before preparing the samples, a 10 c.c. burette is connected to the pore pressure outlet on the base of the cell through a piston valve *a*. The whole system was then de-aired and filled with water. This was accomplished by inverting the cell base in a dish of de-aired water and by applying suction to the open end of the burette. Rapid opening and closing of the valve facilitates the removal of any bubbles, which may lodge in the cell base or in the valve. When no further air bubbles were observed in the burette, the valve was shut and the suction line was removed. The sample of clay was placed on the porous disc, and if required to accelerate consolidation, filter paper drains, which had been saturated with water, were placed in position around the specimen. The rubber membrane was placed over the sample using a membrane stretcher and the lower part of the membrane was sealed to the pedestal with the two O-rings. In order to remove as much air as possible from between the rubber membrane and the specimen, the rubber membrane was gently stroked in an upward direction before the upper porous disc and the Perspex loading cap were placed in position and sealed with two more O-rings. Firm seating of the specimen on the pedestal thus had been ensured. The cell was assembled, care being taken with the alignment of the top of the cell as the ram was inserted into the guide in the loading cap. De-aired water was introduced into the cell from the main supply with the air release valve I open until the cell is nearly full. About 6.5 mm depth of oil was then introduced into the cell at the plug Z. Any remaining air was expelled through the air valve by admitting more water.

The cell pressure was raised to the desired value using the screw control and the mercury pressure system was then brought into the operation. The water level in the burette was adjusted to a suitable height to allow the decrease or increase in volume of the sample to be measured. Consolidation was complete when no significant movement of the water in the burette occurred.

For the compression test, the cell was transferred to the testing machine and the pore pressure system was connected to the base of the cell. In order to make the pore pressure connection without trapping any air, the cell was placed in a tray containing water standing to a depth of about 3.81 cm. The volume-measuring unit is then removed. With the burette h standing so that its water level was slightly above the water level in the tray and with the valve f open, a little water was allowed to flow from the pore pressure apparatus through valve a before it was connected to the base of the cell.

The cell was placed on the loading platform of the testing machine and the proving ring brought into the contact with the ram. The burette h was adjusted so that its water level became at the mid height of the specimen and the zero reading of the pore pressure apparatus was checked before closing valve f. When a large negative change in pore pressure was expected to develop as the deviator stress was applied, the cell pressure should be raised at this stage by an amount at least equal to the anticipated drop in pore pressure. The initial pore pressure will then be high enough to prevent the occurrence of any substantial negative values.

With the motor drive running, the zero reading of the proving ring was recorded. The ram was brought into contact with the loading cap of the sample by the hand control and the zero of the strain indicating dial was set using the adjustable arm. The test was then started. Readings of the proving ring dial and the pore pressure were taken at intervals until a peak deviator stress was reached.

After failure had been reached the pore pressure system was isolated by closing valve a. The sample was unloaded and the zero reading in the proving ring was checked. The cell pressure was then reduced and the water and the oil removed from the cell. The top of the cell was taken off and the surplus water was wiped out from the loading cap. The O-rings and the cap were then removed and the rubber stripped from the sample. The deviator stress was determined using the proving ring calibration. Corrections to the calculated stresses were made to allow the effects of the drains and rubber membrane.

In this series, both the confining stress and the initial pore water pressure remained at 207 kPa and 344.75 kPa, respectively, for all tests. The deviator stress at failure varied in the range of 87.9 kPa to 97.56 kPa.

Cyclic undrained triaxial tests were performed on the samples with sensitivity 7. In this series, samples were allowed to consolidate isotropically under the effective confining pressure before testing for simulating the level ground surface condition in the field. Prior to testing, drainage time was sufficient to allow complete dissipation of excess pore water pressure before starting the tests. A backpressure was applied to ensure the full saturation of the sample. The cyclic testing program was devised where the cyclic deviator stress was taken equal to 33%, 35% and 67% of the ultimate value deduced from the static triaxial compression test results. Cyclic loads were applied in only compression phase. Applications of the cyclic load during testing were achieved through vertical loading and unloading. Loading and unloading had been performed incrementally. Loading was incremented in at least two stages while unloading followed the opposite path of loading. The cyclic deviator stress was reduced to zero and then reloaded incrementally up to the predetermined cyclic deviator stress expressed as percentage of static deviator stress. Static deviator stress at failure was 97.56 kPa. This gives the value of static undrained strength as 48.6 kPa. Loading frequency of 15 cycles per minute was used. These test conditions were chosen to relate the field conditions where the foundation elements subjected to wind or wave forces and these would induce cyclic stresses in the surrounding soil. The pore water pressures and the volume changes were recorded on a regular basis. During unloading, the pore water pressure measurements were taken to observe when and whether equilibrium was reached after a certain number of cycles. In this experimental investigation, tests were repeated at least twice for reproducibility of test data, and the number of cycles required to develop a shear plane or failure plane was considered as the number of cycle at failure.

3.3 Analysis of Static Test Results

Experimental results of soil samples tested under static condition, reported by Chagnon et al. (1979), are graphically presented in this section with the intention to investigate the effect of the chemical and the physical aspects on the static strength behaviour of sensitive clay, as well as on the level of sensitivity.

3.3.1 Effect of Chemical Factor

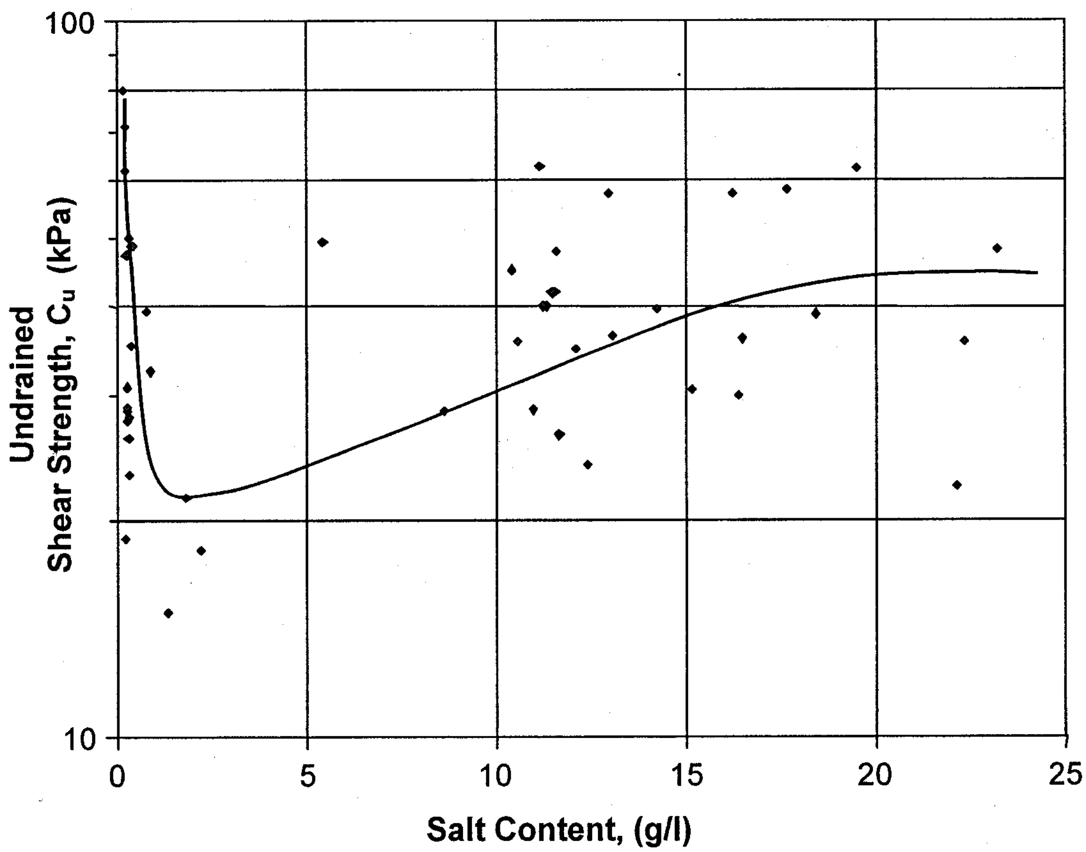

Figure 3.3: Undisturbed Undrained Shear Strength, C_u vs. Salt Content (Data source: Chagnon et al., 1979)

The influence of chemical factor is investigated herein with respect to the total salt content in the pore water of the sensitive clay. In a semi-logarithmic plot, Figure 3.3 presents the test results; reported by Chagnon et al. (1979), in terms of the undrained shear strength, C_u versus the salt content. It can be noted from this Figure that salt has a significant effect on the C_u values. Specifically, increasing salt concentration up to a very small amount (about 2.5 g/l) the undrained shear strength decreases dramatically. Due to further addition of salt, the clay exhibits increase of strength. At a relatively higher salt content, the undrained undisturbed shear strength becomes stable and remains almost constant. This gradual increase in shear strength confirms the behaviour of sensitive clay as noted by Skempton et al. (1952) and Bjerrum (1954). In the literature, no information is found regarding the behaviour of this type of soil during the last part of leaching process, i.e., at zero or very low range of salt content. At very low range of salt content, the compositions of the remaining salt content rather than the amount of the total salt influences the strength properties greatly. Unusual behaviour at low salt concentration is due to the effectiveness of interaction capabilities of the remaining salts. Moreover, it seems that different kinds of salt react differently. All of these factors explain the wide discrepancies observed among the test results. This can be further explained by the fact that the chemical and the mineralogical behaviour are governing the strength parameters of this type of clay at low salt content. Similar observations can be noted for the remoulded shear strength, $(C_u)_r$ versus the salt content, as shown in Figure 3.4.

Both the static undisturbed and the remoulded shear strength will drastically change due to the change of the salt content and/or the water content. Accordingly, sensitive clay with higher water content and low salt content will resist lower static loading as the particles can slip over each other very easily. However, as sensitivity is measured as the ratio of undisturbed shear strength to remoulded shear strength. As both the undisturbed and the remoulded shear strength are reducing due to the leaching process, it can be stated that change in salt contents may not cause any change in the sensitivity value of the clay. From Figures 3.3 and 3.4, it should also be noted that a reduction in the remoulded shear strength is more critical than that in the undisturbed shear strength during leaching action. Due to this fact, it can be stated that the remoulded shear strength, i.e. property in remoulded state, controls the sensitivity property.

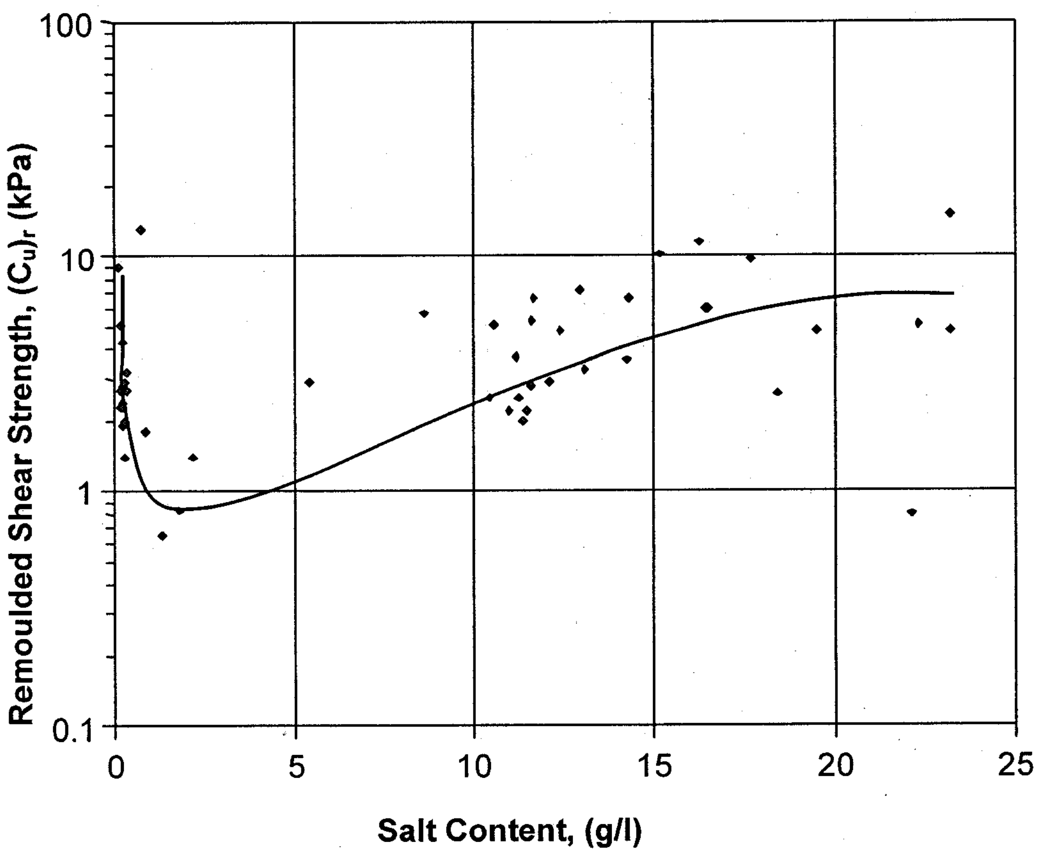

Figure 3.4: Remoulded Shear Strength, $(C_u)_r$ vs. Salt Content (Data source: Chagnon et al., 1979)

Figure 3.5 presents the experimental data as the sensitivity, S_t versus the salt content of the soil. Sensitivity number decreases due to an increase in the salt content. This was also observed by Skempton et al. (1952) and Bjerrum (1954). Moreover, it can be stated that at relatively low salt content, sensitivity becomes independent of total salt content. A similar observation was made by Penner (1965) for an Ottawa area clay with low salt content. As mentioned earlier, sensitivity depends on the fluctuation of the remoulded shear strength during this stage, which is further related to the inherited chemical factor of this material.

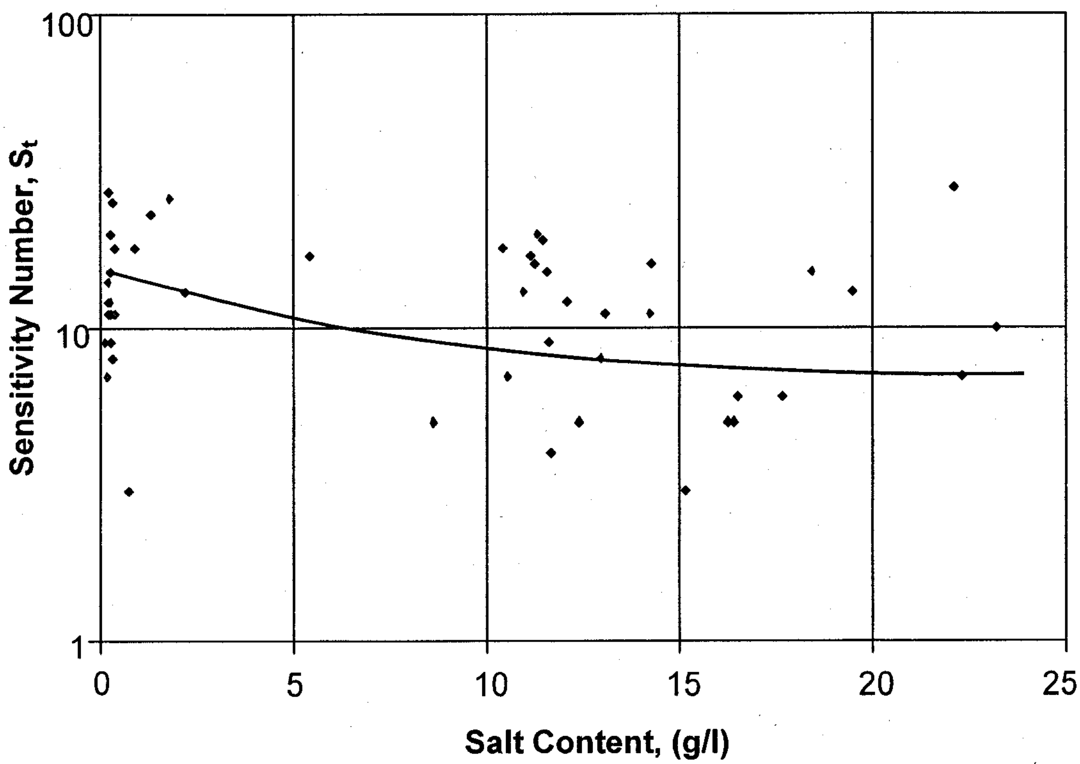

Figure 3.5: Sensitivity Number, S_t vs. Salt Content
(Data source: Chagnon et al., 1979)

3.3.2 Effect of Physical Factor

Sensitive clay, especially quick clay, usually has higher water content and accordingly, has high liquidity index, I_L. It is common for sensitive clay to have I_L more than 1.0 and sometimes it may be as high as 15 or more. Figure 3.6 presents the experimental data of Chagnon et al. (1979) in the form of the liquidity index, I_L versus the undisturbed shear strength, C_u. It can be noted from this Figure that the best-fitting line gives a concave upward curve in a semi-logarithmic plot. The shape of this curve is contradicting the test data produced by Wood (1990), as shown in Figure 3.7. The only conclusion can be drawn at this stage, is that there are other parameters such as the salt content, which were not duplicated between the investigations of Chagnon et al. (1979) and Wood (1990).

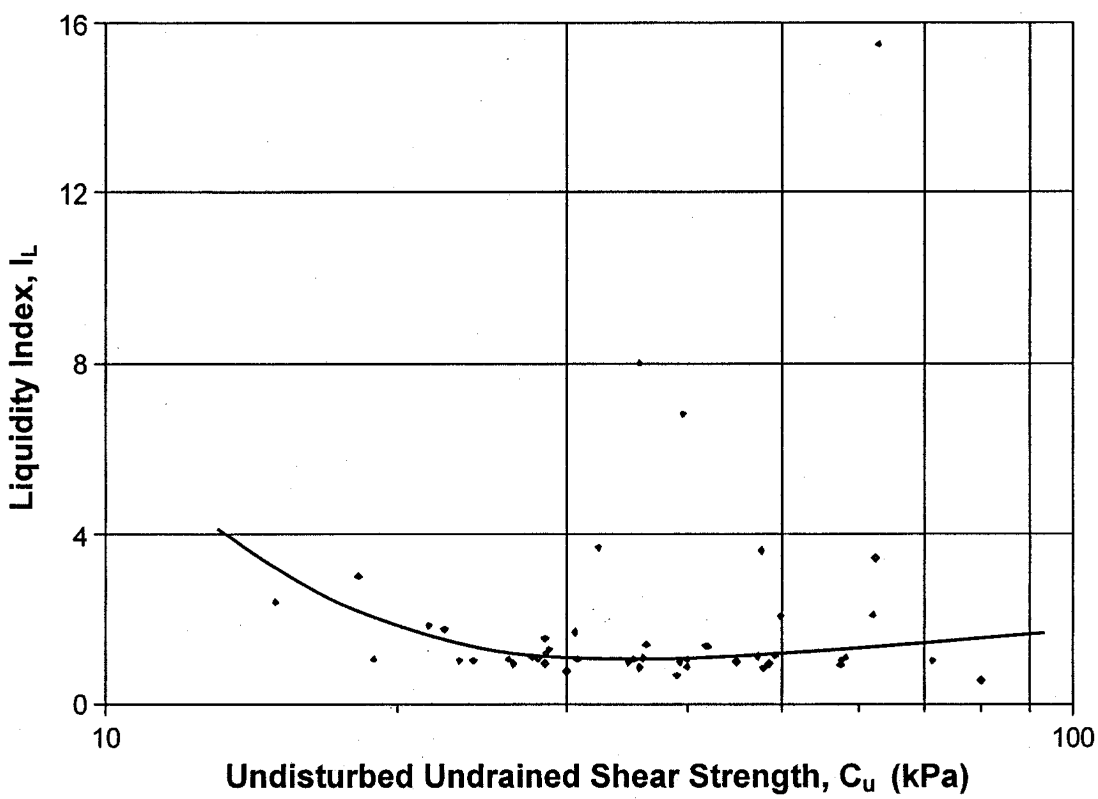

Figure 3.6: Liquidity Index, I_L vs. Undisturbed Undrained Shear Strength, C_u (kPa) (Data source: Chagnon et al., 1979)

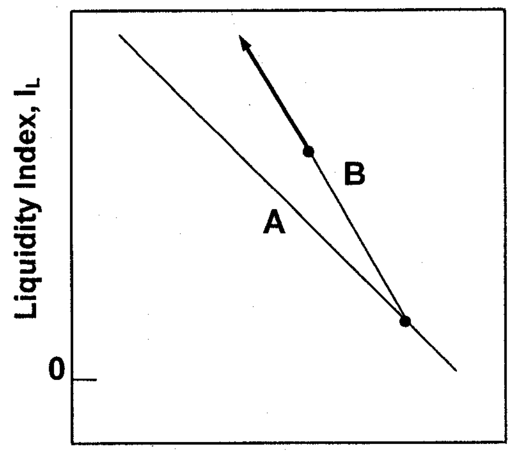

Figure 3.7: Approximate relation between I_L and C_u for Remoulded Soil (Line A) and Undisturbed Sensitive Soil (Line B) (after Wood, 1990)

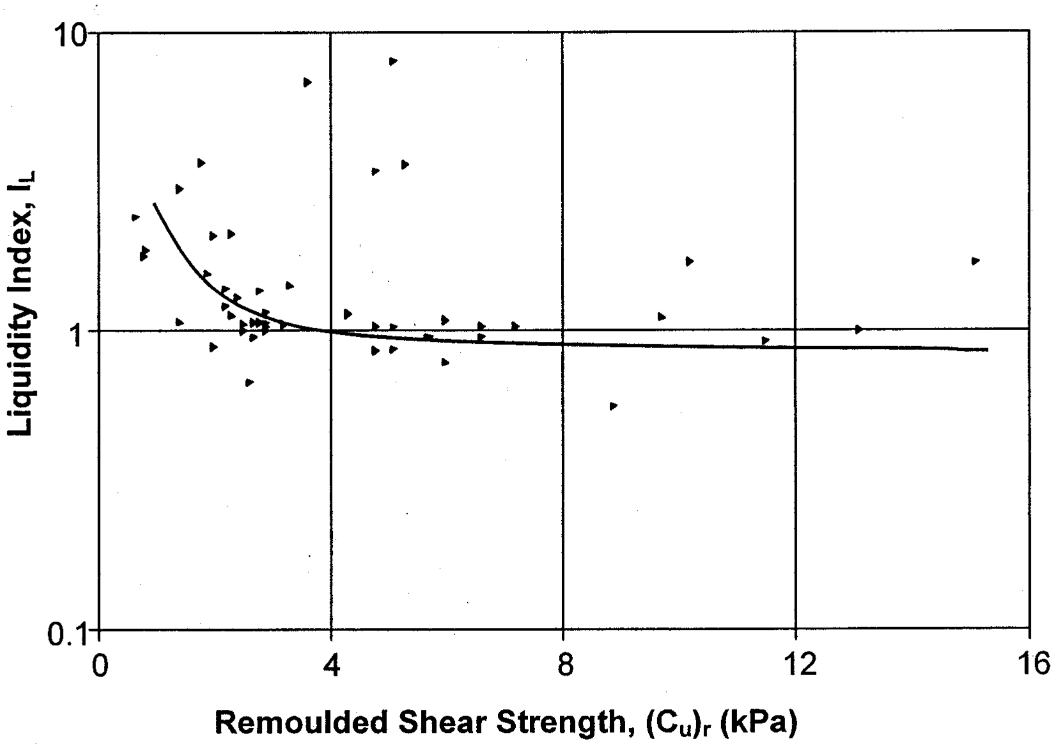

Figure 3.8: Liquidity Index, I_L vs. Remoulded Shear Strength, $(C_u)_r$ (kPa) (Data source: Chagnon et al., 1979)

Figure 3.8 presents the liquidity index, I_L versus the remoulded shear strength, $(C_u)_r$ in a semi-logarithmic space. It can be noted from Figure 3.6 and 3.8 that both the undisturbed and the remoulded shear strengths behave quite similarly due to the change of the liquidity index, I_L. Furthermore, beyond a certain value of I_L, the shear strength changes significantly. Specifically, at this stage, the soil will be in the liquid state.

3.4 Relation Between Physical and Mechanical Parameters

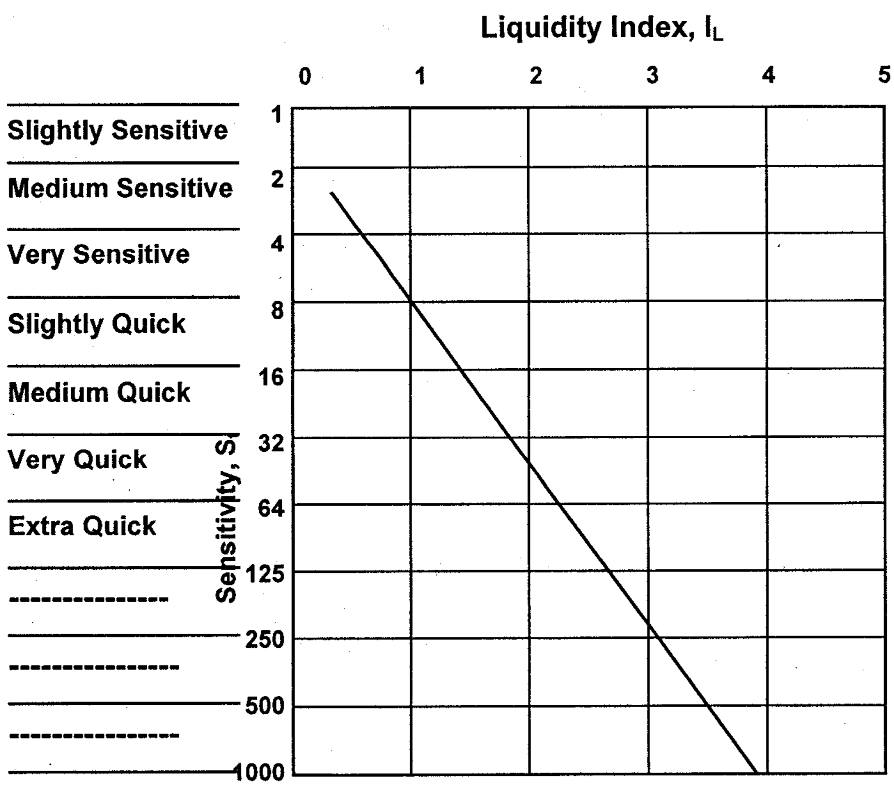

Figure 3.9: Relation between Sensitivity and Liquidity Index for Norwegian Marine Clays (after Bjerrum, 1954)

The variations of the sensitivity number, S_t with the liquidity index I_L, have been studied by a number of researchers in the literature. Bjerrum (1954) established this relation, as shown in Figure 3.9, as a straight line in a semi-logarithmic plot for the Norwegian marine clays. Wood (1990) presented similar results, as shown in Figure 3.10, utilizing the available data in the literature. He further introduced an empirical formula (Equation 2.1), together with a constant of variation, k describing the relation between S_t and I_L. The values of k are usually range between 1 to 3 for sensitive clay. While dealing with sensitive clay, the value of k is not easy to be determined directly from any simple test.

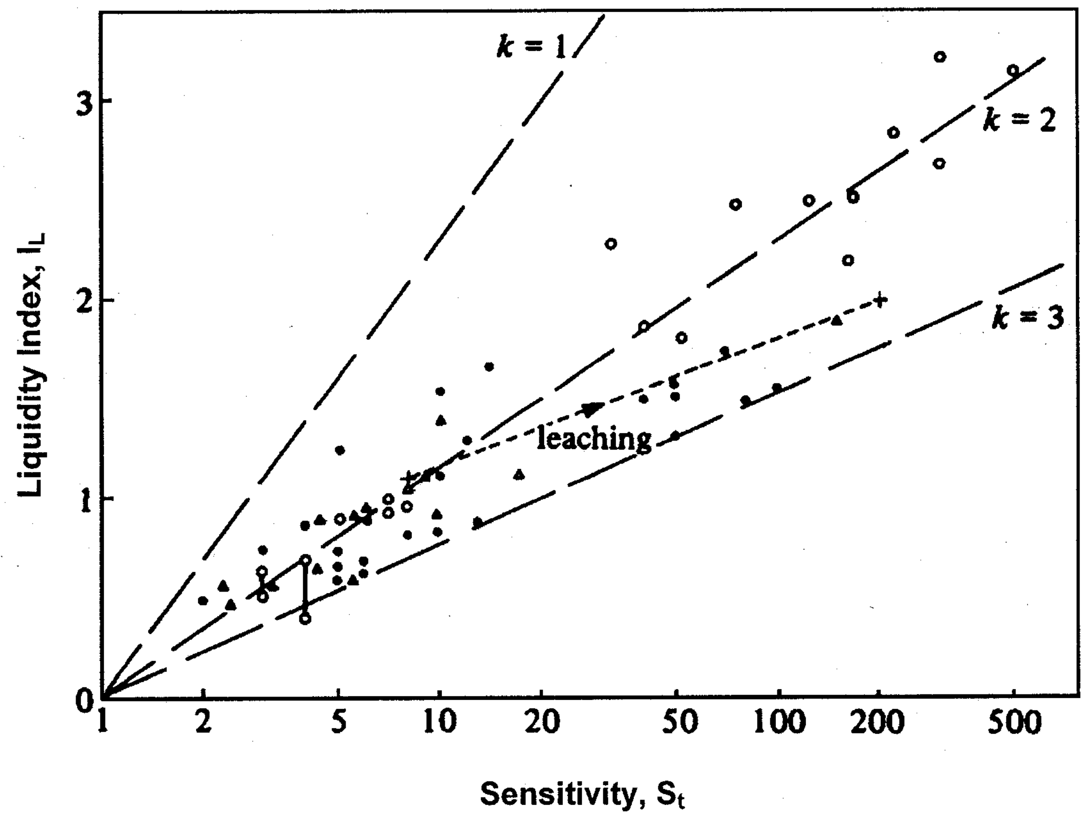

Figure 3.10: Interrelationship between Sensitivity and Liquidity Index for Natural Clays (Wood, 1990)

In other words, the leaching action may change this relation, as shown in Figure 3.10. For this reason, the so-called constant k should not be treated as a constant parameter even for a specific sensitive clay zone. According to this Figure, it can be noted that the process of leaching can significantly affect the value of the constant k. Moreover, the statement of having the constant k as a function of I_L is incomplete, as this relation is also dependent on the preconsolidation pressure, σ_p which is demonstrated in Figure 3.11.

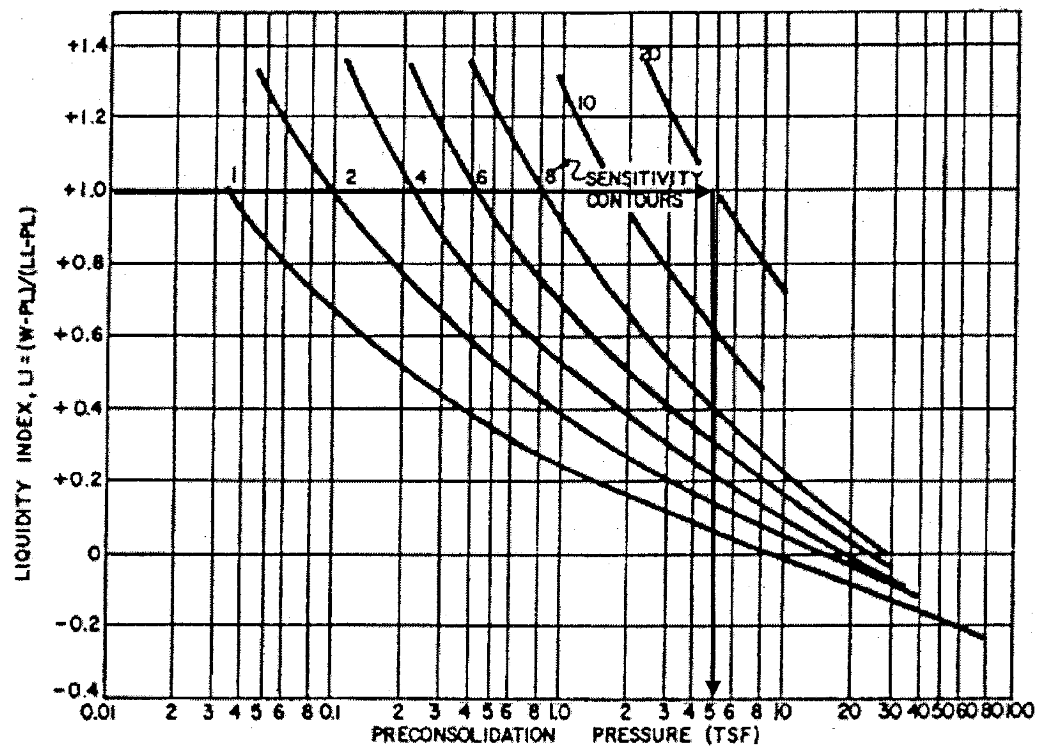

Figure 3.11: Preconsolidation Stress as a Function of Liquidity Index and Sensitivity Number (NAVFAC DM 7.1)

Figure 3.11 presents the liquidity index, I_L versus the preconsolidation pressure, σ_p for different sensitivity numbers, S_t. This relation was developed for soils with sensitivity up to 20. Moreover, this relation was observed for maximum I_L of 1.4. For this reason, highly sensitive clays with the higher values of liquidity index are beyond the scope of this relation. At the lower values of I_L (near about zero), the relation between the liquidity index, I_L and the preconsolidation pressure, σ_p becomes almost independent of the sensitivity number, S_t (up to S_t = 20). Figure 3.11 demonstrates that the liquidity index, I_L and the preconsolidation pressure, σ_p relationship is dependent on the level of sensitivity where the liquidity index, I_L is positive. Sensitive clays usually have water content almost equal or more than the liquid limit and positive values of I_L. Therefore, for sensitive clay it can be stated that preconsolidation pressure, σ_p is a linking parameter between the sensitivity, S_t and the liquidity index, I_L.

In order to check the rationality of the relationship given in Figure 3.11, the data of Mitchell et al. (1977) are used. For this set of experimental data, the values of the liquidity index, I_L, the sensitivity number, S_t, and the preconsolidation pressure, σ_p are 1, 15-25 and 170-180 kPa, respectively. With this I_L and S_t values, Figure 3.11 gives a value of 480 kPa (5 tsf) for the preconsolidation pressure, σ_p showing a deviation of 64% from experimental value of σ_p. It can be concluded that some governing parameters might have been ignored in developing this relationship, while the parameters or conditions had not been duplicated in the other set of tests. Specifically, for sensitive clays, increasing sample disturbances will lower the value of the preconsolidation pressure, σ_p. Inappropriate application of the load increment ratio or the load increment duration during consolidometer test will have the same effect. Accordingly, these relationships cannot be considered as design tools.

3.5 Analysis of Cyclic Test Results

Utilizing the test data on sensitive clay subjected to cyclic loading, available at Concordia University and in the literature, a parametric study on the factors, which are believed to govern this behaviour, is performed in the following section.

3.5.1 Effect of Confining pressure

The data of Lee (1979) shown in Figure 3.12, present the effect of the confining pressure on the cyclic triaxial test results. It can be noted from this Figure that the confining stress, σ_{3c} has a significant effect on the cyclic deviator stress, q_{cyc}. The relative value of the confining pressure, σ_{3c} with respect to the preconsolidation pressure, σ_p dominates the cyclic behaviour of sensitive clay, i.e., the soil can withstand higher deviator stress for a certain number of load cycles when the confining stress is increased.

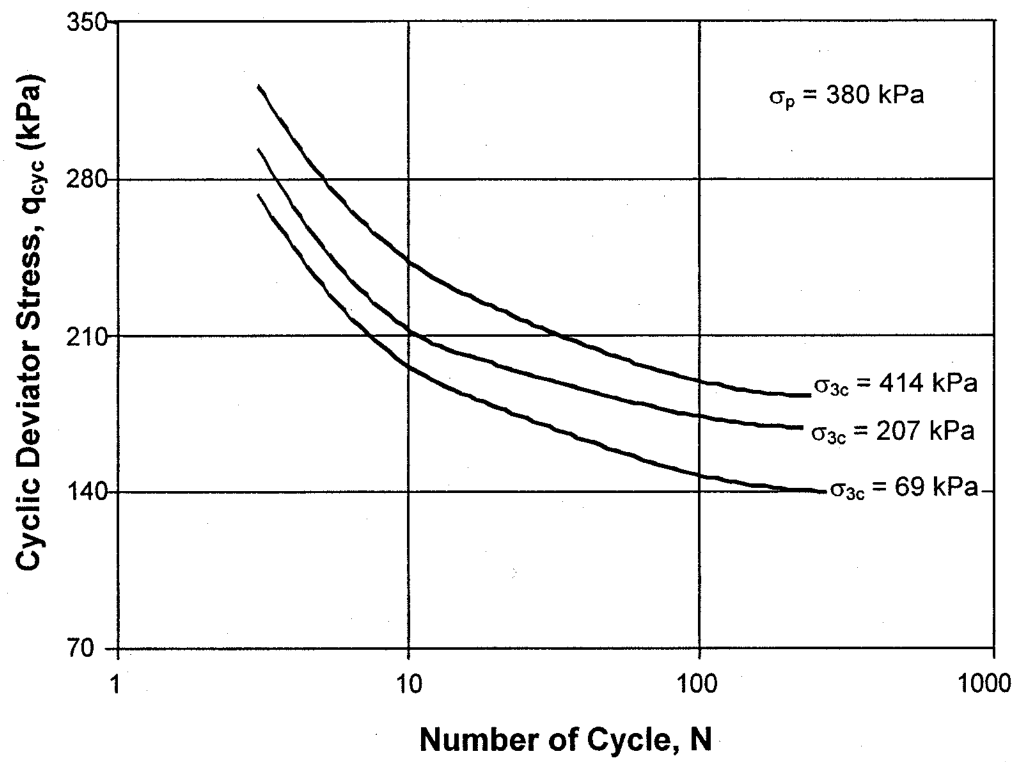

Figure 3.12: Effect of Confining Stress on the Cyclic Shear Strength of the Soil (after Lee, 1979)

It should be mentioned that in Figure 3.12, the clay was normally consolidated for the case of σ_{3c} = 414 kPa and overconsolidated for the other two cases. It is of interest to note that the results presented in Figure 3.12 portray the opposite to what has been reported in the literature for normal clay. Overconsolidated sensitive clay can be observed less strong than normally consolidated sensitive clay. This concept was also the prediction of the model developed by Eekelen et al. (1978). However, quite the opposite behaviour is observed in normal kinds of clay. Once again, it can be concluded that there must be other governing parameters that control such relation. In order to avoid this situation, during laboratory testing, designer should duplicate the field confining stress and the field preconsolidation pressure in cyclic shear strength tests.

3.5.2 Effect of Number of Load Cycles

Table 3.1: Undrained Cyclic Triaxial Test Results for Champlain Sea Clay (Hanna Unpublished Records)

Test	Cyclic Deviator Stress, q_{cyc} (kPa)	Static Strength, C_u (kPa)	Sensitivity S_t	Number of Cycle, N	q_{cyc} / $2C_u$	Drainage Option	Remarks
CYC. UT-04	53.35	48.6	7	10	0.55	Undrained	Failed
CYC. UT-02	32.4	48.6	7	121	0.35	Undrained	Failed
Static. UT	97.2	48.6	7	1	1	Undrained	Failed
CYC. UT-03	29.2	48.6	7	101	0.30	Undrained	Not Failed

The mechanical behaviour of sensitive clay has been investigated here by analysing the cyclic tests conducted at Concordia University. These results provide great benefits as all the tests were conducted on the same Champlain Clay sample. Summary of these test results are given in Table 3.1.

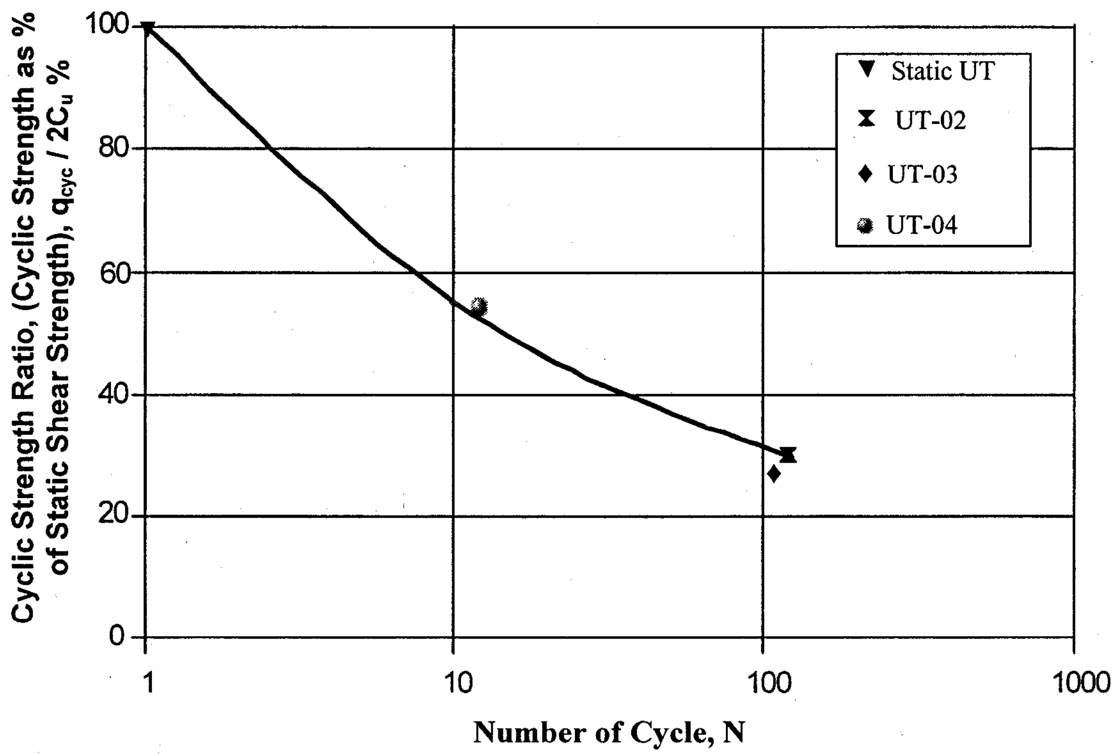

Figure 3.13: Relation between Cyclic Strength Ratio & Number of Cycles at Failure under Undrained Condition for Champlain Sea Clay (Compression only) (Data Source: Hanna Unpublished Records)

Among the three undrained cyclic triaxial tests, tests on two samples were continued until the failures were achieved. In this study, it is intended to present cyclic test data with respect to the number of load cycles, N. It can be noted that the cyclic deviator stress, q_{cyc} and the number of cycle, N at failure are inversely and non-linearly related. In this analysis, static strength is considered as cyclic strength, as if the sample failed at the application of the first load cycle under a cyclic deviator stress, which is equal to 100% of static deviator stress. In this way, the failure curve, as shown in Figure 3.13, for this sensitive clay under undrained condition is developed.

Figure 3.13 presents the cyclic shear strength ratio (cyclic strength as % of static shear strength), $q_{cyc}/2C_u$ versus the number of cycles, N. The curve shown in this Figure can be termed as failure curve. By definition, any points on the curve will represent the failure state. Experimentally, it is impossible to obtain points above this curve. In Table 3.1, for the test UT-03, the loading-unloading was stopped before the sample failed due to the application of subsequent load cycles. This point representing UT-03 test result is located below the failure curve. This case describes a hypothesis in the case of cyclic loading, i.e., any experimental data point located below the failure curve represents a safe condition.

Table 3.2: Undrained Cyclic Triaxial Test Data (Data Source: Lee, 1976)

Test No	Points in Figure	q_{cyc} (psi)	C_u (psi)	N	q_{cyc} / $2C_u$ (%)	Loading Condition	Sensitivity Number, S_t	Remarks
58	A	42	37	400	57	Symmetrical	35	Not Failed
75	B	30	17.5	537	86	Compression only	35	Not Failed

Figure 3.14 presents the experimental data of Lee (1979) as the cyclic shear strength ratio), $q_{cyc}/2C_u$ versus the number of cycles, N at failure. Furthermore, the results of the two tests on the samples, which did not reach failure, are summarized in Table 3.2.

For the test "A", the sample was loaded symmetrically. Due to the application of symmetrical loading, the samples were loaded in tension and compression alternately. For the test "B", the sample was subjected to compression loading.

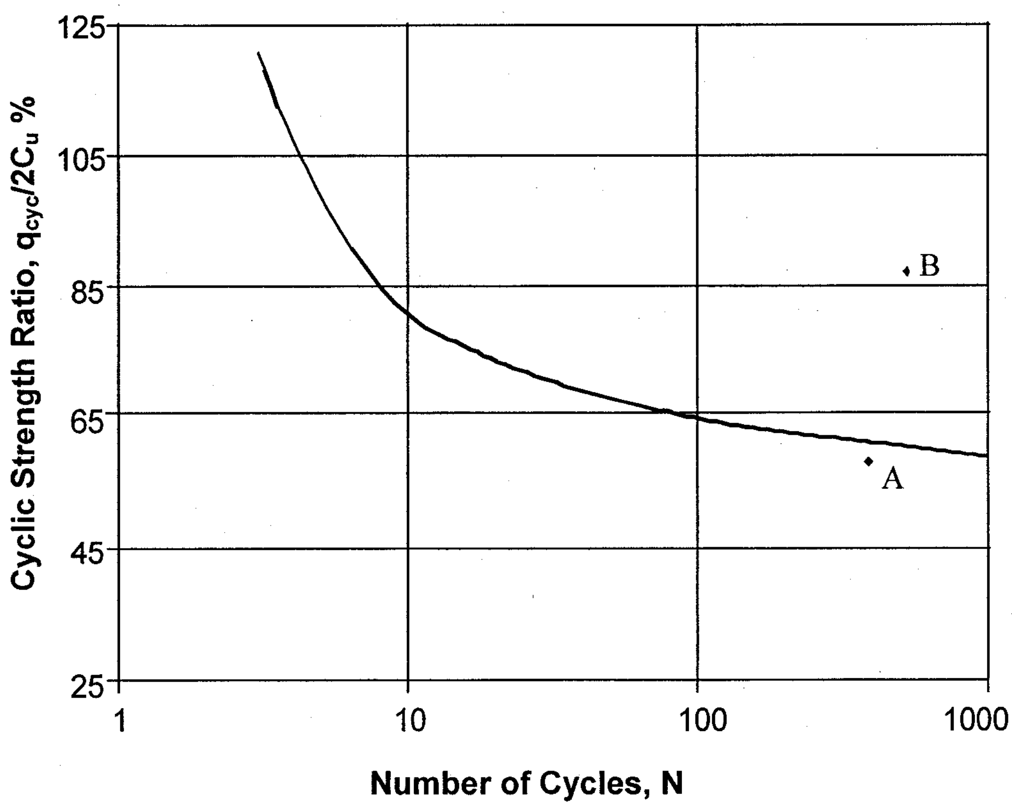

Figure 3.14: Cyclic Strength Ratio vs. Number of Cycles (after Lee, 1976 and 1979)

It can be noted that the point "A" falls below the failure curve while the point "B" is located above that curve. It should be mentioned here that all points on the failure curve in Figure 3.14 and the point "A" are presenting the case of symmetrical loading condition. Therefore, the point "A" validates the hypothesis. It can be concluded herein that the failure curve will identify the safe zone for foundation provided the testing conditions including the loading condition are the same for all tests. This phenomenon explains that the failure curve is unique for specific soil and loading condition.

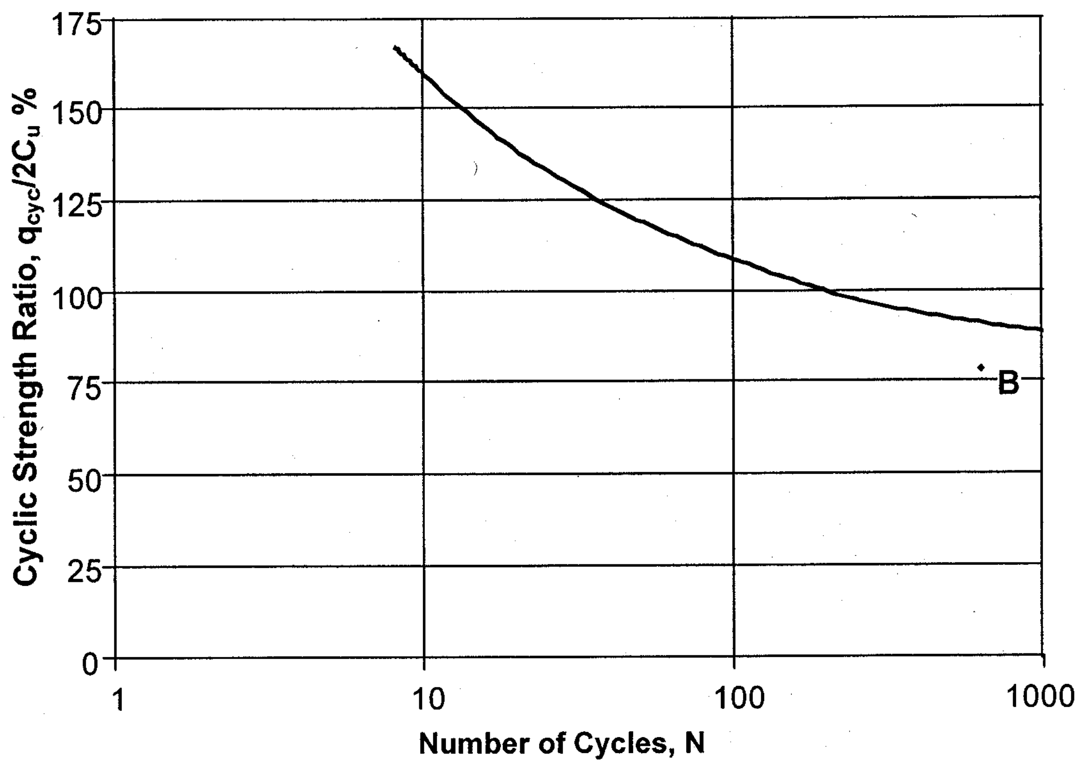

Figure 3.15: Cyclic Strength Ratio vs. Number of Cycles under Unsymmetrical Loading (Lee, 1976 and 1979)

Figure 3.15 presents the failure curve for Outardes clay tested under compression (only) loading condition, as reported by Lee (1979). All samples in this series, as represented by the failure curve and the point "B", were tested under an unsymmetrical loading condition. Moreover, the point "B" represents a non-failure state and is located below the failure curve. This fact again supports the hypothesis previously made in this section.

The three failure curves developed in Figure 3.13, 3.14 and 3.15 are for isotropic consolidation conditions prior to testing. According to Lee (1979), data for anisotropic consolidation condition fit well with the data for unsymmetrical loading condition.

Therefore, the proposed hypothesis is validated for sensitive clay collected from two different regions. Moreover, two types of loading conditions, such as symmetrical (wave shape) and unsymmetrical (compression; both wave shape and sudden increments) cyclic loading conditions, and both isotropic and anisotropic consolidation conditions support this hypothesis.

3.6 Statement of the Facts on Sensitive Clay

Based on the analysis presented in the previous sections and the numerous investigations conducted by different researchers, it can be noted that the salt content, the mineralogy and the chemical environment affect the behaviour of sensitive clay and the level of sensitivity greatly. In fact, the pore water chemistry of sensitive clay plays an important role, especially when considerable artesian pressure at soil-bedrock contact produces a gradient and causes the flow of water through the sediments. The flow of water through sensitive clay sediments may take place due to other reasons as well. As a result, the leaching action occurs through much of the profile and accordingly, changes the salt concentration, and therefore, the concentration of Mg, Ca, K and other ions. The geological factors control the potential leaching action. This process is considered as one of the main factors, causing the change in the level of sensitivity of the soil profile. Moreover, the extent of this change depends on the chemical composition and the original salt content of that clay. In some cases, the clay mineral content is also an important factor. Depending on the amount of mineral content present in sensitive clay layer, it governs the efficiency of the expected post-depositional leaching process.

Furthermore, it can be stated that the geo-chemical and the geological aspects of sensitive clay zone act simultaneously in determining the level of sensitivity.

The rheological response of sensitive clay is an important issue for landslide problems. Depending on the sensitiveness of the soil to the change in the salt content, the rheological characteristics of sensitive clay may change significantly from those of a Bingham type to those of a Casson type. In some cases, at the same liquidity index, these soils can have two values of yield stress. Moreover, these modifications of the rheological properties with increasing salinity would result in a change in the soil micro-fabric from a dispersed state to a flocculated state. In nature, leaching process follows a path in the opposite direction. In the literature, there are some relationships established between the liquidity index and the remoulded strength of sensitive clay utilizing its rheological characteristics. However, due to the effect of the leaching action on the remoulded strength with constant liquidity index, the available correlations become inapplicable.

The structural arrangements of sensitive clay particles may vary from one zone to another. Moreover, in the undisturbed and the remoulded state of the same sensitive clay, it has completely different structural arrangements. Their structures in both the states cannot be differentiated by any parameter. In each state, the amount of the total water content remains the same. Only the free water content increases due to the mechanical load applications. In fact, cyclically applied loading acts as a remoulding agent. Therefore, if the soil structure is flocculated in the undisturbed state, after remoulding the soil will get more densely packed state. Some of these phenomena can be explained by the electro-kinetic potential, the Van der Waal's attraction or other inter-particular forces.

As long as the leaching action is obvious for a specific case, the explanation of such a situation becomes very complex and requires the essential integration of chemical factors with the structural one.

The mechanical behaviour of sensitive clay under static and cyclic loading conditions has already been critically analyzed in this chapter. In addition, the mechanical response of this clay is governed by the pore water pressure, the duration to dissipate the excess pore water pressure, and the permeability of the soil layer.

The soil conditions and the properties of sensitive clay are varying considerably from zone to zone. This clay could be found as strong, and brittle; or as massive and blocky. In the undisturbed state, the clay layers may have a different consistency with or without visible bedding. There may also be some pockets or inter bedded layers of silt or sand. Scattered shells and pebbles are also frequently observed. Brittle clay failures in shearing on one or more well defined planes for one dimensional cyclic triaxial tests of sensitive clay have been observed by many researchers (Mitchell et al., 1977; Yong et al., 1976; Seed et al., 1964). Those clays were from different regions and their level of sensitivity varies within a wide range. All these physical characteristic properties cannot be presented only by the liquidity index.

One of the main reasons behind the complex behaviour of sensitive clay is that each factor's influences are compounded when they interact simultaneously. Secondly, these aspects are all inter-dependent. The full representation of its characteristic properties using parameters noted is incomplete and difficult to achieve. Finally, sensitive clay is primarily controlled by the chemical factor, which could modify the unusual characteristics of this clay from all respects. For these reasons, it is impossible to establish a link among all the influencing factors. While concerning about its mechanical behaviour in foundation design, it is difficult to incorporate the chemical factor in design unless extensive research program is carried out to evaluate its effect and introduce it as a governing parameter.

3.7 Cyclic Strength and the Interference of Different Factors

Cyclic shear strength data available in the literature are presented in this section in an attempt to develop a unique model for sensitive clay for a given region in the world. As demonstrated above, the cyclic shear strength depends on physical, mechanical and chemical parameters, which can only be dealt with by means of experimental investigations. Accordingly, in the following section, a design procedure will be presented.

Figure 3.16 presents the failure curves available in the literature. In this Figure, the cyclic shear strength is presented as a percentage of the static shear strength. This is based on the fact that the undisturbed static (undrained) shear strength has some significant impact on its degradation of shear strength under cyclic loading. It can be noted from this Figure that the higher cyclic strength ratio, $q_{cyc}/2C_u$ can be applied where the lower number of load cycles, N under undrained condition is expected to cause failure.

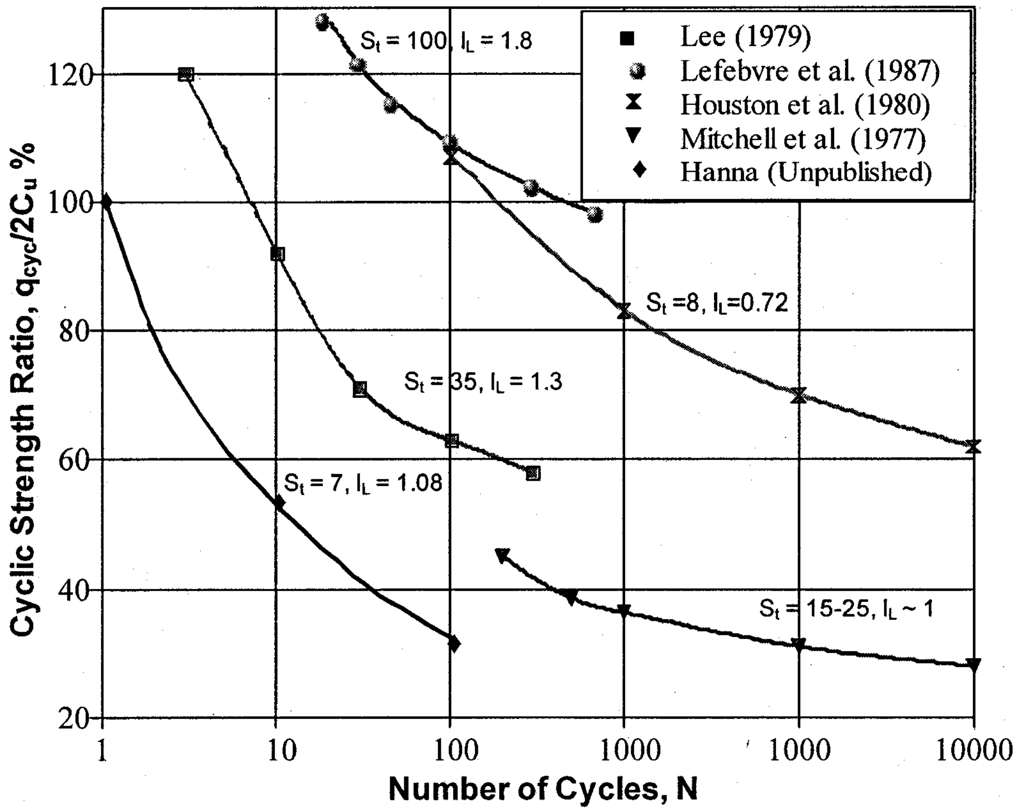

Figure 3.16: Cyclic Strength Ratio vs. Number of Cycles for Four Sensitive Clays

For a given region, these failure curves will follow a pattern, i.e., clay of low sensitivity will support higher cyclic deviator stress and vice versa. However, each of these curves was developed for a different zone. There are significant differences in geology, chemical environment, particle structure, simulation of cyclic loading during laboratory testing, depth of sampling, permeability among the many other conditions, which the curves are representing. For this reason, it is impossible to develop any pattern or mathematical model to represent these variations. The characteristic properties of these five sensitive clays are presented in Table 3.3.

Table 3.3: Properties of Five Sensitive Clays

Parameter Name	Mitchell et al. (1977)	Lee (1979)	Houston et al. (1980)	Lefebvre et al. (1987)	Hanna (Unpublished Records)
Depth in meter	5	-	2.4	6.8	4
Water Content (w) %	33 - 38	35 - 38	68 - 129	50	71
Liquid Limit (LL) %	34 - 41	32 - 36	94 - 100	38	69
Plastic Limit (PL) %	18 - 20	17 - 19	-	24	44
Liquidity Index (I_L)	~unity	1.3	-	1.8	1.08
Pre-consolidation Pressure (σ_p) in kpa	170 - 180	380	-	145	-
Sensitivity (S_t)	15 - 25	35	8	100	7

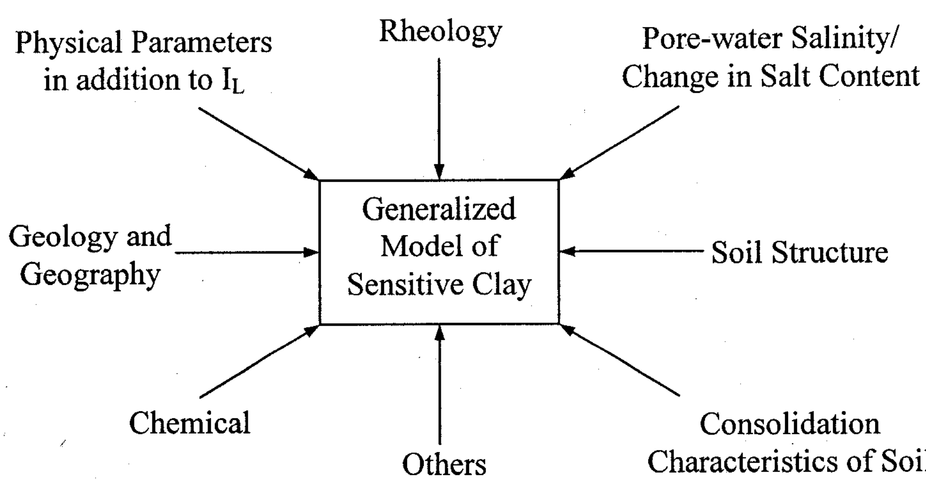

Figure 3.17: Influencing Factors for Generalized Model of Cyclic Strength

It can be stated that mathematical modelling of the cyclic behaviour of sensitive clay becomes complicated due to the interference and the interdependence of many factors. As mentioned earlier, some of these factors may change with time, which in turn may change the mechanical behaviour of this type of clay. For this reason, it is practically impossible to achieve a unique solution for cyclic strength and requires additional parameter(s) that could represent any sensitive clay zone. Figure 3.17 diagrammatically presents the influencing factors that have to be considered for the development of a generalized model of cyclic strength for sensitive clay.

3.8 Design of Foundations on Sensitive Clay subjected to Cyclic Loading

As mentioned earlier, sensitive clay has received the attention of many researchers attempting to develop a rational solution. As a result, equations (e.g., Equations 2.1, 2.2), expressing the relation among different parameters have been developed. These relations are found not to be useful for designing the foundations, especially while dealing with cyclic loading condition. Equations relate the sensitivity number with the liquidity index, the static undisturbed strength with the liquidity index, the remoulded strength with the liquidity index, the consolidation parameters with the plasticity index and so on. It can be noted that almost all the efforts have been given to express the clay's mechanical behaviour in terms of the index properties. Researchers have failed to model the phenomenon of sensitive clay mathematically. One of the main reasons behind this failure is that a generalized model cannot be developed on the basis of index properties only. Moreover, either the index properties or the sensitivity number, S_t impose limitations in using these relations frequently. Finally, up to the date, the behaviour of sensitive clay is observed unpredictable under cyclic loading phenomena, which is considered as the most severe loading condition.

The equations by Hyodo et al. (1994 and 1999) had been developed for marine clay. However, the equations do not include any parameter to control the effect of the sensitivity number on its strength properties. Even, there is no special note with those equations regarding the validity or the limitations of the equations from the sensitivity point of view.

The model by Eekelen et al. (1978) has major drawbacks in its practical use. Firstly, it had been developed for zone specific clay, i.e., Drammen clay. Secondly, the behaviour of clay under cyclic loading had been described in terms of a single fatigue parameter. This parameter is the generated pore water pressure due to cyclic loading. In this model, cyclic loading is not described in terms of fundamental material laws Finally, the sensitivity number, S_t had not been introduced in the equation for the strength reduction model, which is empirical in nature.

3.9 Proposed Design Procedure

With the background information presented herein, a design guideline can now be proposed. The findings of previous sections are employed as the concept behind this proposed guideline. It is already well established that the relation between the cyclic shear strength ratio, $q_{cyc}/2C_u$ versus the number of cycles, N is not only dependent on the sensitivity and the liquidity index, but also on other aspects which interfere with the mechanical behaviour of sensitive clay. This procedure developed is made to neutralize the interference of the unknown parameters including the chemical and the geological aspects. This procedure should be adopted until this phenomenon can be formulated mathematically.

The following design procedure will allow foundation designers to estimate the allowable cyclic deviator stress (q_{cyc}) for an expected number of cycles (N) during the undrained period. This proposed design procedure is explained step-by-step in the following and presented in Figure 3.19 as a flow diagram:

Step 1: Collect representative samples from the proposed site. Samples should be collected from immediately below the proposed foundation level and greater depths. During sampling, expert attention is needed to obtain undisturbed samples. Samples need to be brought into the laboratory with special care avoiding vibrations that may cause disturbances to the samples.

Cylindrical specimens should be prepared and kept in a temperature and moisture controlled room. From each block, at least 3 specimens (disturbance acceptable) for liquidity index tests, 3 undisturbed specimens for unconfined compression tests, and 3 high quality undisturbed specimens for consolidometer tests are to be prepared. In addition to these, 8 to 10 specimens will be used for both static and cyclic triaxial tests.

Step by step testing requires considerable time duration. For this reason, standard care is required for the storage of specimens (i.e., covering them with aluminium foil and wax). If there is any chance that the testing program will be prolonged for over a month, additional specimens will be required for checking the aging effects from the sensitivity and the index property point of view.

Step 2: Determine the soil condition defining parameters (I_L, S_t and σ_p). For these samples, assurance of data reproducibility is required with reasonable deviation. Liquidity index, I_L can be determined following the standard procedure. Sensitivity number, S_t can be obtained by performing unconfined compression tests while pre-consolidation pressure, σ_p is determined graphically from the consolidometer test results.

At the end of each sensitivity test, water content of the remoulded sample has to be checked for estimating the possible loss during remoulding. From the definition of sensitivity number, S_t water content should remain unchanged in both the undisturbed and the remoulded states.

For determination of preconsolidation pressure, σ_p the applied load increment ratio and the duration of load application have to be carefully controlled.

When the number of samples to be tested is large, pre-consolidation pressure can be predicted using the following equation given by Skempton (1957):

$$C_u / \sigma_p = 0.11 + 0.37\, I_p \quad \text{(3.1)}$$

Where,

C_u = Unconfined Undrained shear strength,

σ_p = Pre-consolidation pressure,

and I_p = Plasticity Index.

Step 3: Determine the undrained undisturbed static shear strength, C_u of the clay using triaxial testing apparatus. Field preconsolidation and field confining pressures have to be implemented during testing. The mean value of the static triaxial tests will be used in assigning the values for the cyclic deviator stress, q_{cyc} during cyclic triaxial testing.

Step 4: Estimate the undrained period for the given soil condition and estimate the number of load cycles, N that may occur during the undrained period. This can be achieved by surveying the proposed site, by looking up the city records for winds and earthquake occurrence history, and by following the design requirements.

From the city geological survey report, soil stratification can be identified. The report will provide the presence and the depth of the sensitive clay layer, and the information about each sub-layer. Drainage condition (both-way/one-way) for the sensitive clay layer under consideration will be known from this source, which will constitute the basis for determining the undrained period.

Step 5: As I_L, S_t and σ_p have been experimentally determined in step 2 and for that sample location the respective arithmetic averages of these parameters have to be assigned against each block sample. If more than one-block samples are utilized in this testing program, the average values have to be compared.

Conduct a series (at least 5) of cyclic triaxial tests on samples with the same I_L, S_t and σ_p simulating the type of loading expected at field . In each test, assign a value of the cyclic deviator stress (roughly between 0.4 to 0.7 of the static deviator stress), then measure the number of cycles needed to reach failure.

This range for the cyclic deviator stress ratio is recommended so that some points could be obtained on the left and some on the right side of the approximate 'critical level of repeated loading'. Adopting such values of the cyclic deviator stress will also provide the opportunity to observe that critical level for sensitive clay under consideration.

Continue the cyclic loading test until a shear plane is developed or stop the testing when 3% axial strain is attained.

If it is desired to reach the equilibrium condition in field, test can be performed with very low cyclic stress ratio, for example, 0.2. During testing, duplication of the field confining stress and field preconsolidation pressure is recommended.

Step 6: Plot the cyclic test results in the form of Figure 3.18. The results will produce points on the failure curve. Area below the failure curve defines the safe zone for foundations. Any combination of the cyclic strength ratio, $q_{cyc}/2C_u$ (%) and the number of cycles, N within this zone will lead to a stable ground and accordingly, safe foundations.

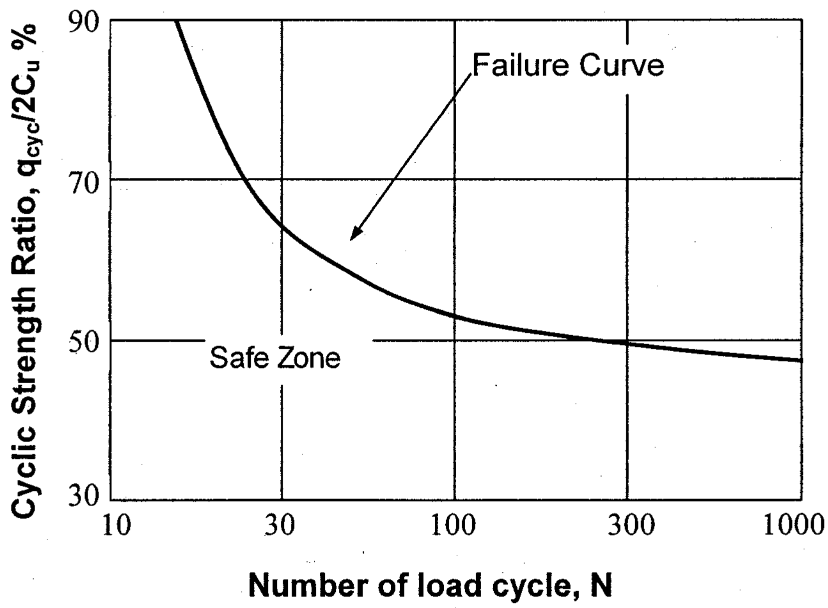

Figure 3.18: Typical Safe Zone for Foundation on Sensitive Clay

Step 7: For the given field and loading conditions and knowing the number of cycles, N that may take place during the estimated undrained period, determine the allowable cyclic strength ratio, $q_{cyc}/2C_u$ using the failure curve developed in step 6. A resonable factor of safety should be implemented depending on the cost and the function of the project under consideration.

In the case of unfavorable conditions, efforts should be made to reduce the undrained period and/or the number of cycles during the undrained period, and/or the cyclic deviator stress and/or perform one of the soil improvement techniques to increase the shear strength of the sensitive clay.

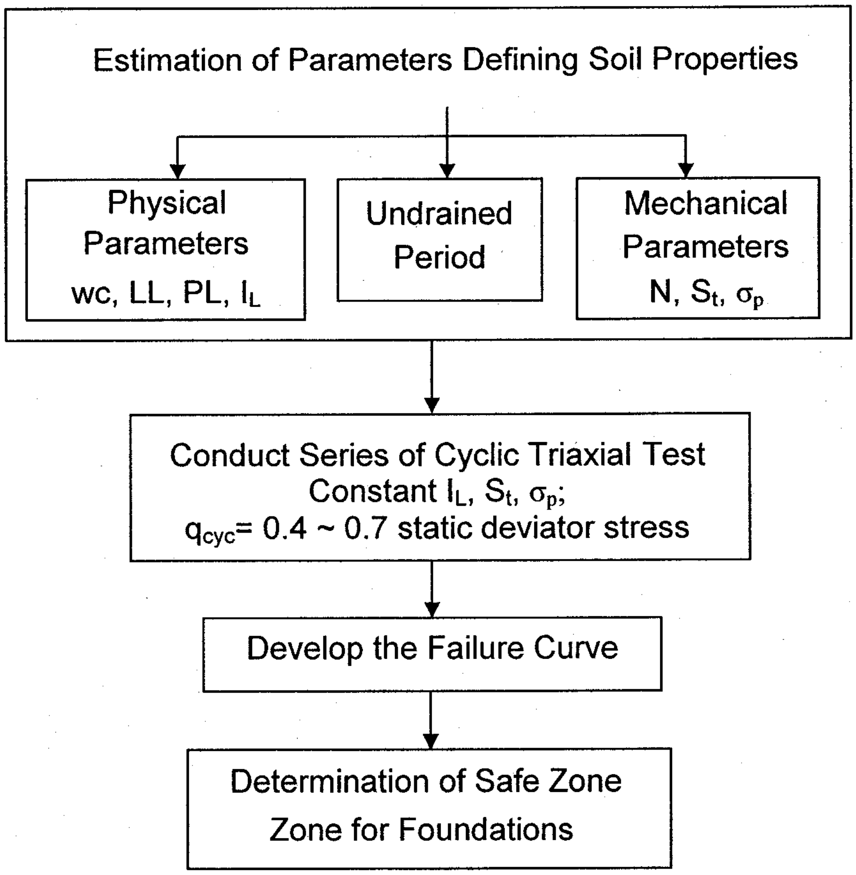

Figure 3.19: Flow Diagram Showing the Design Steps

The failure curve, as shown in Figure 3.18, giving the design values can be termed as the design curve. The design curve can provide the following types of design values:

1. Where the expected number of cycles is not high, any combination leading to a point in the safe zone will give the cyclic strength as a percentage of static strength.
2. Where the expected number of cycles is high, the above approach will also be appropriate or one may choose the condition of attaining the state of equilibrium. The equilibrium state begins where the design curve becomes parallel to the axis for the number of cycles.

The interpretation of the result from the design curve will be as follows:

$q_{max} = q_s + q_{cyc}$

$= q_s + (q_{cyc} / 2C_u) * q_s$

$= q_s * [1 + \text{cyclic strength ratio}]$ .... (3.2)

$q_{allow} = q_{max} / F.S.$ .... (3.3)

Where, q_s = Static Deviator Stress,

q_{cyc} = Cyclic Deviator Stress,

C_u = Undisturbed Undrained Static Strength,

q_{allow} = Allowable Deviator Stress,

$q_{cyc}/2C_u$ = Cyclic Strength Ratio,

q_{max} = Maximum Deviator Stress,

F.S. = Factor of Safety.

Finally, the foundation designer will get the soil carrying capacity as a set of data in the form of (q_{allow}, q_{cyc}, N). In the note, the loading condition must be reported.

3.10 Discussion

Based on these analyses, sensitive clay is considered as a very complex and difficult type of soil. Interference of different factors and their simultaneous interactions make this soil even complicated, especially for foundations. Moreover, many parameters are interdependent; their effects change with respect to each other. Furthermore, sensitivity number is a time dependent index and also varies with regional stratification. Sensitivity property of the clay is developed throughout its geological age. Change in the geo-environment, the chemical environment and other external factors are found responsible to make the soil sensitive. All these factors make it impossible to develop a general unique solution for designing foundation on sensitive clay under cyclic loading condition. A step-by-step design procedure was presented in this chapter. It is flexible and can be used for any sensitive clay knowing the field and the loading conditions for the proposed foundations. Moreover, as sensitivity may change with time, the field and soil conditions have to be examined periodically to establish the existing factor of safety. It can also be used to predict the loss of strength associated with pile driving.

CHAPTER 4

Conclusions

4.1 General

Based on the analysis of the experimental results and the review of the literature, it can be concluded that sensitive clay behaves like a complex and difficult type of clay, especially when structures founded on it are subjected to cyclic loading. In the literature, no unique design formula incorporating all the influencing parameters can be found. Until the chemical and the geological aspects of this clay are fully defined and modelled mathematically, the proposed laboratory design procedure can be followed successfully for the design and the monitoring of the foundations on sensitive clay. The following can be concluded:

1. Low undisturbed (undrained) and low remoulded static shear strengths are usually associated with the higher values of liquidity index, I_L. However, sensitive clay, with low I_L indicates that the factors other than the liquidity index have greater influence on the static shear strengths.
2. Static strengths become very unpredictable during the last part of leaching process. This is due to the fact that the pore water chemistry affects the behaviour of sensitive clay during this stage.

3. In a sensitive clay sample, leaching does not introduce any significant change in the level of sensitivity at the beginning. However, the level of sensitivity becomes unpredictable during the last part of the leaching process. Sensitivity number and liquidity index do not have any unique relation, as the process of leaching cause alteration of this relation. Moreover, preconsolidation pressure acts as a controlling factor between these two parameters.
4. The total salt content has an influence on the strength of sensitive clay. However, the composition of remaining salt content during the last part of leaching process is noted to control the mechanical response of sensitive clay.
5. The concept, that highly sensitive clay will support relatively lower cyclic stresses, is not true for all cases. Cyclic strength of sensitive clay depends not only on sensitivity number, but also on the pore water chemistry, the geo-chemical environment and the geology of the specific zone from where the clay has been extracted. These parameters influence significantly the mechanical behaviour of the sensitive clay.
6. A design procedure is presented to take into account all the parameters affecting the mechanical behaviour of sensitive clay. It is flexible enough to use it for any sensitive clay zone, and be accepted as a practical tool for the following cases:
    a. Determining the allowable and the safe load of foundations on sensitive clay subjected to cyclic loading condition during the estimated undrained period.
    b. Estimating the loss of strength of the surrounding sensitive clay as a result of pile driving operation.

c. Examining the condition and evaluating the prevailing factor of safety of an existing foundation on sensitive clay during the lifetime of the structure, especially during the undrained period.
d. Allowing the foundation engineer to adjust loading or cycling conditions as a remedy for foundations built on sensitive clay, showing some signs of fatigues.

4.2 Recommendations for Future Research

A full-scale laboratory investigation is required to control all the influencing parameters describing all aspects of sensitive clay to support the design procedure given above.

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