The major project on

Consolidation Behaviour of Mix Clay Using Laurent Transform

Submitted in Partial Fulfilment for the Award of the Degree of

MASTER OF TECHNOLOGY

IN

CIVIL ENGINEERING

With Specialisation in

GEOTECHNICAL ENGINEERING

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2011-2013

DELHI TECHNOLOGICAL UNIVERSITY

CERTIFICATE

This is to certify that the project report entitled ***"Consolidation Behaviour of Mix Clay using laurent transform"*** is a bonafide record of work carried out by **Bijendra Singh Mehra (2K11/GTE/04)** under my guidance and supervision, during the session 2013 in partial fulfillment of the requirement for the degree of Master of Technology (Geotechnical Engineering) from Delhi Technological University, Delhi. This is to certify that the above statement made by candidate is correct to the best of my knowledge.

Prof. A. Trivedi

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STUDENT'S DECLARATION

I, Bijendra Singh Mehra, hereby certify that the work which is being presented in the Major project entitled ***"Consolidation Behaviour of Mix Clay using laurent transform"*** is submitted, in the partial fulfilment of the requirement for the award of the degree of "MASTERS OF TECHNOLOGY" with specialization in "GEOTECHNICAL ENGINEERING" at Delhi Technological University is an authentic record of my own work carried under the supervision of **Prof. A. Trivedi**. I have not submitted the matter embodied in this major project for the award of any other degree or diploma also it has not been directly copied from any source without giving its proper reference.

Bijendra Singh Mehra
(2K11/GTE/04)

ACKNOWLEDGEMENT

It is distinct pleasure to express my deep sense of gratitude and indebtedness to my supervisor and H.O.D **Prof. A. Trivedi**, Department of Civil Engineering**,** Delhi Technological University (Formerly Delhi College of Engineering),for his invaluable guidance, encouragement and patient reviews. His continuous inspiration made me able to complete this major project. Without his help and guidance, this major project would have been impossible. He remained a pillar of help throughout the project.

I express my sincere gratitude to the faculty and non-teaching staff of Civil and Environmental engineering department and the library of Delhi Technological University, (Formerly Delhi College of Engineering) for providing the relevant information and help when needed during the course of my project work.

I am deeply thankful towards all the lab assistants of **'Swati constructions'** who have helped me to conduct the experiments. Also I am deeply thankful towards the lab assistants of my college who helped me a lot for conducting experiments here.

Bijendra Singh Mehra
(2K11/GTE/04)

ABSTRACT

Consolidation behaviour of fine grained soil is evaluated commonly by Terzaghi's one dimensional linear elastic consolidation theory. It is based on several simplified assumptions. Out of these assumptions most important assumption is constant value of coefficient of consolidation C_v during the consolidation process. But it leads serious problems to the consolidation of soft clay like bentonite and bentonite –sand mixture. Since Terzaghi's one dimensional consolidation theory gives good results for small strain problems.

Hence in this study Terzaghi's one dimensional consolidation theory is used for evaluating consolidation of pure bentonite and bentonite –sand mixture assuming constant value of coefficient of consolidation C_v for small range of effective stress. Variation of coefficient of consolidation with effective stress for different percentage of bentonite in sand is shown.

Moreover variation of other consolidation properties like coefficient of compressibility a_v, coefficient of volume compressibility m_v, compression index C_c with effective stress is determined and comparison of these properties for different clay content are made.

Based on consolidation property permeability of clay is calculated indirectly for different percentage of clay. It is observed that permeability of clay increases with decrease in clay content.Based on consolidation properties graphs between average degree of consolidation and time factor is plotted and comparison is made for different percentage of clay.

Rate of consolidation is compared here using the two transforms Fourier as well as Laurent and it is clear from the experiments that rate of consolidation is faster using Laurent transform.

LIST OF FIGURES

LIST OF TABLES

INDEX

DeDication

I dedicate this thesis to

my family, my teachers and my friends for

supporting me all the way and doing all the wonderful

things for me.

CHAPTER- 1

INTRODUCTION

1.1 CONSOLIDATION

According to Terzaghi (1943), “a decrease of water content of a saturated soil without replacement of the water by air is called a process of consolidation” Due to the application of an external load, there is an initial increase in pore-water pressure throughout the sample known as the initial excess pore water pressure.

According to Darcy’s law, the excess pore water pressures are the driving force of seepage flow. Flow takes place due to the hydraulic gradient generated by the initial excess pore pressure distribution. At any stage of the consolidation process, the pore-water pressures will vary within the soil layer. The distribution of excess pore-water pressure at any given time after loading can be represented by Isochrones.

Thus, the process of consolidation can be expressed as a series of isochrones that graphically represent the relationship between time and the degree of consolidation over the depth of the soil stratum. Consolidation can be further characterized by the average degree of consolidation, which represents the consolidation of the stratum as a whole and eliminates the variable of depth.

Consolidation is a time dependent phenomenon. Due to application of compressive load on saturated clay pore water pressure increases immediately. Hence pore water tries to sheep towards low pressure area. Since the permeability of clay is very low, pore water takes some time to dissipate.

1.2 BEHAVIOUR OF MIX CLAY

Bentonite is a compressible soil. It shows high swelling and shrinkage due to presence of montmorillonite mineral. Montmorillonite belongs to smectite group. They show structural gap in their mineral structure. Water molecule enters to the gap and stake one above othercausing expansion of the whole mineral structure. Due to application of loading on saturated bentonite and bentonite- sand mix pore water pressure dissipates which causes increase in effective stress. Due to the increases in effective stress void ratio decreases. Hence hydraulic conductivity and coefficient of compressibility which are functions of void ratio changes with void ratio . These changes are very small for small strain problem. But in case of compressible soil and soft soil changes are appreciable. Compressibility can be reduced by mixing sand in compressible soil. But permeability increases with addition of sand in bentonite [**Shirazi (2010**)].

These two properties compressibility and hydraulic conductivity are represented by coefficient of consolidation. Coefficient of consolidation isdirectly proportional to hydraulic conductivity and indirectly proportional to compressibility. Hence variation in coefficient of consolidation will depend on change of hydraulic conductivity and change of compressibility with the addition of sand in bentonite [**Murthy (1987**)].

Permeability and swelling characteristics of bentonite and bentonite - sand mixture are two very important parameter for designing any type of waste disposal applications. Bentonite and bentonite –sand mixture is mainly used as a buffer material for disposal of radioactive waste. Concept of underground Disposal of hazardous material comes from nuclear industry [**Komine (1991**)].

Since environmental requirement for all types of underground disposal will converse towards the approaches developed for long lived radioactive disposal. Deep geological repository is based on multilayer barrier system concept. It ensures long term confinement and isolation of waste from environment [**Shirazi (2010**)].

1.3 FUNCTIONS OF BETONITE-SAND MIX

Basic function of bentonite-sand mix to fulfil as a barrier material in deep geological repository are given below:

1.3.1 Mechanical Functions:

- Buffer material should hold the container in place and prevent collapse of the excavation.
- Plastic deformability of buffer material ensures redistribution of stresses.
- Heat conductivity for transfer of heat from the waste package to the surrounding host rock.
- Hydraulic conductivity should be low to limit the flow of ground water to the waste material.
- Buffer material should create impermeable zone around the container to ensure that radionucleoid released from the waste limited cannot penetrate the impervious zone.
- High swelling properties to guarantee the sealing of any cracks occurring in the fillmaterial hence ensuring impervious barrier.

1.3.2 Physiochemical Functions:

- Buffer material must absorb escaping radionuclides so that their migration to the geoenvironment is retarded.

Buffermaterial must retain their mechanical and physiochemical function over a long span time and at high temperature which may exist at that place. Bentonite–Sand mix fulfils all the above requirements of a buffermaterial hence it used as a buffermaterial in nuclear disposal.Here its permeability characteristics are calculated indirectly using consolidation parameters [**Shirazi (2010**)].

1.4 MAJOR RESEARCHES IN THIS FIELD

Table No 1.1 Major Scientists &their Contribution

Scientists	Contribution
Terzaghi's Analysis, 1925	1-D Consolidation
Biot Theory, 1940	3-D Consolidation with restructuring of soil solids
Schiffman, 1958	1-D consolidation theory under time dependant loading where permeability and coefficient of consolidation vary with time.
Geng et al., 1962	Use of Laplace transforms to obtain solution for non linear 1-D consolidation.
Davis & Raymond, 1965	Relation of permeability of soil with coefficient of consolidation.
Gibson & Hussey,1967	Analysis of 'rate of consolidation' depending upon thickness of soil strata.
Booker and Small, 1976	Finite layer analysis technique for 2-D and 3-D consolidation of multilayered soils.
Oslon.et al., 1977	Incorporate Ramp loading as mode of load application in 1-D consolidation.
Sridharan et al., 1995	Analysis of Time factor for secondary consolidation of extremely clayey soil.
Wang and Fang, 2001	Analysis of Biot consolidation problem for multilayered porous media by a state space method in cylindrical coordinate system.
Conte et al., 2006	Analysis of coupled consolidation of unsaturated soil under plain strain loading.
Rani et al., 2011	Consolidation of mechanically isotropic but hydraulically anisotropic clay layer incorporating compression of pore fluid and solid constituents.
Vinod et al., 2010	Cross section range of fast loading for radial consolidation.
Tewatia et al., 2012	Quick and fast loading methodology to guage/measure rate of consolidation of primary as well as secondary processes.

To fulfil the objective literature has been reviewed in chapter-2

CHAPTER – 2

LITERATURE REVIEW

2.1 CONSOLIDATION: NATURAL PHENOMENON

Consolidation is likely to be linked to the changes in effective stress, which result from the changes in pore-water pressure as seepage flow progresses toward the drainage boundaries. Upon application of an external load, there is an initial increase in the pore-water pressure throughout the sample known as the initial excess pore water distribution. According to Darcy's law, the excess pore water pressures i.e., pressures in excess of hydrostatic are the driving force of seepage flow.

Flow takes place due to the hydraulic gradient generated by the initial excess pore pressure distribution. At any stage of the consolidation process, the pore-water pressures will vary within the soil layer. The distribution of the excess pore-water pressure at any given time after loading can be represented by an isochrones **[Powrie(1997**)]. Thus, the process of consolidation can be expressed as a series of the isochrones that graphically represent the relationship between time and the degree of consolidation over the depth of the soil stratum.Consolidation can be further characterized by the average degree of consolidation, which represents the consolidation of the stratum as a whole and eliminates the variable of depth**. [Taylor (1962)]**.Terzaghi's one-dimensional consolidation theory is commonly adopted to describe the dissipation of the excess pore-water pressure within a consolidating soil over time **[Terzaghi (1925**)].

In fact, Terzaghi's theory is often used in two and three-dimensional problems. Here, one-dimensional consolidation settlements are simply modified by a correction factor proposed by **[Skempton and Bjerrum (1957)]** to account for two- and three-dimensional effects. The behaviour of a consolidating soil subjected to the uniform initial excess pore-water pressure is commonly described in geotechnical textbooks in terms of both degree of consolidation isochrones and average degree of consolidation curves were given by **[Terzaghi (1943); Taylor (1962); Holtz and Kovacs (1981); Berry and Reid (1988); Powrie (1997); Atkinson (2007); Lancellotta (2009)]**. The average degree of consolidation behaviour of other initial excess pore-water pressure distributions has also been analyzed in terms of one-dimensional consolidation.

2.2 GENERAL THEORIES

Since the inception of Terzaghi's 1-D consolidation theory, several solutions have been developed for dealing with issues relating to the consolidation resulting from time-dependent construction loads. This has led time-dependent construction loads to be approximated as constant rate loading scenarios **[Terzaghi (1943); Schiffman (1958); Olson (1998); Zhu &Yin (1998); Conte &Troncone (2006); Hsu & Lu (2006)]**. They range from semi-empirical approximations to more sophisticated theoretical analyses. The simplest of these approaches is that proposed by [**Terzaghi (1943**)] where there is the consolidation settlement at time t (t $<t_0$) is computed assuming that the pressure at that time is applied instantaneously at t/2. Due to its simplicity, this method is still discussed in different textbooks, with suggestions to implement this as a graphical procedure that accounts for the construction time [**Craig (2004**)].

Later, more rational and analytical approaches were proposed by several others which are summarised here briefly [**Olson (1977**)] extended Terzaghi's one-dimensional consolidation theory to incorporate ramp loading, covering consolidation due to vertical flow, radial flow and combined vertical and radial flow. He discredited the ramp loading into very small incremental loads, and applied Terzaghi's one-dimensional consolidation theory and principle of superposition, to develop a mathematical expression for the excess pore water pressure.This was used in developing an expression for the degree of consolidation in terms of time factor. The solutions were also presented graphically in the form of U-T charts. In Terzaghi's one-dimensional consolidation theory, as well as [**Olson's (1977**)] extension, it is assumed that cvremains the same during consolidation.

[**Schiffman (1958**)] studied one-dimensional consolidation under time dependent loading where the permeability and coefficient of consolidation vary with time.[**Zhu and Yin (1998**)] developed solutions for one-dimensional consolidation under depth dependent ramp loading, where the applied pressure varies with time and depth.[**Conte and Troncone (2006**)] developed a solution for consolidation due to more general time dependent loading. [**Mikasa (1968**)] developed the one-dimensional consolidation theory in terms of compressive strains instead of excess pore pressures, which was later extended to multi-layered clays by [**Kim and Mission (2011**); **Conte (2006**)] proposed a method to analyse coupled consolidation of unsaturated soils under plane strain and axi-symmetric loading.

[**Rani (2011**)] studied the consolidation of a mechanically isotropic but hydraulically anisotropic clay layer subjected to axi-symmetric surface loads, allowing for compressibility of the pore fluid and solid constituents. Soil consolidation is often caused by external loadings, such as in building or embankment construction on clayey soil. Consolidation theory was originally developed by Terzaghi for the one dimensional case, and therein, exclusively considered vertical stress and strain, and neglected horizontal effects. Biot later extended Terzaghi's theory to two- and three-dimensional saturated soils.

Biot's reformalization of Terzaghi's theory is also contemporarily referred to as the true two- or three-dimensional consolidation theory, as it permits the total stress to be varied as a function of time during the consolidation process. Both theories assume that loading is instantaneously applied and maintained constant as a function of time; however, realistic loadings in construction are usually applied gradually as a function of time. In many cases, the loading process may proceed over a long period of time, such that a significant part of the consolidation may occur during the loading process. Moreover, during the construction of some special structures, such as silos or fluid tanks, the soils beneath will be subjected to cycles of loading and unloading that periodically repeat over time.

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For example, cyclical loading was evaluated and the corresponding solutions therein were developed. In particular, [**Geng (1962**)] used the Laplace transformtion obtain a general solution to the non-linear one-dimensional consolidation problem for soils that have undergone complicated cyclical loading.[**Conte (1966**)] used the Fourier transform to develop an analytical solution to the one-dimensional consolidation of saturated soil layers subjected to a general time-dependent loading. [**Cai (1981**)] derived a semi-analytical one-dimensional consolidation solution by considering the variation of cyclically loaded soil compressibility.

Process over time, wherein these loadings can significantly change the total stress as the soil consolidation develops. As a result, the impact of the external loading type should be considered in the analysis.Several solutions have been developed for classical one-dimensional consolidation problems consisting of time-dependent loading conditions in the past few decades. [**Schiffman (1958**)] was one of the first researchers to obtain a general solution of consolidation with loadings that linearly increased with time, which has subsequently been modified by later work. For example, cyclical loading was evaluated and the corresponding solutions therein were developed.

In Several investigators have successfully obtained analytical solutions to Biot's equations; however, due to the complexity of the coupled governing differential equations, the solutions to consolidation problems are generally limited to those with simple geometries and specific loading conditions. Several investigators have successfully obtained analytical solutions to Biot's equations; however, due to the complexity of the coupled governing differential equations, the solutions to consolidation problems are generally limited to those with simple geometries and specific loading conditions. Earlier work on three-dimensional consolidation problems has typically adopted a purely isotropic elastic model for the soil skeleton; however, the soil is often transversely isotropic due to its original deposition in horizontal bedding.

Furthermore, imposed strains after deposition can induce additional anisotropy, which in turn leads to a preferred orientation of the plate-shaped clay particles, and has been verified by laboratory tests that show that the stiffness and permeability of soils are directionally dependent. Thus, a more realistic solution to the consolidation problem should accommodate soil anisotropy. Existing solutions to consolidation problems all assume that particles and pore water are incompressible, which may not always be appropriate. In general, it is necessary to relax this assumption, and extend consolidation problem analysis to compressible materials. In this vein, [**Ji (1948**)] obtained a quasi-dynamic solution for the axisymmetric consolidation of a columnar cross-isotropic soil. More recently, [**Chen (1954**)] derived solutions for the axi-symmetric consolidation of transversely isotropic saturated soils using Laplace and Hankel transform techniques, but under instantaneously applied loading that was held constant with time.

Until now, investigations into the consolidation behaviour of semi-infinite transversely isotropic soils are sparse in literature, especially in regard to soils subjected to complex time-dependent loadings. Therein, soil particles and water pore compressibility and loading histories that more satisfactorily represent consolidation problems were analyzed. [**Chen (1954**)]originally derived these formulations for constantly loaded scenarios. Several numerical examples were investigated to verify the proposed approach, and therein, identified the influence of material anisotropy on pore pressure dissipation and soil settlement. Since the development of the three-dimensional consolidation theory by Biotin 1941, it has been used by researchers to solve many consolidation problems of soils. For example, [**McNamee and Gibson (1960**)] obtained the analytical solutions for the plane strain and axi-symmetrical consolidation problems with a semi-infinite body.

[**Schiffman and Funguroli (1968**)] studied the consolidation of a half-space medium on which a uniform tangential load was applied in the cylindrical coordinate system.[**Gibsonand Booker (1974**)]proposed the analytical solutions for the consolidation of a finite layer subjected to surface loading.[**Yue and Selvadurai (1980**)]analyzed the interaction between a circular, flat rigid indenter and a poro-elastic half-space saturated with a compressible fluid. For multi-layered soils, [**Vardoulakis and Harnpattanapanich (1986**)] adopted a numerical method using displacement functionsand integral transforms to analyze two-dimensional and three-dimensional consolidation problems in the Cartesian coordinate system.[**Booker and Small (1976**)]developed a finite layer analysis technique to deal with two-dimensional and three-dimensional consolidation problems of multilayered soils.

[**Senjuntichai and Rajapakse (1989**)] solved the consolidation problems of multi-layered soils by an exact stiffness method in the cylindrical coordinate system.[**Pan(1990**)] employed vector functions and a propagator matrix method in the cylindrical and Cartesian coordinate systems to obtain Green's functions in a multi-layered, isotropic, and poro-elastic half space.[**Wang and Fang (2001**)] analyzed the Biot consolidation problem for multi-layered porous media by a state space method in the cylindrical coordinate system. More recently [**Ai and Han (2003**)]used a state space method to obtain the solution for the plane strain consolidation of multi-layered soils.Numerical methods, such as the finite element method and the boundary element methodhave also been used to solve these complicated consolidation problems.Since Terzaghi printed his consolidation theory and therefore the principle of effective stress, analysis work on consolidation issues has greatly inflated.

The conventional consolidation theories typically neglected the non-linearity of soil for sensible function. Studies of non linear consolidation behaviour of saturated soil started from concerning forty years past. Many efforts are created to get analytical solutions for various forms of one-dimensional consolidation theories**.**

[Davis and Raymond (1965)] derived associate analytical resolution for the opposite constant loading case supported the assumptions that the decrease in porousness is proportional to the decrease in squeezability throughout the consolidation of a soil and therefore the distribution of initial effective pressures is constant with depth. With a similar assumption concerning the squeezability and porousness of a soil, **[Xie (1965)]** developed associate analytical resolution to the one dimensional consolidation drawback for time-dependent loading. The answer given by Davis and Raymond was considered a special case of it.**[Poskitt (1971)]** studied a lot of general one dimensional non linear consolidation drawback by employing a perturbation methodology; however no express resolution was given. For the linear consolidation of superimposed soils, some analytical solutions are reported.However, there appears to be no analytical solution for nonlinear consolidation taking the superimposed characteristics of soil into thought. This could be primarily owing to mathematical problem. During this study, associate analytical resolution comes for 1-Dimensional non linear consolidation of double-layered soil considering time-dependant loading, supported a similar assumption projected by Davis and Raymond apart from loading condition, and everyone the analytical solutions up to now developed square measure summarized within the tables. The nonlinear consolidation behaviour of double-layered soil is additionally mentioned.

The modelling and prediction of the mechanical behaviour of structures like building foundations, embankments, oil- cans, silos and ground anchors resting on a saturated layered soil system area unit common and essential issues in geotechnical engineering. Saturated soil consists of 2 phases, namely, a solid part (the soil skeleton) and a liquid part. These 2 phases act once the saturated soil is subjected to associate in nursing external load.

[Biot (1941)] developed a theory that may justify the complicated interaction between the solid and also the fluid, i.e., the supposed Biot's consolidation theory. This theory will accurately describe the method of soil consolidation during a three-dimensional condition. Within the past few decades, several researchers have advanced the data and theory of soil consolidation. For most mechanisms planned to make a case for secondary effects, one would expect a lot of noticeable secondary result within the laboratory than within the field. Laboratory values of c_v (and k) area unit seemingly to be too low as a result of retarding secondary effects area unit seemingly to be rather more vital within the laboratory than within the field because of the upper strain rate within the laboratory **[Taylor (1942); Barden (1965); Lo et al. (1976); Poskit (1967)]. The** time needed to finish the check victimisation the speedy consolidation methodology [**Sridhar an (1999)]** can be as low as 4-5 h compared with 1 or 2 weeks within the case of the traditional consolidation check on extremely clayey soils.

Rectangular conic section methodology needs knowledge of concerning 70% U for decisive c_v, δ_{100} etc. Where, c_v and δ_{100} area unit constant of vertical consolidation and supreme primary settlement, severally. However, the c_v values area unit not up to true

$c_v$due the consequences of secondary consolidation as secondary consolidation primarily starts at 60% U **[Sridhar an et al. (1995)]**. Also, it is not legendary to what extent c_v values area unit stricken by secondary consolidation.The trend or rate of settlement of pressure acting on clay will be determined at any time while not knowing the past history of pressure increment. It will be a supply of some helpful data like fast analysis of consolidation characteristics within the laboratory and field, time-compression knowledge of the current, past and future, kind and stage of consolidation, emptying conditions, time of load increment etc. **[Tewatia (2012)]**.

Using the speed of settlement idea **[Tewatia (1998); Tewatia and Satyendra N. Bose (2006)]**, a quickest fast loading methodology is projected to guage c_v minimizing the results of secondary consolidation that offers some estimate additionally that to what extent c_v is plagued by secondary consolidation. There square measure range of fast loading ways for vertical consolidation however few exist for radial consolidation **[Vinod et al. (2010); Tewatia et al. (2012a)]**.A fast loading methodology is projected for radial consolidation that's abundant quicker than the ways offered in literature with the higher than mentioned deserves additionally. The projected ways will be used even once the settlement or time of load increment is not glorious.

On the basis of literature review and keeping the objectives in mind a laboratory investigation has been made which is reported in chapter-3

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CHAPTER – 3

EXPERIMENTAL ANALYSIS

3.1 INTRODUCTION

Progress of consolidation with time of cohesive soil is measured by odometer test. Based on odometer test settlement–time and void ratio-effective stress curve are plotted.Now on the basis of void ratio-effective stress curve, compression index is calculated which is further used for total consolidation settlement at the end of dissipation of whole excess pore water pressure. Moreover coefficient of compression and coefficient of volume compressibility are also calculated at different effective stresses.Settlement-time curve is used for determining coefficient of consolidation.

Here coefficient of consolidation is calculated by square root of time method.Now based on coefficient of consolidation time factor is calculated. Time factor is a dimensionlessparameter which is connected with physical parameter time with the help of coefficient of consolidation. Now Terzaghi's one dimentional consolidation theory with fourier transform and Laurent transformare used for determining average degree of consolidation with time factor.

This gives the rate of consolidation or consolidation with time. Finally coefficient of permeability is calculated indirectly with the help of coefficient of consolidation and coefficient of volume compressibility. Permeability, swelling characteristics of bentonite and bentonite- sand mixture are two very important parameter for designing any type of waste disposal applications. Bentonite and bentonite –sand mixture is mainly used as a buffer material for disposal of radioactive waste. Concept of underground Disposal of hazardous material comes from nuclear industry.

Since environmental requirement for all types of underground disposal will converse towards the approaches developed for long lived radioactive disposal. Deep geological repositoryis based on multilayer barrier system concept. It ensures long term confinement and isolation of waste from environment.

3.2 MATERIALS AND METHODS

In the present study an experimental work was conducted to evaluate the consolidation characteristics and other parameters. The index as well as engineering properties have been evaluated. Details of material used and test procedures adopted are described in this chapter.

Characterization of clay:-

1. Moisture Content determination.
2. Density bottle test to determine specific gravity.
3. Hydrometer test to find grain size distribution.
4. Liquid limit & Plastic limit determination.
5. Ash Content & loss of ignition test.
6. Swelling Index

Characterization of sand:-

1. Density bottle test to determine specific gravity.
2. Sieve analysis to find grain size distribution.
3. To determine different properties of Compaction.
4. Determination of coefficient of permeability.

3.3 EXPERIMENTAL ANALYSIS

3.3.1 For Clay :

3.3.1.1 Moisture content determination:

Sample is being taken in three different crucibles of about 10 gms. And kept in the oven for about 24 hours at 55^0C.After 24 Hours weight is again taken through weighing machine and moisture content is determined.

Table No 3.1: Moisture Content Determination

Sample name	Initial weight (gm)	Dried weight (gm)	Water content (%)
A	10	9.3	7
B	10	9.5	5
C	10	9.6	4

Average moisture contentis **5.33%.**

3.3.1.2 Specific gravity by density bottle IS: 2720 (Part III)

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature, generally taken at 4 degree centigrade.

Test procedure:

Determine and record the weight of the empty clean and dry Density bottle, $\mathbf{W_1}$. Place about 150-200 gm of a dry sample (passed through the sieve No. 10) in the Density bottle .Determine and record the weight of the Density bottle containing the dry sample, $\mathbf{W_2}$. Add distilled water to fill about half to three-fourth of the Density bottle. Soak the sample for 10 minutes. Stir the mixture rigorously with a glass rod to ensure removal of all the entrapped air. Fill the Density bottle with distilled (water to the mark),

clean the exterior surface of the Density bottle with a clean dry cloth. Determine the weight of the Density bottle and contents, $\mathbf{W_3}$. Empty the Density bottle and clean it. Then fill it with distilled water only (to the mark). Clean the exterior surface of the Density bottle with a clean, dry cloth. Determine the weight of the Density bottle and distilled water, **W_4**. Empty the Density bottle and clean it.

The Specific Gravity of Clay Matter was found to be **2.79**.

3.3.1.3 Particle size distribution IS: 2720 (Part IV)

There are two types of grain size analysis, first is sieve analysis and second is hydrometer analysis. The grain size analysis is widely used in classification of soils.The particle size distribution (PSD) of a powder, or granular material, or particles dispersed in fluid, is a list of values or a mathematical function that defines the relative amounts of particles present, sorted according to size. PSD is also known as grain size distribution. **Particle Size**: A better indication of the fineness is to determine the particle size sdistribution. For example, one can determine the mass percentage below 10 µm or determine the mean particle diameter. The particle size of organic matter varies from below 75 µm to 300 µm or more. Thus a clay content might have the following distribution (on a mass basis): 0.7-0.8 % below 75 µm, 0.5 % finer than 150 µm, 35-40 % above 300 µm and 60-65 % above 600 µm.

- The percentage of sample retained on each sieve shall be calculated on the basis of total weight of sample retained in sieve.
- Cumulative percentage of soil retained on successive sieve is found.

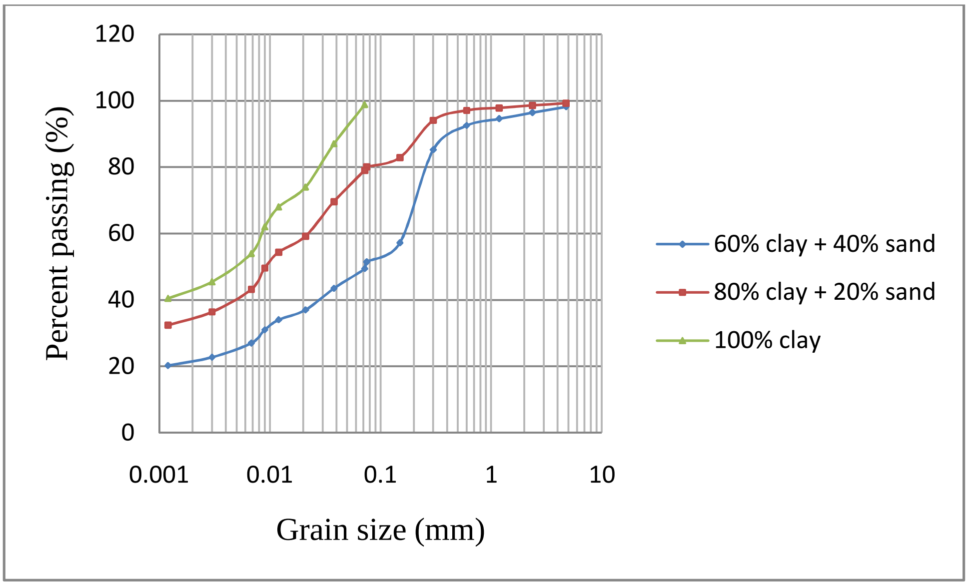

Fig 3.1: Grain distribution of mix clay

3.3.1.4 Liquid limit and plastic limit determination

3.3.1.4 (a) Liquid limit determination

Theoretical background:

Liquid limit is the minimum water content at which the soil is still in liquid state but has a small shearing strength against flowing. In other words it is the water content at which soil suspension gains an infinitesimal strength from zero strength. In the standard liquid limit apparatus from practical purposes, it is the minimum water content at which part of soil cut by groove of standard dimensions, will flow together for as a distance of 12 mm (1/2 inch) under an impact of 25 blows.

Calculations:

Liquid limit $W_L = \frac{W_1 - W_2}{\log_{10}\frac{n_1}{n_2}}$

W_1 = water content corresponding to blow n_1

W_2 = water content corresponding to blows n_2

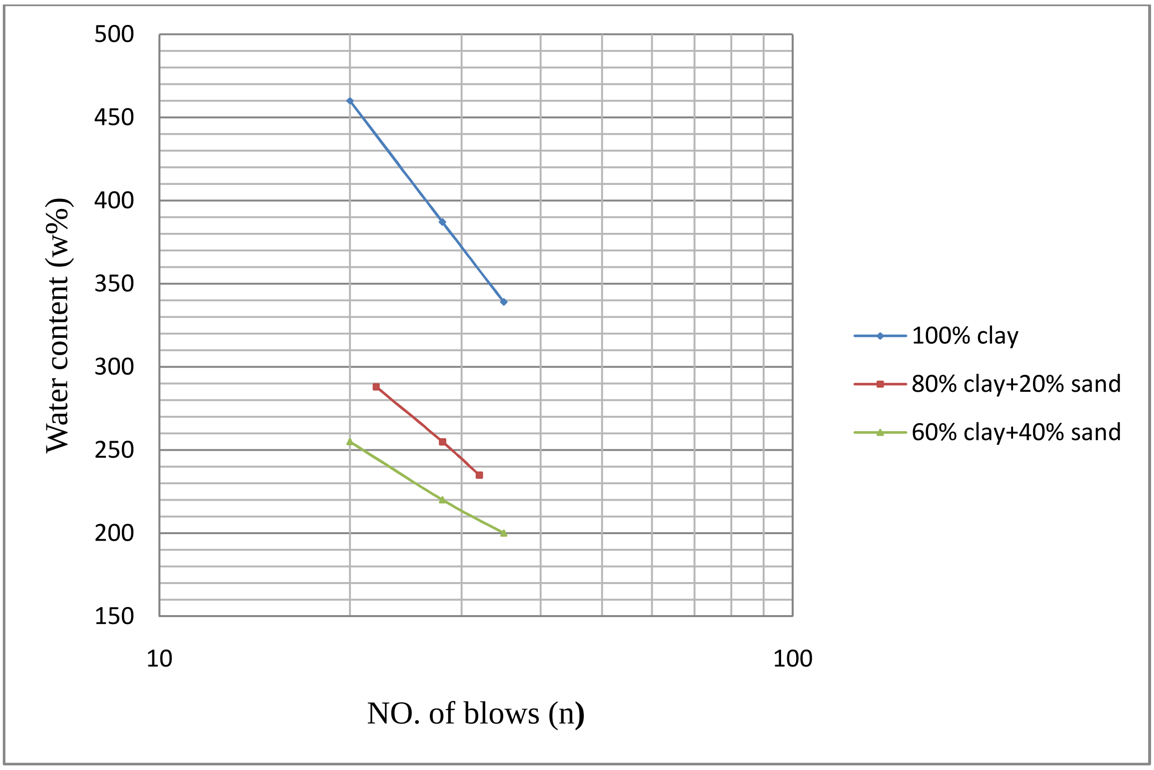

Fig 3.2: Liquid limit determination of mix clay

Liquid Limit of pureclay =**410%**

Liquid limit of 80% clay + 20% sand = **270%**

Liquid limit of 60% clay + 40% sand = **230%**

3.3.1.4 (b) Plastic limit determination

Plastic limit is the minimum water content at which a soil just deigns to crumble when rolled into a thread of 3 mm in diameter. This water content in is between the plastic and semi-soil states of soil.

Thus Plastic Limit of given Clay is **52%**

3.3.1.5 Swelling index IS: 2720 {Part XL (40)} 1977.

Free Swell Index is the increase in volume of a granular material, without any external constraints, on submergence in water.

Calculation:

Free Swell Index, (%) = $(V_d – V_k)/ V_k$ x 100%

V_d = Volume of the specimen read from the graduated cylinder containing

Distilled water.

V_k = Volume of the specimen read from the graduated cylinder containing

Kerosene

Free Swell Index is calculated as **600%.**

3.3.2 Characterization of Sand:

3.3.2.1 Specific gravity by density bottle IS: 2720 (Part III)

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature, generally taken at 4 degree centigrade.

.And the specific gravity of sand sample is **2.65**.

3.3.2.2 Sieve Analysisto find grain size distribution

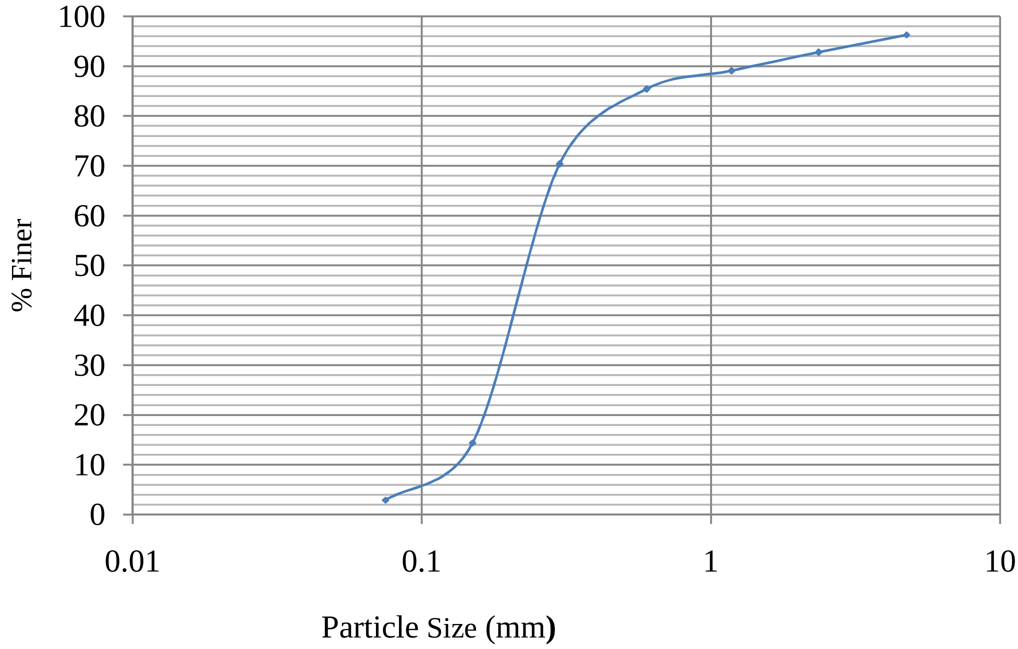

Fig 3.3: Grain size analysis of sand

Following terms were determined with the help of the curve in Fig 3.3 of sieving like

(a) Effective Diameter or Effective Size D_{10} = 0.14

(b) D_{30}& D_{60} , D_{30} = 0.19 , D_{60} = 0.26

(c) C_u & C_c , $C_u = (D_{60}/D_{10})$ = 1.857 , $C_c = [(D_{30})^2/(D_{60}) \text{ X } (D_{10})]$ = 0.992

(d) Nomenclature of Sand sample is silty sand.

3.3.2.3 Determination of compaction properties of sand layer by Standard proctor test IS 2720 (VII):1980

The standard proctor test was invented by R.R.Proctor(1933) for the construction of earth fill dams in the state of California.The bulk density and the corresponding dry density for the compacted soil are calculated from this.

The test is repeated with increasing water contents, and the corresponding dry density obtained is therefore determined. A compaction curve is plotted between the water content as abscissa and the corresponding dry densities as ordinates. The dry density goes on increasing till the maximum density is reached. This density is called maximum dry density (MDD) and the corresponding moisture content is called optimum moisture content (OMC).Optimum Moisture Content (OMC) of sand layer is **13.5%**

(a) Void Ratio of Compacted sand(e) = $(G.\gamma_w/\gamma_d)$ -1 =0.508 = **51%**

3.3.2.4 Determination of coefficient of permeability

Theoretical background:

The property of material which permits fluids to percolate through its voids is called permeability. According to Darcy's law in the laminar range the velocity of percolation is proportional to the hydraulic gradient. $V \propto i$

V= Ki

AV =Kai

AV= flow rate = Q= Kai

$$K = \frac{QL}{hA}$$

By Constant Head permeability method, Coefficient of permeability 'k' for this silty sand is determined and is about **4.26X10^{-4}** mm/sec.

3.4 CONSOLIDATION BY ODEOMETER

Consolidation is determined by using Odeometer after preparing the sample. Specification of apparatus:

1. Consolidometer consisting essentially;
    a. A ring of diameter = 60mm and height = 20mm
    b. Two porous plates or stones of silicon carbide, aluminium oxide or porous metal.
    c. Guide ring.
    d. Outer ring.
    e. Water jacket with base.
    f. Pressure pad.
    g. Rubber basket.
2. Loading device consisting of frame, lever system, loading yoke dial gauge fixing device and weights.
3. Dial gauge to read to an accuracy of 0.002mm. .
4. Stopwatch to read seconds.
5. Sample extractor.
6. Miscellaneous items like balance, soil trimming tools, spatula, filter papers, sample containers.

Procedure:

- Sample is prepared by in the mould.
- Saturate two porous stones either by boiling in distilled water about 15 minute or by keeping them submerged in the distilled water for 4 to 8 hrs. Wipe away excess water. Fittings of the consolidometer which is to be enclosed shall be moistened.

- Assemble the consolidometer, with the soil specimen and porous stones at top and bottom of specimen, providing a filter paper between the soil specimen and porous stone. Position the pressure pad centrally on the top porous stone.

- Mount the mould assembly on the loading frame, and centre it such that the load applied is axial.

- Position the dial gauge to measure the vertical compression of the specimen. The dial gauge holder should be set so that the dial gauge is in the begging of its releases run, allowing sufficient margin for the swelling of the soil, if any.

- Connect the mould assembly to the water reservoir and the sample is allowed to saturate. The level of the water in the reservoir should be at about the same level as the soil specimen.

- Apply an initial load to the assembly. The magnitude of this load should be chosen by trial, such that there is no swelling. It should be not less than 50 g/cm^2(5 kN/m^2) for ordinary soils & 25 g/cm^2 (2.5 kN/m^2) for very soft soils. The load should be allowed to stand until there is no change in dial gauge readings for two consecutive hours or for a maximum of 24 hours.

- Note the final dial reading under the initial load. Apply first load of intensity 0.1 kg/cm^2 (10kN/m^2) start the stop watch simultaneously. Record the dial gauge readings at various time intervals (and fill in the table). The dial gauge readings are taken until 90% consolidation is reached. Primary consolidation is gradually reached within 24 hrs.

- At the end of the period, specified above take the dial reading and time reading. Double the load intensity and take the dial readings at various time intervals. Repeat this procedure fir successive load increments.

- The usual loading intensity are as follows: 0.1, 0.2, 0.5, 1, 2, 4 and 8 kg/cm^2.

- After the last loading is completed, reduce the load to half (1/2) of the value of the last load and allow it to stand for 24 hrs. Reduce the load further in steps of 1/4th the previous intensity till an intensity of 0.1 kg/cm^2 is reached. Take the final reading of the dial gauge.

- Quickly dismantle the specimen assembly and remove the excess water on the soil specimen in oven, note the dry weight of it.

Three samples of Bentonite-Sand mixture are prepared with water content of 1.5 times of liquid limit (W_L).Pressure applied at each sample are 20kPa, 50kPa, 100kPa, 200kPa and 400kPa respectively.Corresponding to each load increment settlement-time curves are plotted for 24 hours. These curves are presented in fig 3.5, fig 3.6 and fig 3.7 i.e. for pure clay, 80%clay and 60% clay respectively.

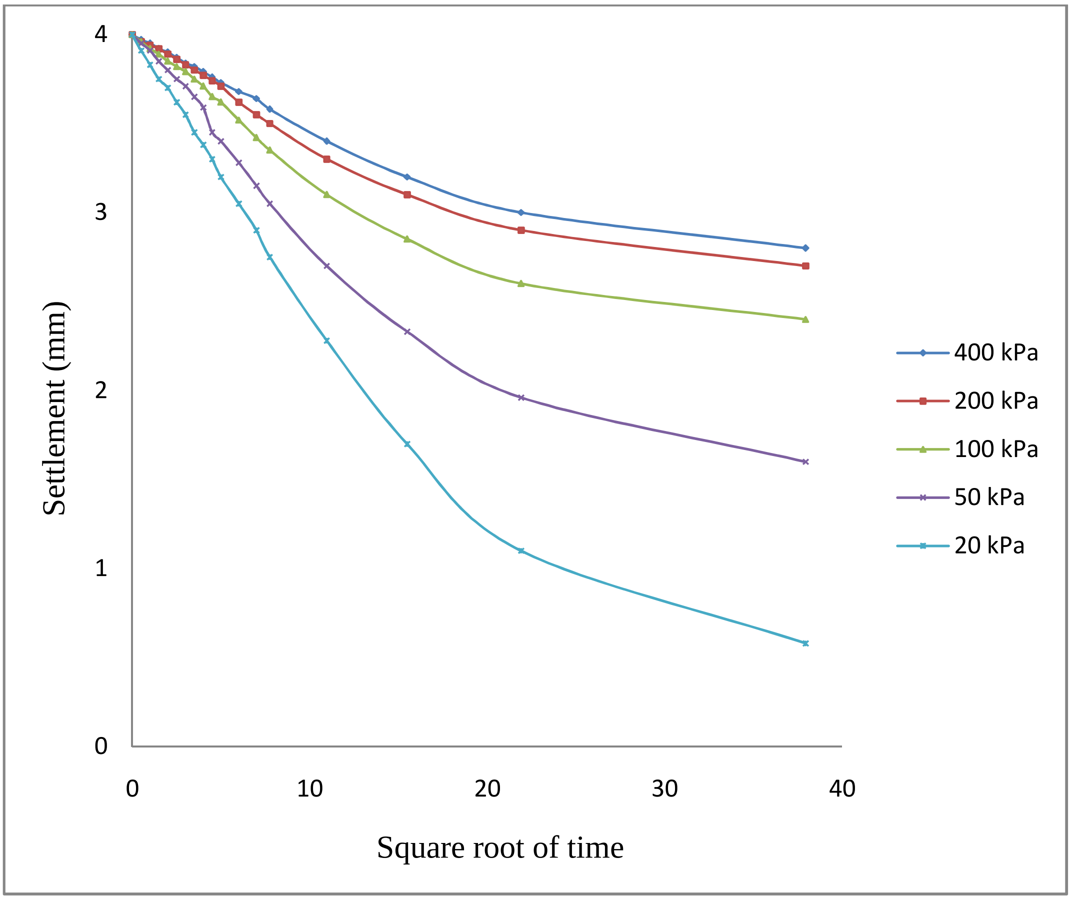

Fig 3.4: Settlement-square root of time curve for pure Bentonite

Initially when load is 20 kPa void ratios are very large. Void ratio decreases with time because effective stress increases with time. The rate of settlement or slope of the curve is very large initially.as loading is increased slope of curve decreases since void ratio decreases.Gapping between the curves is also decreasing as we moves towards higher loading. Initial void ratio for 20 kPa is very large hence overall settlement is also very large. But for higher loading initial void ratio is lesser then lighter loading hence less overall settlement.

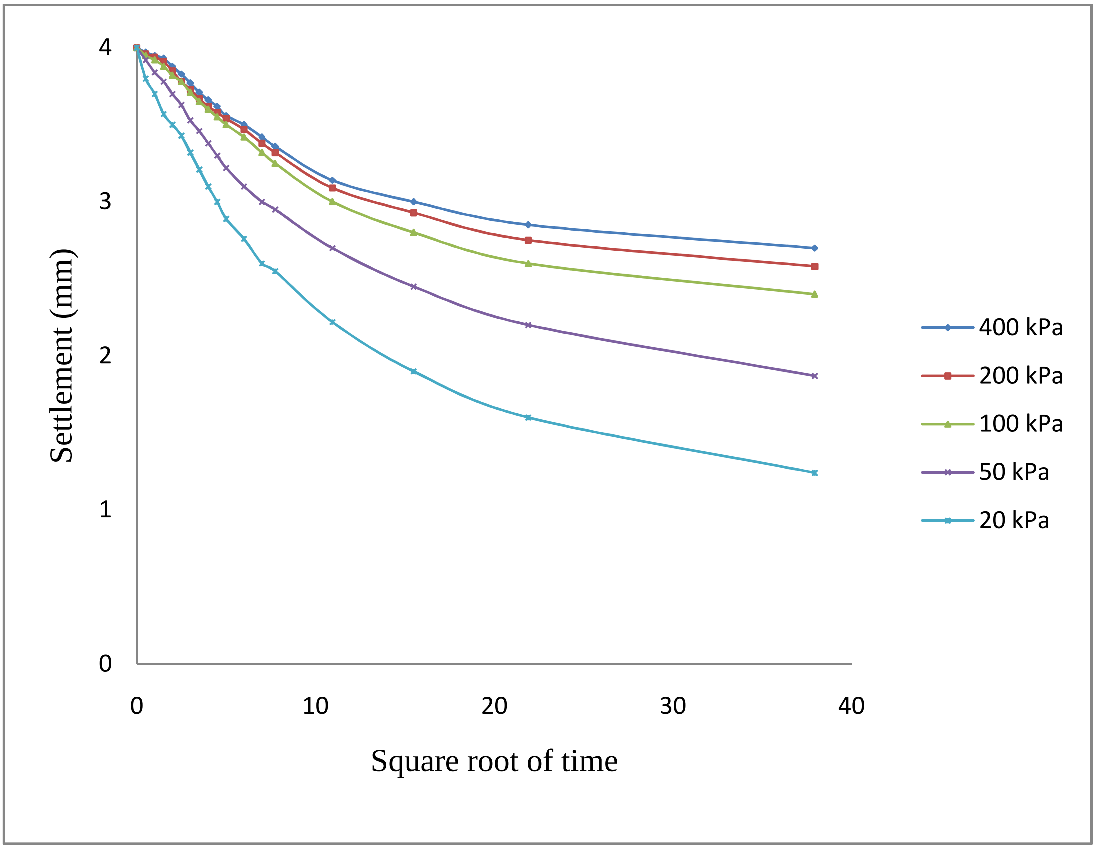

Fig 3.5: Settlement-square root of time curve for 80% Bentonite+20% Sand

Pattern of the curves are similar as for pure bentonite but overall settlement is less than pure bentonite. Gapping between the curves for different loading is smaller than pure bentonite. Since sand decreases the initial void ratio of pure bentonite which causes the decrease in overall settlement. The rate of settlement is lesser than pure bentonite. When small percentage of sand is added to the bentonite it decreases the initial void ratio and bentonite sand matrix behaves as a less permeable material.

As amount of sand in bentonite sand mix is increased overall settlement increases but the curves are stepper. Hence the rate of settlement is increased at fix higher percentage of sand. Gapping between the curves is also increased.It shows lesser overall settlement at higher percentage of sand.

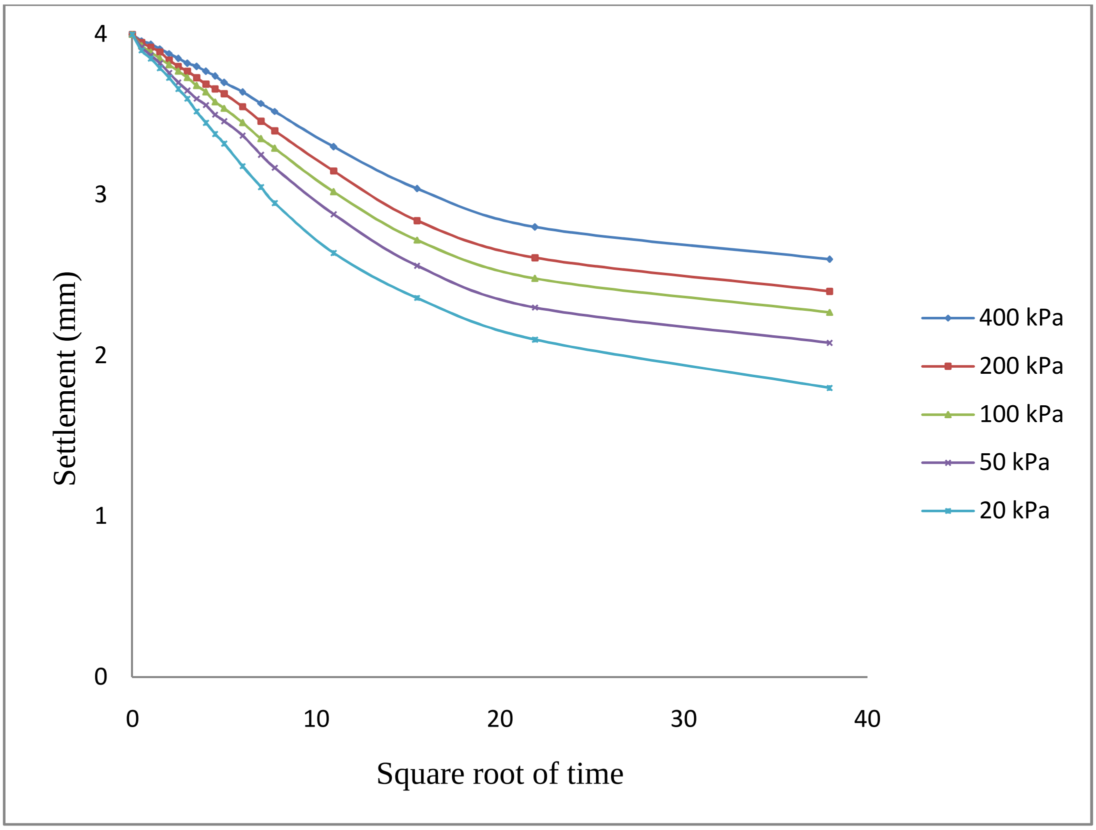

Fig 3.6: Settlement-time curve for 60% Bentonite + 40% Sand

On the basis of laboratory experiment results and discussion are reported in chapter-4

CHAPTER- 4

RESULTS AND DISCUSSIONS

4.1 Variation of void ratio with consolidation pressure

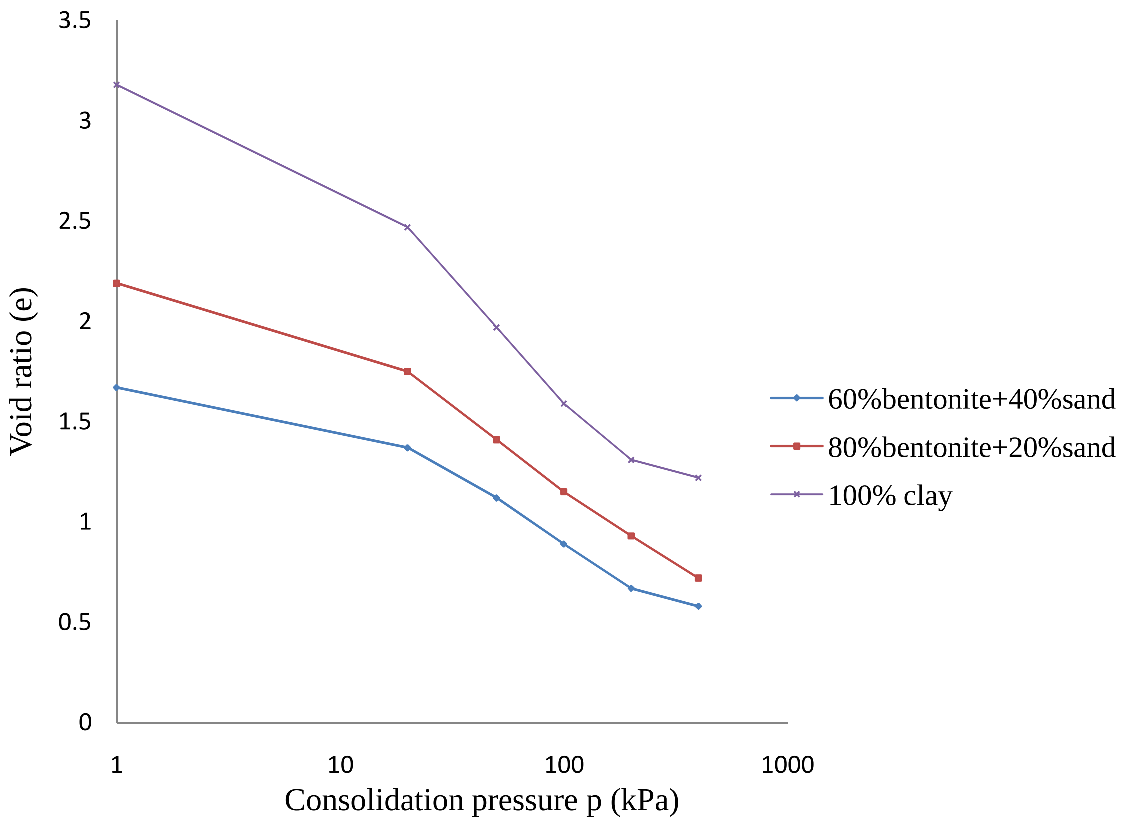

Fig 4.1: Void ratio – log p curve.

It is clear from the curves that higher percentage of bentonite have steeper slope compared to low percent bentonite content. Direct contact between coarse grained particles is prevented by bentonite particle by forming a coating around the coarser sand particle. Coarser particles float in the matrix provided by bentonite clay. Moreover final void ratio is higherfor higher per cent of bentonite although size of void is very small in case of bentonite but is has very large specific surface .hence final void ratio is large in comparison to mixed bentonite with sand.

4.2 Variation of coefficient of compressibility with consolidation pressure

Variation of coefficient of compressibility with consolidationpressure is presented in fig 4.2

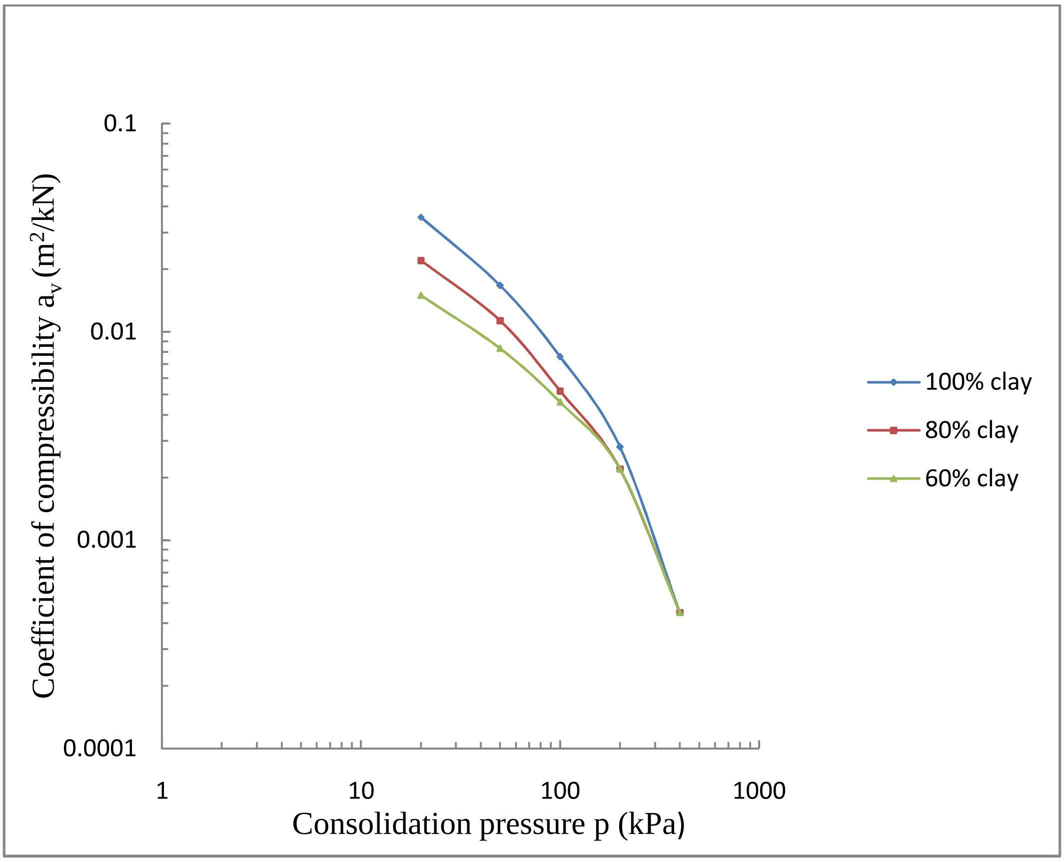

Fig 4.2: log a_v-log p curve for Bentonite-Sand slurry

Table No. 4.2 Coefficient of compressibility a_v (m^2/kN)

Load range(kPa)	Pure clay	80%clay+20%sand	60%clay+40%sand
0-20	0.0355	0.022	0.015
20-50	0.0167	0.01133	$8.33x10^{-3}$
50-100	$7.6x10^{-3}$	$5.2x10^{-3}$	$4.6x10^{-3}$
100-200	$2.8x10^{-3}$	$2.2x10^{-3}$	$2.2x10^{-3}$
200-400	$4.5x10^{-4}$	$4.5x10^{-4}$	$4.5x10^{-4}$

Coefficient of compressibility decreases with increase in effective stress. Rate of decrease of coefficient of compressibility decreases with decrease in bentonite content. Increased quantity of sand increases stiffness of mixture hence compressibility decreases with increase in sand.

4.3 Variation of coefficient of volume compressibility with consolidation pressure

Coefficient of volume compressibility varies with consolidation pressure. Variation is plotted on log-log plot as presented in fig 4.3

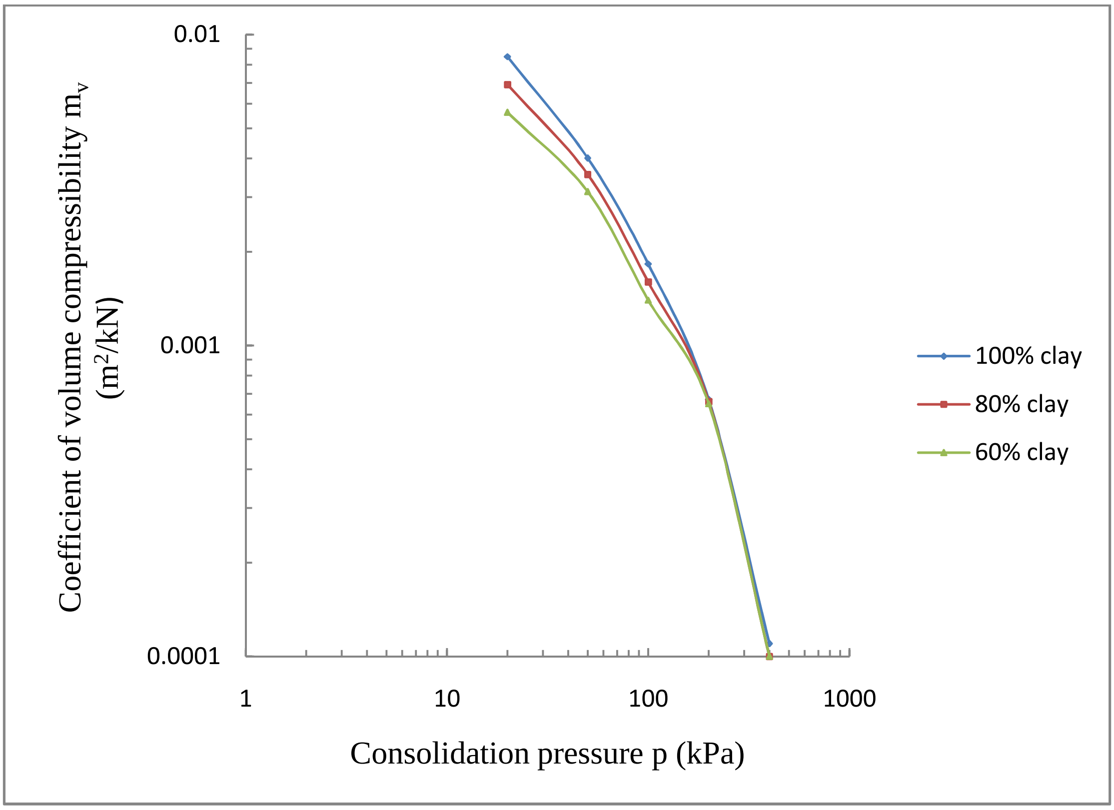

Fig 4.3: log m_v-log p curve for Bentonite –Sand slurry

Table No. 4.3 Coefficient of volume compressibility m_v (m^2/kN)

Load (kPa)	Pure clay	80%clay+20%sand	60%clay+40%sand
0-20	0.00849	0.0069	0.00562
20-50	0.004	0.00355	3.12×10^{-3}
50-100	1.82×10^{-3}	1.6×10^{-3}	1.5×10^{-3}
100-200	6.7×10^{-4}	6.5×10^{-4}	6.4×10^{-4}
200-400	1.1×10^{-4}	1×10^{-5}	1.7×10^{-5}

4.4 Variation of compression index with consolidation pressure

Compression index measures change in void ratio with increase in effective stress. Compression index is decreasing with increase in effective stress. Test results are represented in fig 4.4

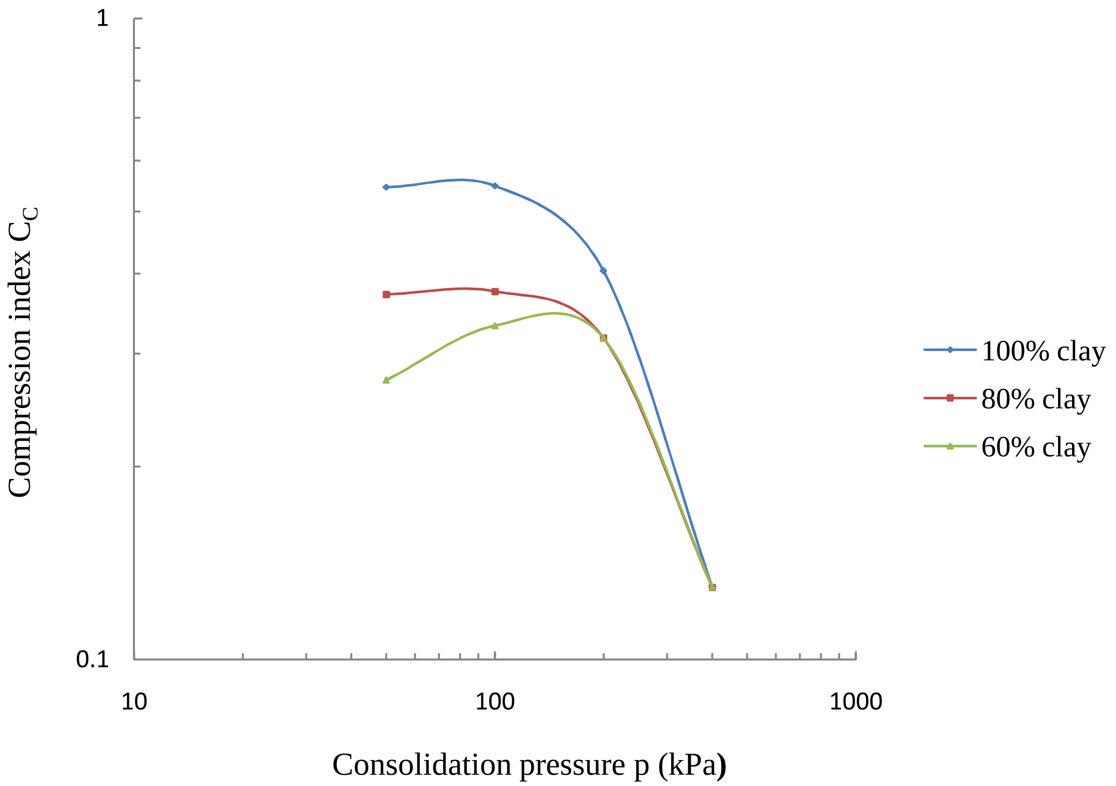

Fig 4.4: log C_C-log p curve for Bentonite-Sand slurry

Table No. 4.4 Compression index C_c

Load range(kPa)	Pure clay	80%clay+20%sand	60%clay+40%sand
20-50	0.54578	0.37106	0.272839
50-100	0.54822	0.3751	0.33182
100-200	0.40395	0.31739	0.317393
200-400	0.12984	0.30296	0.129842

Value of Compression index for pure bentonite is greater than mixed bentonite. Its value further decreases with increase in sand. Increase in sand increases stiffness of bentonite. Hence compression index decreases with increase of sand in bentonite.

4.5 Variation of coefficient of consolidation with consolidation pressure

Coefficient of consolidation represents combined effect of compressibility and permeability. It is assumed constant in Terzaghi's one-dimensional analysis but actually it varies with effective stress for a given soil. Here coefficient of consolidation is calculated by square root of time method corresponding to 30% of consolidation. It's variation with effective stress is shown in fig 4.5

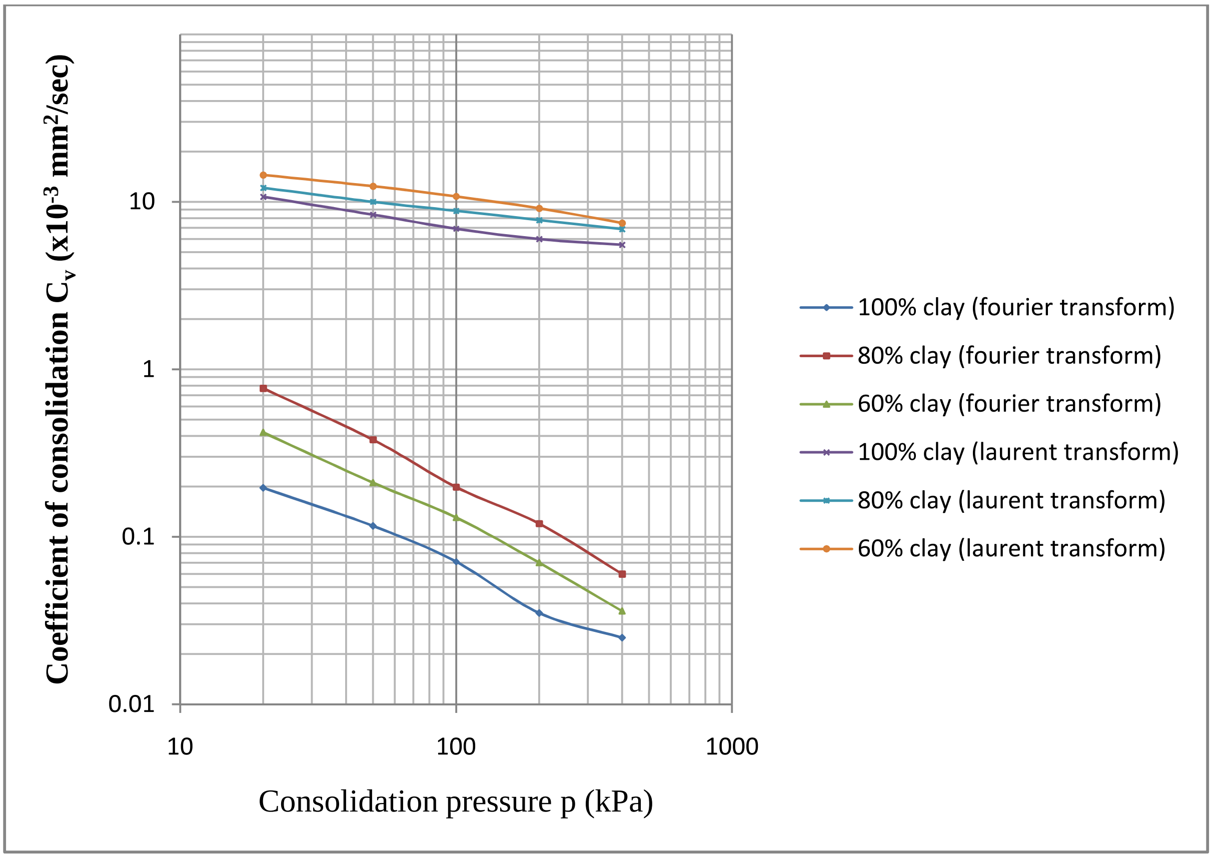

Fig 4.5 : Coefficient of consolidation- consolidation pressure curve

From the curve it is clear that coefficient of consolidation increases with increase of sand. Since coefficient of consolidation is directly proportional to permeability and inversely proportional to volume compressibility. Permeability effect of bentonite dominates the compressibility effect. Permeability of pure bentonite is less than bentonite sand mixture hence coefficient of consolidation of pure bentonite is less than mix bentonite with sand.

4.6 Variation of permeability with consolidation pressure

Permeability measures rate of flow of liquid through soil. Its value depends on void ratio, viscosity of liquid, and path of travel of liquid. Here permeability is measured indirectly by consolidation parameters. It's variation with void ratio is shown in fig 4.6

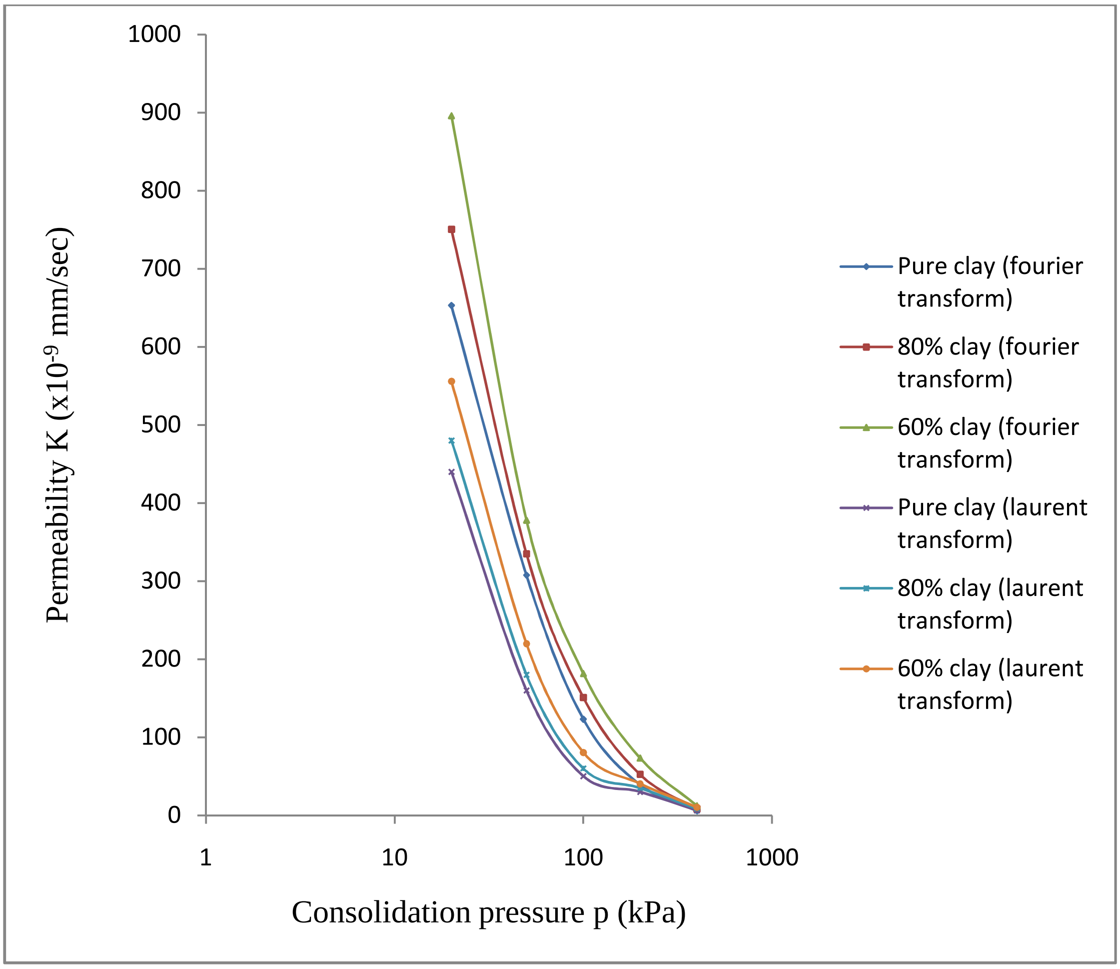

Fig 4.6: Permeability- consolidation pressure curve for Bentonite-Sand slurry

Table No. 4.6 Coefficient of permeability by Fourier transform

Load range (kPa)	Pure clay	80%clay+20%sand	60%clay+40%sand
0-20	$8.919x10^{-7}$	$7.506x10^{-7}$	$7.958x10^{-7}$
20-50	$3.276x10^{-7}$	$3.348x10^{-7}$	$3.782x10^{-7}$
50-100	$1.233x10^{-7}$	$1.41x10^{-7}$	$1.817x10^{-7}$
100-200	$3.93x10^{-8}$	$5.254x10^{-8}$	$7.345x10^{-8}$
200-400	$5.967x10^{-9}$	$2.221x10^{-8}$	$1.244x10^{-8}$

Table No. 4.7 Coefficient of permeability by Laurent transform

Load range (kPa)	Pure clay	80%clay+20%sand	60%clay+40%sand
0-20	$4.4x10^{-7}$	$4.8x10^{-7}$	$5.56x10^{-7}$
20-50	$1.6x10^{-7}$	$1.8x10^{-7}$	$2.2x10^{-7}$
50-100	$0.5x10^{-7}$	$0.6x10^{-7}$	$0.8x10^{-7}$
100-200	$3.93x10^{-8}$	$0.35x10^{-8}$	$0.04x10^{-8}$
200-400	$5.967x10^{-9}$	$0.08x10^{-8}$	$0.1x10^{-8}$

From the curve it is obvious that permeability of mix bentonite with sand increases with increase in sand quantity at a given void ratio. Specific surface area for sand is less than bentonite. Hence size of void increases with increases in sand quantity. Hence permeability of bentonite sand mixture is greater than pure bentonite. Value of coefficient of permeability is larger by fourier transform than Laurent transform but as load increases permeability converges to the common value.

4.7 Variation of average degree of consolidation with time factor

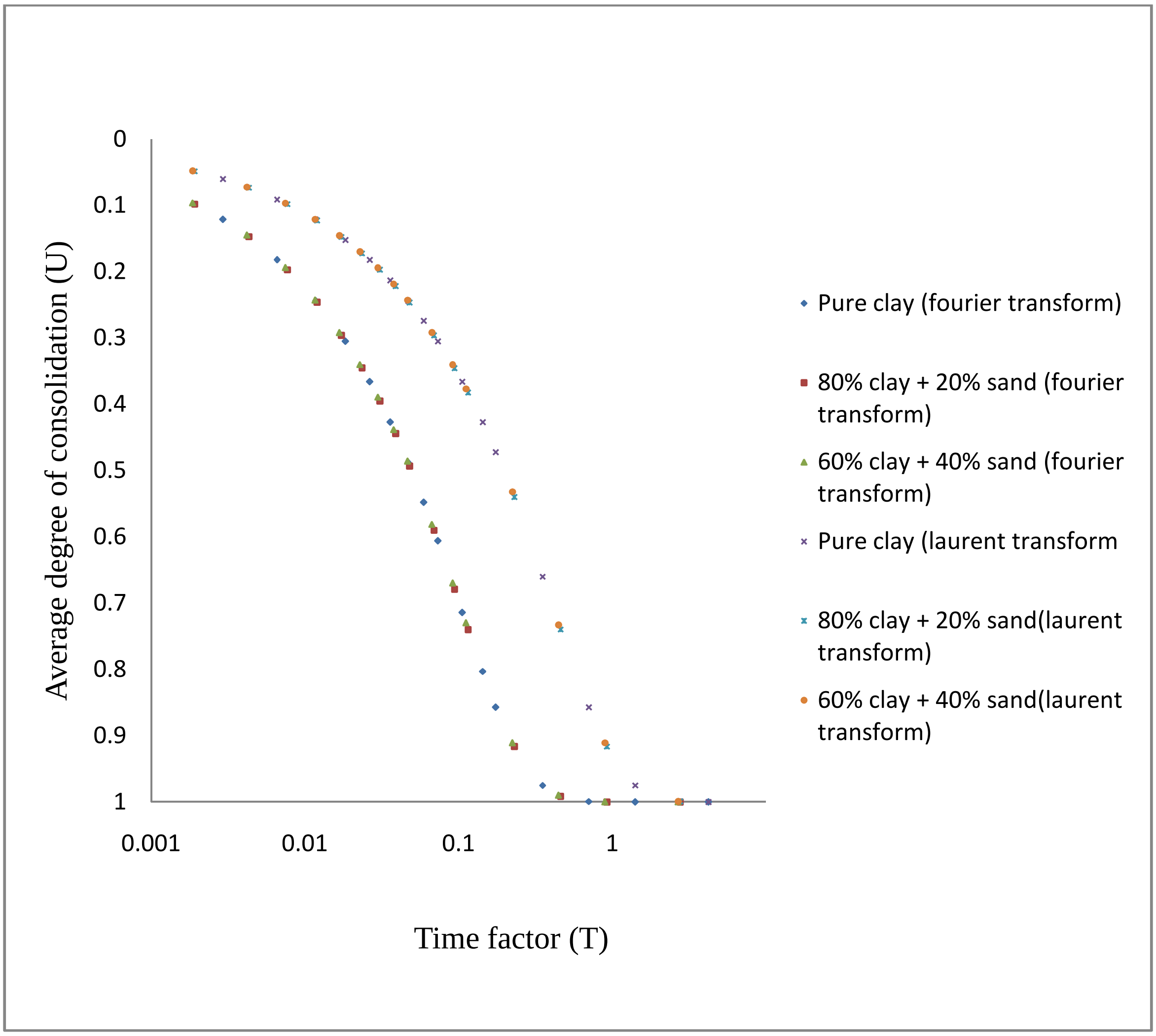

Fig 4.7 : Average degree of consolidation – time factor curve for different percentage of clay by Fourier and Laurent transform

Average degree of consolidation measures the rate of settlement with time. It is normally expressed in terms of non-dimensional parameters. Variation of degree of consolidation with time factor is expressed in fig 4.7

From the above curves it is clear that rate of consolidation increases as the percentage of sand in bentonite increases. Since permeability increases as the percentage of sand in bentonite increases hence increase of permeability causes increase of rate of consolidation.

It is also clear from the curves that Laurent transformation gives faster rate of consolidation than fourier transformation.

CHAPTER-6

CONCLUSION

Effect of loading and effect of sand on consolidation behaviour of bentonite clay is studied. These variations are represented clearly by different coefficient used in consolidation phenomenon. Finally rate of consolidation are compared using fourier and laurents transformation. From the study the following conclusions are drawn.

(1) Coefficient compressibility of pure clay decreases with increment of loading. Variation is non-linear. When loading is increased from 20 kPa to 400 kPa. it's value decreased to 98.73%.

(2) Coefficient of compressibility decreases with increasing percentage of sand in pure clay. Decrease in compressibility is 30% corresponding to 20% sand while it is 57.74% corresponding to 40% sand in clay.

(3) Compression index of clay decreases with increasing load. At lower loading variation is very small but at higher loading it decreases sharply.

(4) Compression index decreases with increasing percentage of sand in clay. Decrease in compression index corresponding to 20% sand is 32% and corresponding to 40 % it is 50% at 20 kPa to 50 kPa.

(5) Coefficient of consolidation decreases with increasing load. It's value decreases to 30% when loading increased from 20 kPa to 400 kPa.

(6) Coefficient of consolidation increases with increase in percentage of sand in pure clay. Increase in coefficient of consolidation is 3.5% corresponding to 20% of sand whole it increases to 34.79% at 20 kPa.

(7) Coefficient of permeability decreases with increasing load on clay. Decrease in Coefficient of permeability is 99.33% when loading is increased from 20 kPa to 400 kPa.

(8) Coefficient of permeability decreses to 15.84% when 20% sand is added while after 20 percent. it's value decreases to 6.02% with another 20% addition of sand.

(9) Rate of consolidation is increased using laurent transformation compaired to fourier transformation.

5.1 Limitations:

(1) Change in behaviour of soil is very complex. It depends upon a lot of factors. So exact calculation of the results are not 100% accurate.

(2) Bentonite is a compressible soil. Hence coefficient of consolidation is not constant during consolidation. But results are based on Terzaghi's one dimentional consolidation theory in which coefficient of consolidation is assumed constant.

(3) Value of coefficient of consolidation is calculated by square root of time method which is an empirical method. Hence calculation of coefficient of consolidation is not exact but empirical.

(4) Coefficient of permeability is calculated based on coefficient of consolidation hence error in calculation of coefficient of consolidation is inherent in calculation of coefficient of permeability.

(5) Initial pore water pressure is assumed to be constant in calculation of average degree of consolidation but actually it varies sinusoidal with depth.

5.2 Future scope:

(1) Variation in consolidation behaviour of bentonite is studied with 20% increment of sand. Change in behaviour can be observed for 5%, 10%......i.e. more smaller increment of sand.

(2) Mainly consolidation behaviour is studied here but swelling behaviour of bentonite can be studied.

(3) Based on consolidation parameters hydraulic conductivity is calculated. But heat conductivity is another important parameter for buffer material which is necessary to calculate for designing underground waste disposal.

(4) Chemical properties of bentonite are another important factors which affect the chemical behaviour of bentonite with waste material.

CHAPTER-7

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