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STUDY USE RECYLED / WASTE POLYTHENE CHIPS, CEMENT AND SAND TO REDUCE THE SHRINKAGE POTENTIAL OF CLAY LINER IN ARID / SEMI ARID REGIONS

Submitted By

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STUDY THE USE OF RECYCLED /WASTE POLYTHENE CHIPS, SAND AND CEMENT T0 REDUCE THE SHRINKAGE POTENTIAL OF HIGH PLASTIC CLAY LINER IN ARID/ SEMI ARID REGIONS

DISSERTATION FOR THE DEGREE OF MSC

NATIONAL INSTITUTE OF TRANSPORTATION RISALPUR
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MAJOR SHAFQAT MAHMUD

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ABSTRACT

STUDY USE RECYLED / WASTE POLYTHENE CHIPS, CEMENT AND SAND TO REDUCE THE SHRINKAGE POTENTIAL OF CLAY LINER IN ARID / SEMI ARID REGIONS

BY

SHAFQAT MAHMUD

Clay rich soils used as liners and covers for containment of industrial waste and other materials, are compacted wet of optimum with a view to minimize the hydraulic conductivity of the liners and covers. Resultantly the shrinkage potential of the clayey soil increases in arid / semi arid regions where liners and covers are subjected to seasonal drying.

The use cement, sand and waste / recycled polythene to reduce the shrinkage potential of the compacted high plastic clay liner was studied. The experimental study comprised a number of volume shrinkage, permeability and unconfined compression tests on samples of compacted high plastic clay (PI: 31 or more) with various compositions of clay, sand, cement and polythene chips by percent of weight. Variation of volume shrinkage potential, i.e.,the governing parameter, unconfined compression strength and permeability were analyzed with samples compacted at dry of optimum moisture content (OMC), at OMC and wet of OMC at constant compactive effort (modified compactive effort). Variation of volume shrinkage potential was analyzed with varying amount of sand, cement and varying sizes and proportion of polythene chips also. Samples of composite material having varying proportion of sand, cement and polythene chips were also analyzed for volume shrinkage potential and test results of samples having desired volume were analyzed for variation of permeability and unconfined compression strength.

Evaluation of test data of specimen compacted at varying moisture content showed that volume shrinkage potential increased with increase in molding water content, exceeding the desired limit whilst resultant reduction in unconfined compression

Strength and permeability remained within permissible limits, To reduce the volume shrinkage potential, each of sand, cement and polythene with varying proportion were added to clay but results showed no reduction in volume shrinkage potential. Polythene was found unsuitable to be used for reducing volume shrinkage potential as increase in its size and proportion would make compaction and its uniform distribution over entire sample difficult and would reduce bounding between soil particles. Either sand or cement alone also did not produce effective results.

Tests performed on composite material gave better results and showed a positive trend towards reduction of volume shrinkage potential and samples having 30 % sand,6 % cement with varying amount of polythene chips i.e., 0.5,0.75,1.0 %. Showed reduction of volume shrinkage potential to average of 3.5 %. Results of unconfined compression strength and permeability tests performed on these three combinations were well within limits i.e., more than 360 Kpa and less than 0.0785×10^{-9} m/s respectively.

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CHAPTER 1

INTRODUCTION

1.1 Compacted Clay Liner

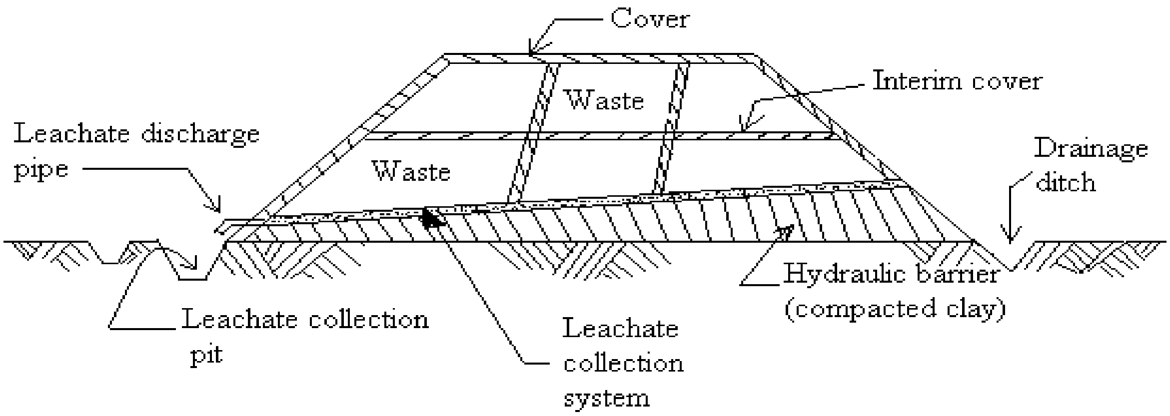
The use of modern materials for waste material containment, like geosynthetics, geomembranes etc has not minimized the importance of compacted clay liners. Clay cores in dams, clay dikes and clay liners have long been used to impede flow of water. Compacted clay liners are constructed by compacting naturally occurring clay or mixtures of natural or commercially processed clay and other soils. A common requirement is that the 'clay' should contain a minimum percentage of fine-grained material of 20 to 50% passing ASTM No. 200 sieve, have a Plasticity Index of about 10 to 20 and have a hydraulic conductivity (Permeability) below 10^{-9} m/s (Evens, 1991). In order to achieve such a low value of hydraulic conductivity, the clays are typically compacted wet of optimum. The hydraulic conductivity and volumetric shrinkage potential of geotechnical material is probably the most important parameter involved, in the assessment of contaminant migration in the sub surface and the design of barriers for hazardous waste control. Permeability tests in this regard can be performed in the laboratory or in the field.

Schematic diagrams of landfills to include compacted clay liners are shown in Figure 1.1.

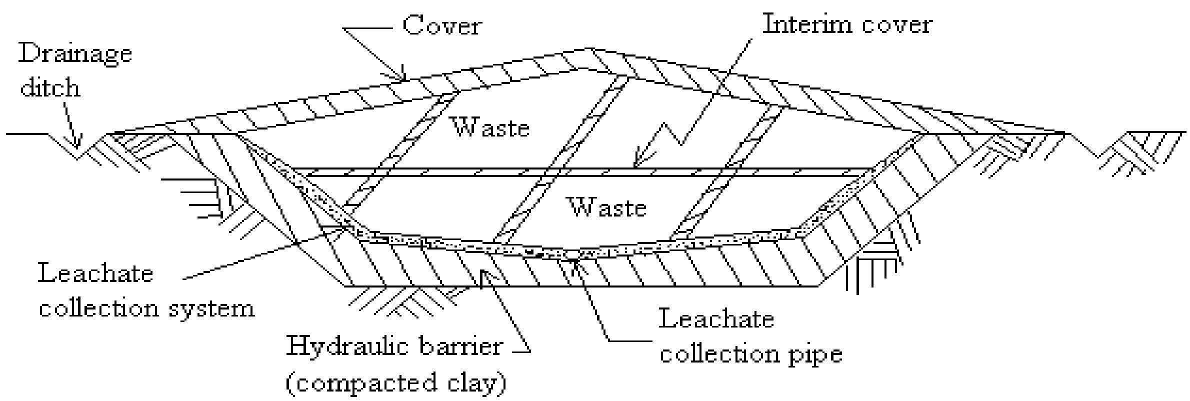
1.2 Problem Statement

Clay rich soils used as liners and covers for waste containment units are typically placed wet of optimum water content. Although, the hydraulic conductivity is minimized through this process but in relatively arid or semi arid zones or sites where the clay could be subjected to seasonal drying, this practice could be counter productive if the liner eventually desiccates. Large cracks can occur in wet, compacted clays when these are allowed to dry. This is result of shrinkage potential of soil which increases with the increase in molding water content. This problem can be controlled by using materials like cement, sand etc. which would reduce the shrinkage potential to some percentage if added to clay without adversely affecting the permeability of the liner

In a study on clay liners, used Polythene bags being health / environmental hazard have been used to reduce Shrinkage potential (Waqar, 1995). This study was carried out using low and medium plastic clays only.



a. Above Ground Landfill



b. Below Ground Landfill

Fig 1.1 : Schematic Diagrams of Landfills (After Daniel,1987)

In continuation of above mentioned study, the behavior of Used Polythene Bags, sand and cement content in high plastic clay liners was studied and behavior of various compositions of polythene chips, sand and cement in a composite clay liner was also evaluated.

1.3 Objectives

1.3.1 Main Objectives

The two primary objectives of this study were to design a clay liner for arid / semi arid regions by adding certain amount of cement, sand and polythene chips with low shrinkage potential as well as low permeability besides having adequate strength and to establish the suitable range of these materials to achieve the best suited results in a high plastic clay soils.

1.3.2 Sub Effort

Various sub efforts to support the main objectives were as follows:

1.3.2.1 Literature Review

This effort consisted of reviewing all available literature pertaining to liners in general and compacted clay liners in particular, previous research on various factors effecting hydraulic conductivity, strength and volumetric shrinkage behavior of compacted clays and the behavior of cement stabilized clays. This effort involved the review of literature concerning methods of laboratory testing also. This effort is described in Chapter 2.

1.3.2.2 Selection of Soil

This effort involved selection of clay rich soil to be used in the research work. Since this study involved use of High plastic clay soil in arid/semi arid regions and Kashmore falls in arid/semi arid zone and its soil is also mostly high plastic clay. Hence Kashmore soil was selected for this study. After classification / identification through testing at the site , the desired soil was excavated and brought to the laboratory for this research work. This effort has been described in Chapter 3.

1.3.2.3 Laboratory Testing

This effort included:

a. Evaluate the effect of variation of compaction moisture content on volumetric shrinkage by compacting the soil samples at constant compaction effort and varying moisture content i.e., dry of optimum, at optimum and wet of optimum. This effort is described in Chapter 4.

b. Evaluate variation in volumetric shrinkage of a high plastic clay for arid/semi arid regions by varying cement, sand and polythene chips content at constant moisture content and compaction effort. This effort is described in Chapter 5.

c. Evaluate the effect of variation of polythene chips content in a composite clay liner at constant moisture content and compaction effort. This effort is described in Chapter 6.

1.3.2.4 Statistical Analysis

This effort involved statistical analysis of test results to establish:

a. Confidence level of test results and trend in variation of volumetric shrinkage, unconfined compression strength and permeability with varying moisture content and constant compaction effort.

b. Trend in variation of desired properties with varying amount of cement, sand and polythene chips mixed in high plastic soil and compacted at constant compaction effort and moisture content. The analysis has been carried out in Chapters 4 & 5.

c. Trend in variation of desired properties with varying polythene chips content in composite clay liner compacted at constant moisture content and compactive effort. These analyses have been carried out in Chapter 6.

1.3.2.5 Discussion on Results

This effort was meant to discuss the results obtained and analyzed in chapters 4, 5 and 6. This effort is given in Chapter 7.

1.3.2.6 Summary, Conclusions and Recommendations

This effort was to summarize the research work, to deduce salient conclusions from the research and to recommend suitable amount of cement, sand and polythene chips in High plastic clay to work satisfactorily as clay liner in arid / semi arid regions and to make recommendations for further research work. This effort is included in Chapter 8.

1.4 Experimental Matrix

Experimental matrix was formulated to summarize the number of laboratory tests required to be performed with varying amounts of cement, sand and Polythene chips in order to deduce the behavior of soils. Amounts of cement, sand and polythene chips to be added to clay were arbitrarily selected. It was decided that volumetric shrinkage, being the governing property, was given priority and this property was checked for all the samples. However, permeability and unconfined compression test were performed on samples compacted wet of optimum and having desired volumetric shrinkage i.e., less than 4 %. Table 1.I & 1.2 gives the experimental matrix for this thesis work.

Table I .1 Experimental Matrix for Laboratory Work

MATERIAL USED	MATERIAL ADDED % CONTENT BY WEIGHT	NO OF TESTS
		VOLUMETRIC SHRINKAGE TEST
CH Soil only	2% Dry OMC AT OMC 2% Wet OMC	3 3 3
CH Soil with Cement	3 % 6 % 9 %	3 3 3
CH Soil with Sand	15 % 30 % 45 % 60 %	3 3 3 3
CH Soil with Polythene Chips Size		
a. 0.5x0.5 cm	0.5 % 0.75 % 1.0 %	3 3 3
b. 1x 1 cm	0.5 % 0.75 % 1.0 %	3 3 3
c. 1.0x 0.5 cm	0.5 % 0.75 % 1.0 %	3 3 3
Total No of Tests		57

Table I .2 Experimental Matrix for Composite Material

Type of Sample	Clay plus Added Material Content by Percent Weight			No of Laboratory Tests
	[P]	**[S]**	**[C]**	**Volume Shrinkage Test**
Sample # 1	0.5	15	3	3
Sample # 2	0.75	15	3	3
Sample # 3	1.0	15	3	3
Sample # 4	0.5	30	3	3
Sample # 5	0.75	30	3	3
Sample # 6	1.0	30	3	3
Sample # 7	0.5	15	6	3
Sample # 8	0.75	15	6	3
Sample # 9	1.0	15	6	3
Sample # 10	0.5	30	6	3
Sample # 11	0.75	30	6	3
Sample # 12	1.0	30	6	3
Total No of Tests				36

Note : [P] , [S] , [C] in the column headings stand for Polythene chips, Sand and Cement content mixed in the clay respectively.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Contamination of the subsurface environment due to hazardous and toxic wastes has become most important environmental problem. In most of the cases, hazardous waste site conditions include contamination of ground water and soil necessitating control and remediation systems interacting with the subsurface environment. The Geotechnical engineer’s role in this regard includes:

a. Planning and implementation of site assessments to characterize the type, distribution and migration of contaminants in the subsurface.

b. Development of remedial alternatives to control contaminant migration to protect public health and environment.

Principles of geotechnical engineering employed in conventional practice are frequently applied to these hazardous waste control problems. However there are specific aspects of the art and practice of geotechnical engineering that must be considered differently from those for more conventional problems (Evans, l 991).

As Superfund site remediation chugs along, the methods of dealing with hazardous waste have similarly evolved. At one time, old-fashioned "excavation and removal" was the popular choice for treating pollutants, but stabilization and solidification has since emerged as a viable option so modified clays have proved to be of great help in this context (Alther et al. 1990).

Until recent decades, landfills for hazardous and non-hazardous wastes were little more than holes in the ground in developed countries heightened awareness of the consequences of poor disposal practices and increased regulations of landfills have spurred major improvements in the land disposal of wastes for example the use of synthetic flexible membrane liners, and leachate detection and collection systems), but the vast majority of landfills are still lined with simply a layer of compacted clay (Elsbury and Sraders. 1989).

2.2 Composition of Waste

It is impossible to characterize solid waste because composition varies tremendously. Some solid wastes such as building debris are relatively inert and produce little or no gas or leachate. Municipal refuse tends to be compressible, degradable and capable of producing both gas and leachate. Industrial wastes are sometimes toxic and may be flammable, volatile, and reactive with other compounds. The term 'solid waste' is itself poorly defined, some people consider sludges that flow, to be solid and some consider them to be liquid. At some locations, liquids may be dumped into a solid-waste landfill and at other locations they may not.

The ground engineer should assume the worst about the composition of the waste, unless there is good reason to believe otherwise, The worst assumption is that the waste is compressible, contains liquids, will produce gas, will produce leachate, and will contain enough nutrients to attract burrowing animals or to support plant roots (Daniel, 1987).

2.3 Landfill Gas Generations and Movement

The gases produced at most landfill sites are Methane and Carbon dioxide. Methane tries to rise, and so if an impermeable cover is placed over the entire landfill, it will eventually move laterally beyond the landfill boundaries to a permeable soil where it can escape to the atmosphere. The distances involved can be thousands of feet.

To avoid such problems the methane gas must be properly vented. Each cell cover should be shaped so that a vent is located at the uppermost slope of the bottom of the cover. The carbon dioxide produced, being 1.5 times denser than air and 2.7 times denser than methane, separates and moves to the bottom of the landfill. Since it is readily soluble in water it will enter, the leachate that has formed there. If the landfill liner is inadequate, the gas will escape (generally along with the leachate, although it can escape independently because of its higher diffusion characteristics) and enter into the groundwater. Once dissolved, it lowers the water's pH causing an increase in hardness and mineral content.

To collect the carbon-dioxide gas a sand-gravel drainage layer above the bottom of the landfill liner is needed, or a bulky geotextile of adequate transmittivity can be used, along with a perforated pipe collection system for gathering the gas (and leachate) for proper disposal at the ground's surface(Koerner, 1985).

2.4 Landfill Leachate Generation and Movement

Generally high moisture content of a landfill (10 to 35 percent values are not uncommon), any rainfall or snow melt which gets through the landfill cover, and lateral moisture movement from beyond the landfill into it will all react with the waste materials themselves to form a polluted, liquid called leachate, which often has dreadful characteristics.

Leachate should be properly collected above the primary liner at the bottom of the landfill, adequately treated, and disposed off in accordance with accepted environmental engineering principles. The basic strategy should be to keep the leachate from escaping the landfill site. To begin with, both primary and secondary liners can be used to achieve this end (Koerner 1985).

2.5 Types of Landfill Liners

Landfill liners for both bottom and side use are constructed from a wide range of materials. R.M.Koerner (1985) classified and described the liners as:

2.5.1 Rigid

- Shotcrete or gunite
- Concrete
- Soil cement
- Bituminous concrete (asphalt)
- Bituminous panels

2.5.2 Flexible (Natural Materials)

- Compacted soils
- Chemically treated soils
- Bentonite clays

2.5.3 Flexible (Synthetic Materials)

- Geomembranes (Plastics and Elastomers)

2.6 Rigid Liners

2.6.1 Shotcrete or gunite liners

For this type of liner a mixture of cement, sand and water is blown, under a pressure of 20 to 100 lb/in^2, onto the prepared bottom and sides of the waste pit. Usual mix proportions are 1 part cement to 4 or 5 parts sand, with water / cement ratios in the range of 0.4 to 0.6. Nearly vertical sides can be treated. Many swimming pools are constructed by this method. To prevent cracking during the curing period, as well as afterwards should settlement occur, wire mesh is often placed before shotcreting. For landfills, the technique is not often used because of its high cracking potential for all the most rigid of foundations, e.g., a landfill built on rock(Koerner,1985).

2.6.2 Concrete Liners

These liners are constructed in much the same way as is rigid highway pavement. The aggregate is stone and sand, which is then mixed with cement to which water is added to make the final product. Concrete is usually brought to the job site from a concrete plant ready for placement on a prepared subgrade. It can be reinforced with wire mesh or reinforcement bars, depending upon conditions of the subgrade. Its strength is far greater than that of shotcrete, but it suffers from a drawback that construction and expansion joints must be incorporated in the pour, both of which are subject to leaks. Water-stops are used for this purpose, but they are expensive and might not be chemically compatible with the leachate that will eventually develop in the landfill. Further, the paving of side slopes, with angles greater than 30 to the horizontal presents major construction problems when using concrete and the material's high cost greatly limits its use (Koerner, 1985).

2.6.3 Soil Cement Liners

Soil cement liners are meant to duplicate the qualities of concrete as closely as possible, but more cheaply by using on site materials and. construction methods. If an ample supply of stone and sand is available at the site, cement can be added to the blended aggregates and mixed in place using road graders or disk harrows, water can be added, and then it all can be graded and compacted. Soil cement that has few fines (less than 20% silts and clays) in the aggregate but sufficient cement (varying from 3 to 12 percent by weight) can perform adequately as a liner. A major problem, however, is the final homogeneity of the material, i.e., the very real danger of its having soft areas that contains insufficient cement. This is avoided by using a batch process to make it, in which aggregates are mixed, then blended with the cement in exact proportions and water added at the plant or after the dry mix is spread on the prepared subgrade. This, of Course, closely resembles the way concrete liners are installed, and costs obviously increase proportionately (Koerner, 1985).

2.6.4 Bituminous Concrete (Asphalt)

Centrally mixed bituminous concrete, or asphalt, has been used often for landfill liners. The technology follows asphalt pavement procedures as used in highway engineering and construction, but places greater emphasis on permeability considerations. Compacted thickness range from 1.5 to 6.0 in, and the material is often placed in more than one layer. The uppermost layer should be similar to a road topping, with the aggregate being finer than 3/8-in-sized material. Base layers can use large-sized aggregate, depending upon their thicknesses. Asphalt of 40 to 70 penetration grade are usually used. The amount of asphalt used depends upon the location of the lining. Material having high asphalt content is generally avoided on slopes in warm climates. The viscous nature of the material causes it to creep in such cases. Compaction is critical in limiting seepage losses and values near 100 percent Marshall Stability should be targeted for.

While many liners have been (and will continue to be) made from bituminous products (their cost is very competitive), there are several areas of concern connected with their use. A major consideration is the chemical compatibility between the landfill contents and the bituminous material used in the liner. Of lesser concern, but still realistically problematic are the accelerated aging of bituminous liners and the weed growth that can occur through them, producing preferential, seepage, paths for leachate and other landfill contents (Koerner, 1985).

2.6.5 Bituminous Panels

Beginning in the early 1950's, 1/2-in-thick asphalt panels were developed for lining water reservoirs. These relatively thin (in comparison with concrete or bituminous concrete and other major materials in use at that time) liners had low strength, high deformation characteristics, but with a good sub-base could serve as an effective seepage control system. A number of joint systems became available (most of which were heat welded using lap joints, or using butt joints with all added strip covering the ends to be joined), but the panels were heavy and awkward to handle. Later 1/4-in-panels were developed; these were placed on top of one another in a staggered pattern and bonded together with a cold-applied liquid adhesive. Asphalt panels can follow ground contours to concrete surfaces of appurtenant structures very well (Koerner, 1985).

2.7 Flexible Liners (Synthetic Materials)

The use of flexible liners or geomembranes made from synthetic materials (plastics, and rubber) began with the introduction of polyvinyl chloride (PVC) and butyl rubber and has emerged over the past 15 to 20 years as an important method used in some very impressive projects. The main catalyst for use of geomembranes seems to be governmental regulations pertaining to environmental safety and the cost effectiveness. It has some serious problems, i.e.; delamination can arise, proper seams can be difficult to make, and a wicking action by the scrim can occur unless the ends of the sheets are properly sealed (Koerner, 1985).

2.8 Flexible Liners (Natural Materials)

2.8.1 Chemically Treated Soils

The addition of chemicals to soils can drastically alter their behavior. Chemical grouting, besides adding strength and reducing the compressibility of soils, also decreases their permeability. In general, the process is only cost-effective when zero or negative cost chemicals are used.

Soils can also be treated chemically by spraying. Asphalt emulsions can, be used, as can chemicals like polyvinyl alcohol. These materials either infiltrate into the voids of a soil or form a crust on its surface, thereby blocking off the voids. While the idea of chemically treating soil is technically sound, the practical construction problems of doing so are usually formidable. In particular, it is difficult to achieve uniformity of treatment; there is no guarantee against irregularly treated zones having high permeabilities. Furthermore, how permanent the various types of chemical treatments will prove to be, has not been documented (Koerner, 1985).

2.8.2 Bentonite Clays

These are Sodium montmorillonite clays, which are suitable for the purpose of lining due to very fine nature of the particles and very low permeability. The use of clay lining in industrial and municipal solid waste landfills has progressed to the point where many companies are mining, processing and supplying clays for special purpose (Koerner, 1985).

2.8.3 Compacted Clay Liners

Salient aspects of "Compacted Clay Liners" deduced from the papers by D.E.Daniel(1987) and J.C.Evens(1991) are summarized below:

a. Compacted clay liners are constructed from naturally occurring clay or mixtures of natural or commercially processed clays and other soils. A common requirement is that the clay contain a minimum percentage of fine-grained material (typically a minimum of 20 to 50 % passing the ASTM No. 200 sieve), have a plasticity index that is at least a certain value (usually 10 to 20%) and have a hydraulic conductivity below a specified value (typically 10^{-9} to 10^{-10} m/s) Daniel, 1987).

b. Clay liners are constructed by spreading clay into lifts, adding appropriate water to the soil if required and then compacting a lift. The un-compacted thickness of lifts can be as little as 100 mm or as much as 300 mm; thicknesses of 150-250 mm are commonly used. The clay is normally compacted at a water content that is equal to or slightly greater than optimum water content determined from a standard proctor compaction test (Daniel, 1987).

c. Clay is typically excavated from a borrow pit and then is either stockpiled or is taken directly to the location where the liner is to be constructed. If the clay in the borrow area has too high a water content, the clay is spread over the ground surface and dried down to the desired water content. If the clay is too dry, it may be moistened at the time a lift is spread or it may be pre-moistened by spreading the clay over the ground surface and spraying water on it. Premoistening has the advantage that the clay can hydrate for a period of time, which helps to distribute moisture uniformly. Without hydration, the exterior portion of clods of clay are moist, but the interior may be relatively dry (Daniel, 1987).

d. After clay has been spread to form an uncompacted lift, attempts may or may not be made to mix the clay or to break down large clods of clay. Discs, rototillers, and other devices are used for such purposes. The engineer should pay very close attention to clod size; large clods are difficult to break down and may lead to high hydraulic conductivity (Daniel, 1987).

e. Several types of compaction equipment are used, but sheep-foot rollers are probably the best for compacting clay liners due to the remolding action that is achieved. The feet should have a length that is slightly greater than the thickness of an uncompacted lift to help achieve good bonding between lifts and to ensure that clods of clay will be remolded during compaction (Daniel, 1987).

f. A number of factors 20 into determining the thickness of a liner, including cost, hydraulic conductivity of the compacted clay, and the nature of the waste to be contained (Daniel, 1987).

g. For large projects, a test section is prepared. The test should include a section of the clay liner, preferably constructed to full thickness, and covering an area of roughly 10 to 20 m. Dykes are constructed around the clay liner so that 1 to 3 m water can be pounded, on the liner. The liner is underlain by a freely-draining layer (sand, filter fabric, or some other suitable material) which in turn is underlain by an impermeable flexible membrane. Seepage that percolates through the liner is collected by a gravity drainage system connected to the under lain and the rate of seepage is measured and used to compute the hydraulic conductivity of the test section (Daniel, 1987).

h. The most cost-effective covers can usually be completed utilizing native clay materials from local borrow sources. In many areas, local sources are available to provide compatible clays of relatively low hydraulic conductivity. The recommended minimum thickness of clay barrier layer is generally 2 ft. The top few inches of a clay cap is not as well compacted as the remainder owing to lack of confining pressure at the surface. Further, in the long term, it is difficult to maintain the clay density in the top few inches, owing to potential desiccation cracking and frost action. The bottom of the clay barrier layer may be mixed with the subgrade material during installation (Evens, I991).

i. Compacted clay covers must be protected from erosion due to rain water, cracking due to drying, differential subgrade movement, penetration by deep tap roots of vegetation, rutting due to traffic. Conventional hydrologic studies are required to determine erosion potential and prevent erosion of the cover materials (Evens, 1991).

j. A clay cover can generally be placed on a slope that is 2 horizontal to I vertical or flatter. Flatter slopes more readily permit compaction and reduce the risk of local instability. A geotechnical analysis of slope stability is required to assess the potential for instability. The thickness of the clay on the side slopes may be slightly less than the thickness on top of the area, since the larger side slopes gradients enhance run-off, greatly reducing the time available for precipitation to percolate downward into the waste containment area (Evens, 1991). Despite the potential discrepancy between field and laboratory values of hydraulic conductivity, clay liners and covers are frequently used as components in hazardous waste control systems. Their specific advantages include low cost and the ability to attenuate, that is, adsorb contaminants that may migrate through the barrier. Further calculations have shown that clay barriers, if properly constructed may be more efficient than geomembranes, owing to mass transport through synthetic liners by diffusion (Evens, 1991).

2.9 Key Construction Factors

There are few important technical factors that might affect how well a clay landfill liner will resist the passage of water. These factors were evolved by McClelland Consultants (Southwest), Inc, (USA) after having constructed a test liner. B.R.Elsbury and G.A.Sraders (1989) summarized these factors as:

2.9.1 Basic Compaction Objectives

Two basic objectives must be met if the clay liner is desired to be sufficiently impermeable and these include the destruction of clods and proper inter lift bonding.

2.9.2 Choices to Achieve Compaction Objectives

a. Moisture content of the soil should behigh enough that the roller being used can readily remold the clods into a new homogeneous mass.

b. The lift should be thin enough that the roller feet should penetrate sufficiently and thoroughly remold the loose soil in the top of preceding lift.

- **c.** The size of clods contributes the problems with the liner. Large clods are liable to survive the compaction process and would result in increased permeability.
- **d.** Type of roller would also affect the permeability of the clay liner. The roller should be heavy enough to remold the soil and produce higher density. The roller should make enough passes to ensure that all of the lift is remolded.
- **e.** Soil density and saturation has less of impact on the liner's permeability.

2.9.3 Other Considerations

- **a.** Soil preparation
- **b.** Construction quality assurance

2.10 Influence on Hydraulic Conductivity of Compacted Clay

Mitchell, et al. (1965) reported hydraulic conductivity tests on soil compacted in the laboratory. The experiments isolated some of the fundamental variables that influence the hydraulic conductivity of compacted soil. The critical variables identified by Mitchell, et al. include molding water content, method of compaction, compactive effort and degree of saturation.

Several investigations of in-situ hydraulic conductivity of compacted soils have been performed. An important finding was that laboratory measurements of hydraulic conductivity sometimes underestimate the in-situ hydraulic conductivity and was suggested that discrepancies between the hydraulic conductivity of laboratory and field compacted soil may be partly due to the differences in clod size between the field and laboratory. Craig H. Benson and David E. Daniel (1990) carried out research to study the influence of clods on Hydraulic Conductivity of compacted clays. Important findings or salient conclusions are as follows:-

2.10.1 Influence of Clod Size

Clod size has a large influence upon the hydraulic conductivity of the compacted soils. Samples of soil with initially small clods and compacted dry of optimum had hydraulic conductivities that up to six orders of magnitude lower than samples compacted from material with initially large clods. The hydraulic conductivity of specimens compacted wet of optimum, however, did not depend on Clod size. It appeared that relatively high moisture content rendered clods soft and compressible and the size of the soft, wet clod, was unimportant with regard to hydraulic conductivity after compaction. This is because the soft, easily remolded clods could be adequately deformed and compressed into a relatively homogeneous mass of low hydraulic conductivity soil regardless of the size of the clods.

2.10.2 Particle Orientation versus Clod Structure

Examination of the compacted soils indicated that the fate of clods and inter-clod voids controlled the hydraulic conductivity. The driest specimens compacted with standard Proctor effort looked more like granular material than clay soil; Standard Proctor effort was not sufficient to press the dry, hard clods together and eliminate large inter-clod voids. These specimens had large hydraulic conductivity. Compaction of soil at the same water content but with modified Proctor effort resulted in greater deformation of clods reduction of large voids and lower hydraulic conductivity. The energy imparted by modified Proctor effort was sufficient to press the dry, hard clods together. Furthermore, soils compacted wet of optimum by standard or modified Proctor effort showed no evidence of remnant clods or inter-clod pores. The hydraulic conductivity of all specimens compacted wet of optimum was very low. Clearly, the fate of clods and inter-clod pores controlled the hydraulic conductivity of the compacted specimens.

2.10.3 Design of Laboratory Compaction

To achieve low hydraulic conductivity in soils that form clods large inter-clod voids must be eliminated during compaction. Large inter-clod -pores can be minimized and the effects of clods can be overcome in *highly plastic soils* by:

a. Compacting soil at moisture content that is large enough to soften the clods so that they can be remolded by the compaction equipment.

b. Using a sufficiently large compactive energy to destroy even relatively dry, hard clods.

2.11 Influence of other Factors on Hydraulic Conductivity

Research was carried out by Stephen -S. Boynton and David E. Daniel (I984) to evaluate the effect of type of permeameter, direction of water flow relative to stratification from lifts of clay, diameter of test specimen, storage time, and desiccation cracking on Hydraulic conductivity of compacted clays. Reasons for the research with salient conclusions are as under:-

2.11.1 Type of Permeameter

Permeameters are of two general types:

- Rigid-wall permeameters
- Flexible-wall (or triaxial-type) permeameters

There has been considerable debate over which type of permeameter should be used to measure permeability (k) of compacted clay. Proponents of rigid-wall point to simplicity of equipment and ease of testing compacted specimens directly in the compaction mould. Proponents of flexible-wall cells argue that leakage may occur along the contact between the soil and the rigid-wall and confinements of the test specimen with a flexible membrane is required to minimize spurious side-wall leakage.

While the test results did vary from one permeameter to another, the type of permeameter did not have a large effect on the measured hydraulic conductivity. Differences in 'k' were substantially less than one order of magnitude. The differences between conductivities measured with different permeameters may be attributed to a variety of differences in the equipment, test procedures and applied stresses. It should be noted that great care was taken in mixing the soil uniformly, breaking down large clods of clay during mixing and compacting reasonably homogeneous test specimens. When testing a natural soil from the field, one often finds large clods of clay which are not readily broken down during the compaction process. The result is a compacted specimen with very rough and irregular sidewalls. There may be an opportunity for sidewall leakage to occur when testing such soils in rigid-wall cells.

It was found that the hydraulic conductivity (k) of compacted specimens of kaolinite and fire clay varied only slightly when different types of permeameters were used to measure 'k'. There was no evidence of sidewall leakage in the tests with rigid-wall permeameters and no basis for concluding that one type of permeameter yielded better measurements than the others. The tendency for compacted soils to swell when moistened probably helps to minimize sidewall leakage in compaction-mould permeameters.

2.11.2 Direction of Flow

Most engineers assume that compacted clay has a higher hydraulic conductivity for flow parallel to the lifts of clay compared to flow perpendicular to lifts. Hydraulic anisotropy is assumed to exist because of effects of soil fabric and effects of imperfect bonding between lifts of clay. However a search of the literature did not show any data on hydraulic anisotropy in compacted clay. Some investigators have suggested that compacted clays are flocculated when compacted dry of optimum. If such is the case then it would seem that the degree of hydraulic anisotropy should be nil for the samples compacted dry of optimum (when the particles are oriented randomly), but might be significant for samples compacted wet of optimum (when particles are aligned parallel to one another).

It has been suggested that the horizontal conductivity of compacted clay may be many times larger than the vertical conductivity because of the effects of soil fabric or flow along planes between lifts. For laboratory-compacted slabs of fire clay with good bonding between lifts, the vertical and horizontal conductivities were found to be essentially identical in the tests performed for the investigation. Anisotropy in the field would likely to be the result of poor bonding between lifts or the presence of more permeable material in some of the lifts. Such effects can only be studied with relatively large- scale field tests.

2.11.3 Size of Test Specimen

It is generally recognized that hydraulic defects such as cracks, fissures, and sand lenses control the hydraulic conductivity of fine-grained soils. Most engineers would assume that the measured ‘k’ would tend to increase with increasing sample diameter because defects have a statistically better chance of being present in a large sample compared to a small one. In most engineering laboratories, hydraulic conductivity is measured on compacted specimens with diameters in the range of 1.5 to 6 in. (3.8 to 15.2 cm). One wonders whether ‘k’ is really very sensitive to sample diameter, for example, whether it makes some remarkable difference if the sample diameter is 1.5 in. (3.8 cm) or 6 in. (15.2 cm).

The purpose of testing specimens with different diameter was to determine if 'k' tends to increase with increasing diameter of the specimen. Fire clay was compacted into slabs at three different remolding water contents, and then cylindrical specimens of various diameters were trimmed and set up in flexible-wall permeameters. Optimum water content for the fire clay compacted into slabs was 18%. The effect of sample diameter depended upon compaction water content. The sensitivity of 'k' to sample diameter was greatest for samples compacted well dry of optimum water content. For samples compacted slightly dry of optimum, ‘k’ was essentially independent of sample diameter. Although the sample compacted wet of optimum showed a tendency of increasing ‘k’ with increasing sample diameter, the scatter in the data for samples compacted wet of optimum was greater than for other water contents. However, regardless of the compaction water content, increase in hydraulic conductivity with increase in sample size was not very significant. A suggested practice is to test a sample with a diameter that is several times larger than the largest clod of clay that is observed when the soil is compacted so that representative inter-clod flow will take place.

2.11.4 Age of Specimen

Mitchell et al., (1965) has found that ‘k’ of compacted clay tends to increase with the time a sample is stored before permeation is initiated. If this pattern holds for all soils, important questions are raised about the long-term integrity of compacted clay barriers. The purpose of testing samples by Boynton and Daniel (1984) with various storage times was to determine if fire clay exhibited the same tendency for a time-dependent increase in ‘k’ as observed by Mitchell et al., (1965) on two other type of soils. Four samples of fire clay were compacted to identical water content and densities. The soil for all four samples came from the same batch of mixed material and all four samples were prepared on the same day by same compaction equipment. The compaction water content was approximately 16.5%, which is about 2% dry of optimum. Three of compacted specimens were placed in a moist room in plastic bags after being extruded from the compaction mould. The fourth was set up immediately in a flexible-wall cell, and was permeated. After various periods of storage, the other specimens were tested in the same manner.

The hydraulic conductivities of the four test specimens showed the function of time between compaction and initiation of permeation. There was no clear trend in the data, although the first three tests indicated the decrease in 'k' with time. It is uncertain whether there is an error in the last test, or there is a tendency for 'k' to increase after some period of storage, or whether the values of ‘k’ are randomly scattered about some mean value. In any case, storage time did not have a major effect. This finding conflicts with data obtained by Mitchell et al., who found that ‘k’ increased by as much as an order of magnitude for storage times of 2 to 8 weeks. A review of Atterberg limits and other index properties of the soils used in the study and the soils used by Mitchell et al., did not indicate any major differences. However, the samples compacted for the study and stored for various periods were compacted dry of optimum. Mitchell et al., studied a range in compaction water content and found that storage time had more effect on the hydraulic conductivity of samples compacted wet of optimum. The tests on fire clay showed that not all the compacted clays tend to undergo a significant time-dependant increase in hydraulic conductivity during storage.

2.11.5 Desiccation Cracking

It is known that desiccation of compacted clay can result in the formation of tension cracks. The rate at which cracks propagate had not been documented, the effect of cracks on ‘k’ had not measured, the depth to which cracks may extend had not been determined, and the tendency for the cracks to close when the soil is moistened had not been evaluated. Study was carried out by Boynton and Daniel (1984) to determine if desiccation would seriously impair the ability of a compacted clay to serve as a hydraulic barrier. Desiccation cracks fully penetrated the slabs in approximately 4 hours. The cracks grew larger as time passed and seemed to be stabilized after 2 hours of desiccation. Open cracks as wide as 1 mm typically penetrated the slab by the end of 8 hours.

Specimens that were desiccated for 3 days were removed from the moulds, and 4-in. (10cm) diameter specimens were carefully trimmed from the portions of the slabs that contained desiccation cracks. The specimens were then set up in a flexible wall permeameter and were permeated at effective stresses of 2, 4, 8 and 15 psi. 'k' decreased markedly as the effective stress increased. It appeared that confining pressures in the range of 4 to 8 psi (28 to 56 KPa) were sufficient to begin closing the cracks, and confining pressures in excess of 8 psi (56 KPa) closed the cracks and led to a much reduced ‘k'.

It is concluded that desiccation cracks can penetrate compacted clay to a depth of several inches in just a few hours. When the soil is moistened, the cracks tend to close, but the hydraulic conductivity is not as low as for un-desiccated specimens, unless a suitably large confining pressure is applied to force the cracks to close.

2.12 Short Term and Long Term Permeabilities of Contaminated Clays

The influence of chemicals on the hydraulic conductivity of clayey soils is a major concern in determining the long-term performance of clay liners for waste impoundment. It has been reported that extremely large values of hydraulic conductivity were observed when liquid hydrocarbons dominated the fluid phase. The hydraulic conductivities of contaminated clays approached values of the order 10^{-4} cm/s, which were more characteristic of fine sand than clay. If the hydraulic conductivity values are increased during the seepage of chemicals, the contaminants can move with greater ease and reach critical ground-water models assuming no change in hydraulic conductivity.

When soils are contaminated with chemicals, physico-chemical interactions may occur between the soils and the chemicals. Due to these interactions, the physical properties of soil may change drastically. This drastic change occurs during the passage of chemical contaminants. Yet, it is controversial whether there is an improvement or deterioration of the hydraulic conductivity of soils due to chemical contamination. For a conservative estimate of the extent of the groundwater contamination, adverse values of hydraulic conductivity have to be used in the hydrogeological computations (Meegoda and Rajapakse,(1991).

2.13 Shrinkage of Soil Samples with Varying Clay Concentrations

During Shrinkage water moves from the inside of soil blocks to outside surfaces, from where it evaporates. Evaporation increases the curvature of the water menisci in pores at the soil surface, creating a force moving water to the surface and consequently decreasing the volume of the block, Forces of water retention associated with the clay particles oppose this water loss and therefore, oppose shrinkage. If the soil particles are in contact with each other, friction forces also oppose shrinkage (DeJong and Warkentin, 1965).

The total amount of shrinkage depends upon the proportion of clay, type of clay mineral, exchangeable cations, orientation of clay particles, and degree of aggregation of the soil. These factors determine both the swelling on wetting and moisture content at the shrinkage limit and therefore the amount of shrinkage also With an area of soil with uniform clay minerals, variation in shrinkage is due largely to the variation in the proportion of clay.

Research was carried out, by E.DeJong and B.P. Warkentin (1965) in this regard and they concluded that total shrinkage and moisture content at the shrinkage limit increases linearly with the percentage of clay as long as the clay forms a continuous matrix. At lower clay concentration, shrinkage decreases rapidly.

2.14 Volume Change Behavior of Desiccated Soils

The understanding and the prediction of the volume changes occurring in soils due to changes in stress are of vital importance in geotechnical engineering. While the importance of the shear strength behavior of any soil cannot be denied, more often than not its compressibility governs designs in civil engineering practice. Arid and semi-arid regions cover large parts of the earth's land surface. Soils in these regions are described as desiccated soils (Sridharan and Allam, 1981).

A study on volume change behavior of such soils was carried out by A. Sridharan and M. M. Allam(1981) to analyze theoretical considerations. They concluded:-

a. The two mechanisms governing the volume change behavior of pure clays (viz. Mechanism I, wherein the compressibility of a clay is primarily controlled by the shearing resistance at the near contact points and volume changes occur by shearing displacements or sliding between particles and Mechanism II, in which compressibility is primarily governed by the long range osmotic repulsive forces) also govern that of natural desiccated saturated soils.

b. Desiccation bonds present in these soils, by imparting additional intrinsic effective stress to them (and thus shearing resistance at particle contacts) and by forming crumbs, affect the volume change behavior of such soils and prevent the mechanisms independently governing their volume change behavior.

c. For desiccated soils containing nonexpanding lattice-structured clay minerals in their clay fraction, the desiccation bonds increase their resistance to compression by increasing the shearing resistance at particle contacts.

d. For desiccated soils containing expanding lattice structured clay minerals in their clay fraction, the desiccation bond, by creating crumbs with the resulting reduction in effective specific-surface areas, prevent the physicochemical mechanisms (Mechanism II) from solely deciding their responses to loading and unloading. If bonding is less, Mechanism II governs the behavior. If bonding is large, the soil is reduced to one chiefly composed of non-expansive clays. Mechanism I will then govern the compression behavior until bonds are disrupted.

This means that depending on the degree of bonding, either Mechanism II alone or Mechanisms I and II successively will govern the compression behavior, while the rebound behavior is governed primarily by Mechanism II.

2.15 Desiccation - Induced Cracking

Several studies of desiccation-induced cracking of low-hydraulic-conductivity, compacted soil barriers have been performed. Kleppe and Olson (1985) investigated desiccation of two highly plastic clays (Taylor marl and Elgin fire clay) and sand / bentonite mixtures. Kleppe and Olson compacted cylindrical specimens and found that the volumetric shrinkage produced by desiccation was linearly proportional to molding water content but was insensitive to dry density. Cylindrical specimens shrank into smaller cylinders but did not crack significantly. To study cracking Kleppe and Olson compacted slabs of clay and constrained the slabs along their edges during drying. Major cracking, which Kleppe and Olson defined as development of cracks greater than 10 mm wide, occurred when volumetric shrinkage strains in cylindrical specimens compacted to the same water content and dry densities were greater than 4%. Kleppe and Olson also found the shrinkage strains were far less in clayey sands than in soils with little sand. For example, at the same molding water content, shrinkage strains in mixtures of sandand bentonite containing 88%, 50%, 25%, and 0% sand were 4%, 11%, 14%, and 18% respectively.

DeJong and Warkentin (1965) mixed Leda clay at the liquid limit with small glass beads in varying percentages, then measured the linear shrinkage strains that, occurred in bars. Practically no shrinkage occurred when at least 70% of the material was glass beads, shrinkage was minimal.

Boynton and Daniel (1985) described research in which 64 mm (2.5 in.) thick slabs of soil were compacted and desiccated. Cracks, that fully penetrated the compacted slabs, developed in less than 24 h. When undisturbed specimens trimmed from the slabs were back-pressure saturated and permeated the hydraulic conductivity was sensitive to the effective confining stress which was sequentially increased after each permeation stage. At low stress re-wetting the soil did not result in full self-healing. These data suggest that desiccation cracks are of particular concern for final cover systems where the overburden stress on the compacted soil liner is low. For compacted clay buried beneath substantial overburden, the compressive stress from the overburden help close preexisting desiccation cracks, and prevent the development of new ones provided the foundation is properly prepared.

Summarily, the studies have demonstrated that compacted clay cracks, when dried. Covered systems are especially vulnerable due to the potential for drying of soils located near the surface and the low compressive stress acting on the liner. Water content and the percent of coarse material (e.g., sand) seem to be the most important soil characteristic that affect desiccation cracking; Sandy soils compacted at the lowest possible water content have the lowest potential for shrinkage cracking upon drying. Protection of clay from drying is also critical; a thin layer (less than or equal to 450 mm,[18 in.] of cover soil alone is probably not adequate (Daniel and Wu, 1992).

2.16 Structure and Strength Characteristics of Compacted Clays

The physical properties of a compacted soil depend largely on the soil material, moisture and density. In addition, the structure and the conditions of compaction that produced it were important in cohesive soils. When the cohesive soil is compacted at moisture contents less than optimum, an aggregated structure is formed. When the soil is compacted at high moisture contents, a dispersed structure is formed, with the flaky particles aligned in parallel (Sowers, 1979).

In cohesive soils, compaction occurs by both reorientation and distortion of the grains and their adsorbed layers. This is achieved by a force great enough to overcome the cohesive resistance or inter-particle forces. Vibration and shock are of little help, although they provide a dynamic force in addition to the static, this is largely offset by the increased cohesive resistance that accompanies dynamic loading. For greatest efficiency, the compaction force must be high enough to distort the particles and shift the individual grains but not great enough to shear the mass. In a cohesionless soil the strength is dependent on confinement. This can be provided by a wide area of load application. In cohesive soils the strength is dependent on void ratio, moisture and largely independent of the confinement (Sowers, 1979).

Research studies were carried out by H.B.Seed and C.K.Chan(1959) and L.Barden and G.R.Sides(1969) to demonstrate and explain the significance of various factors in relation to structure of clays in the ‘as compacted’ conditions. Salient conclusions of their research works are as under:-

2.16.1 Effect of Structure on Soil Properties

a. Shrinkage

Samples compacted dry of optimum and having essentially flocculated structures exhibited considerably less shrinkage than samples of same composition compacted wet of optimum (Seed and Chan, 1959).

b. Swelling

Samples compacted dry of optimum exhibit higher swelling characteristics and swell potential than those compacted wet of optimum (Seed and Chan, 1959).

c. Undrained Strength

At large strains (about 20%), if the specimen has not yet reached its maximum resistance at that point, structure has little or no influence on soil strength. At lower strains, the structure has a pronounced influence on the strength of compacted soils, with dispersed arrangements producing much lower strengths than flocculated arrangements (Seed and Chan, 1959).

d. Permeability

Permeability is complex function of the parameters structure and saturation. For the same value of saturation, the structure formed at low molding water content results in higher value of permeability. At high degree of saturation, permeability can fall by many orders in the region of optimum water content, confirming a marked change in structure dry and wet of optimum. At a given structure, permeability is dominated by the degree of saturation and can change by many orders as the saturation increases over the range of 60% to 100%. Test results have indicated that coefficient of permeability of soil samples compacted wet of optimum was two to three orders of magnitude less than coefficient of permeability of similar soil samples compacted dry of optimum (Barden and Sides, 1969).

2.16.2 Effect of Method of Compaction on Soil Strength and Structure

a. For samples prepared dry of optimum, all methods of compaction produce no appreciable shear deformations. Consequently, flocculated structures, essentially, flocculated structure, which are sufficiently similar shows that the method of compaction has no appreciable effect on the soil strength characteristics.

b. For samples of the same composition prepared wet of optimum. Kneading compaction causes the largest shear strain during compaction and therefore the highest degree of dispersions, the highest pore water pressures and the lowest strengths at low strains and the highest shrinkage. Impact compaction causes slight shear strain during compaction and consequently the degree of dispersion is not quite so great as for kneading compaction, the strengths at low strains are slightly higher and the shrinkage is slightly less. Static compaction causes little shear strain during compaction, resulting in a relatively flocculated structure, the lowest pore-water pressures and highest strength at low strains and least shrinkage. Vibratory compaction covers the entire area of a sample as used in these tests, should give little chance for shear strain to occur in the samples and thus produce the same structure as is obtained by static compaction. However, it appears that the vibrations enable particles to reorient to a more dispersed arrangement than is possible with static compaction, resulting in somewhat lower strengths at low strains and higher shrinkage.

c. Differences in structure and strength resulting from impact and kneading methods are apparently small.

d. The data also indicate that impact and kneading methods of compaction are apparently sufficiently similar that samples prepared wet of optimum to the same density and water content by either method may show the slightly higher strengths at low strains, depending on the magnitude of the tamping pressures and the hammer blows used for preparation of the samples (Seed and Chan, 1959).

2.17 Pore Sizes and Strength of Compacted Clay

The load deformation and water transmission characteristics of compacted clays are recognized as being dependent upon the packing and arrangements of the particulate units. Measured values of these characteristics are affected by the bulk porosity. A particular unit weight can be achieved with a variety of particulate arrangements, so that correlation with soil characteristics should improve when soil structure or fabric can also be taken into account. It is logical to approach this aspect soil fabric by the measurement of pore size distribution that result from application of different levels of moisture content, compactive effort and compaction type. Study in this regard was carried out by S.Ahmed, C.W. Lovell and S.Diamond (1973) which concluded:-

a. For three different methods of compaction carried out in such a way as to follow a common moisture-density curve, the pore size distribution of a compacted Illite clay after freeze drying has been found to depend most strongly on the compaction moisture content.

b. The method of compaction affected the pore size distribution of the compacted Illite clay very little, at least under the imposed constraint that samples were compacted to the same moisture-unit weight conditions by each of the different methods of compaction.

c. Stress-strain curves measured in unconfined compression for the compacted Illite clay varied systematically with molding water content. For samples compacted and tested dry of Proctor optimum, brittle failures occurred at low axial strains. Quasi-brittle failures at moderate strains were observed for samples compacted at Proctor optimum. Gradual shear failure at high strains occurred for the wet side compaction samples. The highest peak strength was recorded for samples compacted at the standard Proctor optimum. Samples compacted by kneading compaction on the wet side of Proctor optimum showed the lowest strength. On the average, the peak strength of samples compacted on the wet side of Proctor optimum was about two third that for samples compacted on the dry side of Proctor optimum. These compactions were made at equal unit weights.

2.18 Water Content - Density Criteria for Compacted Soil Liners

2.18.1 Traditional Approach

Currently, design engineers usually require that soil liners be compacted within a specified range of water content and to a minimum dry unit weight. The “acceptable zone” shown in Figure 2.1 represents the zone of acceptable water content / dry unit weight combinations based on typical current practice. The designer will usually require that the dry unit weight 'γ_d' of the compacted soil be greater than or equal to a percentage 'P' of the maximum dry unit weight ($\gamma_{d,max}$) from a laboratory compaction test:

$$\gamma_d \;>\; (P / 100)\, \gamma_{d,max} \quad \ldots\ldots\ldots \quad 2.1$$

'P' is usually 95% of'$\gamma_{d,max}$' from Standard Proctor compaction (ASTM D – 698) or 90% of '$\gamma_{d,max}$' from Modified Proctor compaction (ASTM D - 1557). The range of acceptable water content varies with the characteristic of the soil, but for clay liners and covers might typically be about zero to four percentage points wet of standard or modified Proctor optimum.

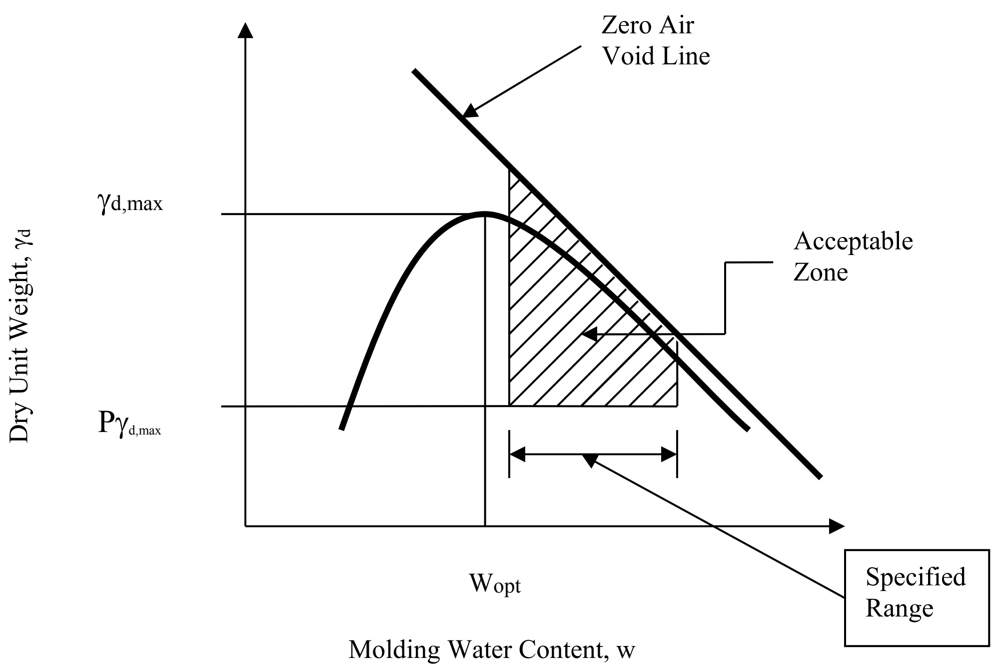


Figure 2.1 :Traditional Method for Specification of Acceptable Water Contents and Dry Unit Weights for Compacted Clay Liners (After Daniel and Benson, 1989)

The shape of the acceptable zone has been evolved empirically from construction practices applied to roadway bases, structural fills and earthen dams. The specification is based primarily upon the need to achieve minimum'γ_d' for adequate strength and limited compressibility. Soil liners are compacted wet of optimum because wet-side compaction minimizes hydraulic conductivity (Daniel and Benson, 1989).

2.18.2 Modified Recommended Approach

The recommended procedure involves establishing w-γ_d ranges needed to achieve the required hydraulic conductivity and then modifying these ranges to account for other factors besides 'k'. This approach recommended by D.E. Daniel and C.H. Benson (1989) involves following steps:

a. Compact soil in the laboratory with modified, standard and reduced Proctor compaction procedures to develop compaction curves. Approximately five or six different specimens should be compacted with each effort. Other compaction procedures can be used if they better simulate field compaction and span the range of compactive effort expected in the field.

b. The compacted soils should be permeated to determine the hydraulic conductivity of each compacted specimen. Care should, be taken to ensure that permeation procedures are correct, with important details such as degree of saturation and effective confining stress carefully selected. The measured hydraulic conductivities should be plotted as a function of molding water content.

c. The dry unit weight - water content points should be re-plotted with different symbols used to represent compacted specimen that had hydraulic conductivities less than or equal to the maximum acceptable value. The acceptable zone should be drawn to encompass the data points representing test result meeting or exceeding the design criteria. Some judgment may be necessary in constructing the acceptable zone.

d. The acceptable zone should be on other considerations, e.g., shear strength, interfacial friction with an overlying geomembrane shrink / swell considerations or local practice. For example, if shear strength is of concern, a limit on the water content and / or dry unit weight should be specified to ensure that excessively weak soils are not produced. The same procedure can be applied to other factors (e.g., shrink / swell potential) relevant to any particular project.

2.19 Cement Stabilization

Cement Stabilization of soil involves mixing of pulverized soil, cement and water and compacting this mix to high density, which renders the material resistant to various physical, thermal and chemical stresses. Depending on the type of soil and size of the aggregate used there are various different products of cement Stabilization. As the cement hydrates, a gel is formed that upon hardening forms a cellular matrix that encapsulates the soil particles or forms strong bridges between aggregates, thus producing a hard, durable structural material. If size of the soil particles is smaller than that of the cement, the soil particles surround the cement particle and weaker bonds are formed. When properly mixed and constructed, a cement stabilized soil system generally performs well for the intended purpose, even when exposed to wetting-drying or freeze-thawing cycles.

Cements which react with water to form strongly bonded systems are called hydraulic cements. The common hydraulic cements are mixtures of calcium silicates and aluminates and include the portland, natural, slag and alumina cements (Winterkorn and Pamukcu, 1991).

2.19.1 Portland Cement

The most commonly used cement in stabilization is Portland cement, which is finely powdered hydraulic cement, essentially consisting of hydraulic calcium silicates (specifications in AASHTO M85 and ASTM C150). The particle size of the portland cement ranges from 0.5 to 80 μm, with major part of it passing No. 200 sieve and the specific gravity of the particles ranges from 3.12 to 3.20. The major compounds in portland cement are tricalcium silicate ($3CaO.SiO_2$;C_3S), bicalciumsilicate ($2CaO.SiO_2$;C_2S), tricalciumaluminate ($3CaO.Al_2O_3$;C_3A) and tetracalcium aluminoferrite($4CaO.Al_2O_3.Fe_2O_3$;C_4AF). These compounds react with water to form very stable hydrated silicates aluminates and also calcium hydroxide {$Ca(OH)_2$} (Winterkorn and Pamukcu, 1991).

The ASTM C150 specification describes five types of portland cements, in which Type-I is for standard use. Type-II cement contains reduced quantities of C_3A and C_3S to provide resistance for sulphate attack and lower heat of hydration. Type-III is high early strength cement with increased quantities of C_3A and C_3S, which also produces high heat of hydration and thus greater drying shrinkage. Type-IV contains severely limited quantities of C_3A and C_3S, which renders a low heat of hydration cement. Type-V is designed for maximum sulphate attack resistance with strict limitations on C_3A content.

2.19.2 Type of Cement Stabilization

Basically there are three types of cement-stabilized systems as described by Winterkorn and Pamukcu (1991):

2.19.2.1 Soil - Cement

Soil – Cement contains sufficient cement to produce a hard and durable construction material and only enough moisture to satisfy the hydration requirements of the cement and the soil, and also to provide sufficient lubrication for the compaction of the mixture to a high density. The resulting material has well-defined resistance to weathering. Standard laboratory tests have been developed to judge the performance of soil-cement, such as strength, durability, water susceptibility, and. frost resistance. Soil-cement is commonly used for stabilization of road bases of flexible and rigid pavements, sub-bases, embankment slopes, earth dam cores, reservoir linings, building foundations, trenches, and for frost protection and reinforcement of load-bearing layers.

2.19.2.2 Cement - Modified Soil

It is a hardened or semi-hardened mixture of soil and cement. Relatively small quantities of portland cement are added to a granular or silty clay soil to improve certain physical and chemical properties of the soil. The intended improvements are reduction of volume change tendency and plasticity, and increasing load-bearing capacity of the soil. There is sufficient cement to interact with the silt and clay fractions and to deprive them of their water affinity, but not enough to bond all of the soil particles into a coherent system. The result is an improved soil rather than a new building material with standardized properties such as soil-cement. Cement improvement of soils is often used for erosion and frost protection, and to reduce shrinkage and expansion of foundation and base layers.

2.19.2.3 Plastic Soil-Cement

It results in a hardened product but contains, at the time of placement, sufficient water to produce a consistency similar to that of a plastering mortar. This allows it to be placed on steep or irregular areas where access of construction equipment is difficult or not possible. This material compares with soil -cement, and like soil -cement it is required to strict strength and durability conditions. It is most commonly used for lining of ditches, irrigation canals, and trenches, and for protection of such surfaces against erosion.

2.19.3 Cement –Soil Reactions

In a neat cement paste, the major hydration products are 'calcium silicate hydrates', 'calcium aluminate hydrates', and 'hydrated lime'. The first two products constitute the major cementitious components, whereas the lime is deposited as separate crystalline solid phase. With fine-grained soils, which because of the size of the primary soil particles is of the same order as and frequently much smaller than the cement particles, and the cement content is relatively low, a cement particle is much more likely to be surrounded by soil particles than by other cement particles. Thus, there will be virtually no opportunity for direct bonding between cement particles. In addition, the soil particles, because of their high specific surface and established tendency to retain substantial quantities of hydrous, non-crystalline silica and alumina on their surfaces, exert a profound buffering action on alkaline systems and are capable of reacting with or exchanging cations with great rapidity. As a consequence, the Kinetics of cementation and the nature of the reaction products formed during the cure of soil-cement may be materially different from those of cement alone, or of concrete. In soil-cement, virtually all of the reactive calcium present initially in the cement is eventually available for production of cementitious silicate gel; hence the quantity of cementitious material available for bonding in soil-cement is inherently greater than that in heat cement (Moh, 1962).

2.19.4 Water and Cement Requirements

Practical limitations on susceptibility of soil to cement stabilization derive from the water requirements during the compaction and hardening period. The system must contain enough water for hydration of cement and silt-clay constituents and for workability of the soil. In the working of the soil, water acts as an interparticle lubricant. Water used in should be relatively clean and free of harmful amounts of salts, alkalies, acids, or organic matter. Water fit to drink is satisfactory. A well-graded soil containing gravel, coarse sand, and fine sand with or without small amounts of silt or clay requires 5 percent or less cement by weight. The remaining sandy soils generally require 7 percent. Non-plastic or moderately plastic soils generally require about 10 percent and plastic clay soil requires 13 percent or more (Winterkorn and Pamukcu, 1991).

2.19.5 Shrinkage Cracking and Curing

As the hydration of cement proceeds, drying and evaporation of the surplus water results in shrinkage of the soil-cement system. The severity of the phenomenon increases with the increasing water affinity of the soil. This sets a natural limit to the types of soil that can be practically stabilized with portland cement. Limiting the plasticity of the soil used, thorough pulverization of the soil, and thorough mixing with cement has been demonstrated to produce good results with respect to control of shrinkage cracking. For cohesive soils the required cement content increases with the water affinity of the soil. Higher cement content reduces volume change tendency; however, it increases the tensile strength of the soil-cement giving the undesirable pattern of cracks with lower frequency and larger width. The density and cement content also influences the amount of shrinkage. Increased cement content and density increases the thermal coefficient of expansion of a soil-cement system, which renders it more susceptible to temperature variations. However, increased cement content reduces volume change tendencies and thus shrinkage. It also increases water absorption capacity and reduces the overall permeability of the system. Prevention of the loss of excessive amounts of water, during curing, aids in minimizing the shrinkage cracking. This is often done by covering the stabilized system with various waterproofing agents, water, or emulsified asphalts. Other materials like waterproof paper, moist straw, or soil can be satisfactory also. Curing periods vary from 7 to 14 days, extending to 28 days in some cases (Winterkorn and Pamukcu, 1991).

2.19.6 Mixing and Compaction

Depending on the size and conditions of the project, soil stabilization with portland cement, or any other stabilizer, may be achieved with various means that range from the most primitive hand tools to sophisticated single-pass machines. The basic steps in the construction of a soil-cement base as described by Winterkorn and Pamukcu (1991) are;

a. Pulverizing the soil
b. Spreading the cement and mixing
c. Addition of water
d. Compaction, rolling and finishing
e. Curing of the completed system

f. Quality control.

2.19.7 Laboratory Testing

Detailed laboratory testing procedures have been developed by the Portland cement Association and adopted by AASHTO and ASTM. Basic tests to determine the compressive strength and moisture-density relations are;

a. Test for compressive Strength of Molded Soil-Cement Cylinders, ASTM D1633.

b. Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory, ASTM D1632.

c. Test for Moisture-Density Relations of Soil-Cement Mixtures, ASTM D558.

The preparation of test specimens in the laboratory should represent, on a small scale, the steps and processes actually employed in construction. Pulverization of cohesive soil and clay clods, or homogenization of non-cohesive soils by removing the oversize particles, is the initial step in specimen preparation for laboratory testing. Determination and addition of the proper amounts of cement and water, thorough mixing, compaction at or around maximum density and optimum moisture content according to the requirements, and finally proper curing with prevention of water loss are the steps followed in specimen preparation (Winterkorn and Pamukcu, 1991).

2.20 Polyethlene

Polyethylene, commonly known as ‘polythene’ is a thermoplastic material which has linear molecular chains. It is easily stretched and is not rigid. As the chains are independent of each other, they can easily flow past each other and so the material has a relatively low melting point, no energy being needed to break the bonds between the chains. The absence of bonds between chains also means that none is broken when the material is heated. The removal of heat allows the material to revert to its initial harder state (Bolton, 1987).

2.20.1 Types of Polythene

Polythene is made in two forms i.e. low density and high density.

2.20.1.1 Low Density Polythene

Low density polythene is essentially a linear chain polymer with a small number of branches. The effect of this is, that only limited crystallization (A crystalline structure is one in which there is an orderly arrangement of particles e.g metals) is possible. It results in lower density than if complete crystallization had been possible. Low density polythene softens in boiling water, the high density does not. Both forms of polythene have excellent chemical resistance, low moisture absorption and high electrical resistance (Bolton, 1987).

2.20.1.2 High Density Polythene

High density polythene is completely linear polythene and a high degree of crystallization is possible, resulting in the higher density. It does not soften in boiling water (Bolton, 1987).

2.20.2 Properties of Polythene

Some of the salient properties as described by R. M. Koerner (1985) and W. Bolton (1987) are:

Property	Low Density	High Density
Specific Gravity	0.92 – 0.94	0.94 – 0.96
Density (g/cm^3)	0.91 – 0.925	0.941 – 0.965
Melting Point (°C)	115	135
Tensile Strength (KN/ m^2)	8965 – 17240	16550 – 33000
Thickness (mils)	20 – 100	20 – 100
Elongation (%)	100 – 600	50 – 800
Resistance to Acids	Poor to Good	Good
Resistance to Bases	Good to Excellent	Good to Excellent
Maximum Service Temperature (°C)	85	125

2.20.3 Uses of Polythene

Low-density polythene is used mainly in the form of films and sheeting, e.g., polythene bags; 'squeeze' bottles, ball-point pen tubing, wires and cable insulation. High density polythene is used for piping, toys, filaments for fabrics and household ware. Low and high-density polythene can be blended to give a material with properties between the two separate forms to be used for some specific purpose. The additives commonly used with polythene are carbon black as a stabilizer, pigments to give colored forms, glass fibers to give increased strength and butyl rubber to prevent cracking in service (Bolton, 1987). Some of the current uses of polythene as described by R.M. Koerner (1985) are:

a. Landfill caps (Closures)
b. Waterproofing within tunnels
c. To prevent infiltration of water in sensitive areas
d. Beneath asphalt overlays as a waterproofing layer
e. Flexible form where loss of material cannot be allowed
f. Floating reservoir covers to prevent pollution
g. Liners for waste liquids (acidic and basic)
h. Within zoned earth dams for seepage control

2.20.3 Forming Process

It is formed by 'extrusion' process. The process is comparable with the squeezing of the toothpaste out of its tube. The polymer is fed into a screw mechanism which takes the polymer through a heated zone and forces it out through the die. If thin film or sheet is the required product, a die may be used which gives an extruded cylinder of material. This cylinder while still hot is inflated by compressed air to give a sleeve of thin film. Another way of obtaining film or sheet is to use a slit die and cool the extruded product by allowing it to fall vertically into some cooling system. The system yields continuous lengths of product. Intricate shapes can be produced and a high output rate is possible. Curtain rails, household guttering and polythene bags and films are produced by the extrusion process (Bolton, 1987).

2.21 Desired Specifications of a Clay Liner for Arid/Semi Arid Regions

Review of literature pertaining to compacted clay liners for arid/semi arid regions reveals three essential features:

- Low hydraulic conductivity (Less than 1×10^{-9} m/s)
- Low shrinkage potential (volumetric shrinkage to be less than 4%)
- Adequate strength to be stable (Significant for covers)

CHAPTER 3

SELECTION OF SOIL

3.1 General

To select a soil, classification test has to be performed. Various classification systems are available to classify the soil properties by association with soils of the same class whose properties are known to provide the engineer with an accurate method of soil description. The most frequently used soil classification system by geotechnical engineers, is Unified Soil Classification System. According to the system, soils are divided into coarse grained and fine-grained classes. The coarse grained soils have material more than 50% by weight retained on No. 200 sieve whereas fine grained soils have more than 50% passing No 200 sieve. Fine grained soils are divided further on basis of Atterberg limits, which establishes stages of soil consistency to describe quantitatively the effect of varying water content on consistency. It divides fine gained soils in groups of low and high compressibility with liquid limit (LL) of 50 being the dividing boundary (Sowers, 1970). Fine-grained soils are further categorized, on the basis of behavior of these soils with respect to plasticity index (PI) as shown below reference:

Soil Plasticity	PI
Non Plastic	0 – 3
Low Plastic	3 – 15
Medium Plastic	15 – 30
High Plastic	31 or more

Clay rich soils are considered suitable for use in compacted clay liners keeping in view their extreme low permeability. Tests with low and medium plastic clay were carried by Afzal, 1995. Therefore, high plastic clay was selected to study its behavior as clay liner, when mixed with sand , cement or polythene chips either separately or his combination. This portion of thesis work was intended to select the high plastic soil for the compacted clay liner.

3.2 Methodology

In order to use appropriate soil for the research work, soil of Kashmore being in Arid/Semi Arid Zone was tested. For this purpose Atterberg limit apparatus was taken to field and soil was extracted by digging pit and Atterberg limits were found out. Then the desired soil having LL > 50 and PI > 31 was collected in polythene bags and brought to laboratory for the research work. Density, natural moisture content, grain size analysis, Atterberg limits etc were determined for classification and index properties.

3.2.1 Density/Unit Weight

Tests were conducted as per the procedure adopted by ASTM D 2937-83. Density of the samples ranged from 19.22 to 19.72 KN/m^3. Calculations are reflected in Table 3.1.

3.2.2 Moisture Content

For determination of moisture content laboratory tests were conducted as per the method outlined by ASTM D 4643-87. Moisture content ranged from 20 % to 22 %. Calculations are summarized in Table 3.2.

3.2.3 Atterberg Limits

The tests were conducted according to ASTM D 4318-84. Plasticity Index (PI) of the three samples ranged between 31 to 46. Results of Atterberg limits are appended in Table 3.3 Calculations for the Atterberg limits for the samples are shown in Appendix A.

3.3 Summary (Test Results)

Summary of the test results of soil selected for the Research Work i.e., Density, Moisture content, and Atterberg limits for the soil procured from Kashmore are shown in Table 3.4. This soil was used for the subsequent research work.

Table 3.1 : Density Calculations for Selection of Soil

S. No	Location	Weight of soil +mold (W_1) gms	Weight of mold (W_2) Gms	Weight of soil (W_1-W_2) gms	Volume of mold (V) cm^3	Density (W_1-W_2) / V	
						gm / cm^3	KN/m^3
1	Kashmore	3092	1040	2052	1029.63	1.99	19.54
2		138.71	58.49	80.22	39.90	2.01	19.72
3		283.03	122.51	160.52	81.92	1.96	19.22

Table 3.2 : Moisture Content of the Soil Selected

S. No	Description	Weights (gms)		
		Sample No		
		1	**2**	**3**
1	Weight of wet soil + can	34.16	49.24	71.76
2	Weight of Dry soil + can	29.97	42.62	62.40
3	Weight of can	11.21	11.32	14.94
4	Weight of Water	4.19	6.62	9.36
5	Weight of Oven dried Soil	18.77	31.30	47.46
6	Moisture Content %	**22.33**	**21.15**	**19.72**

Table 3.3 : Atterberg Limits of Soil Selected for Research Work

Ser No	Density	Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Remarks
	KN / m^3	%	%	%		
1	19.54	22.33	71.65	26.02	45.63	HIGH
2	19.72	21.15	62.10	23.07	39.03	PLASTIC
3	19.22	19.72	58.6	27.6	31	CLAY

Table 3.4 : Summary of Test Results of Soil Selected for the Research Work

S. NO	TEST / PROPERTY	RESULT	REMARKS
1	Wet Density (KN / m^3)	19.72	**HIGH PLASTIC CLAY**
2	Moisture Content (%)	22.15	
3	Atterberg Limits a. Liquid Limit b. Plastic Limit c. Plasticity Index	 71.65 26.02 45.63	
4	Sieve Analysis a. Sand (%) b. Fines (%)	 0.62 99.38	
5	CLASSIFICATION OF SOIL	CH	

CHAPTER 4

EFFECT OF VARIATION OF MOISTURE CONTENT ON VOLUME CHANGE BEHAVIOR OF COMPACTED CLAY

4.1 General

Molding water content has a significant impact on the behavior of compacted clays. As the moisture content at compaction varies, it causes a change in the density, structure and the properties of the compacted soils. With the increase in moisture content, the density of soil increases up to a certain limit i.e., maximum dry density and an aggregated structure is formed. Maximum density is achieved at optimum moisture content. Further increase in moisture i.e., beyond optimum moisture content would cause a decline in the density and a dispersed structure would be formed with the flaky particles aligned in parallel. This changes the behavior of compacted clays.

Main concern in the design of clay liner is to reduce the seepage of water or contaminated fluid through it and therefore should have permeability less than $1x10^{-9}$ m/s (Evans, 1991). Permeability is affected by volume change behavior of clay. In case the clay liner is to be used in arid/semi arid regions, low shrinkage potential is another requirement i.e., volume shrinkage should not exceed 4 percent (Kleppe and Olson, 1985). Because, as the volume changes in excess of that, it would lead to desiccation cracks on drying and thus render the clay liner useless. Molding water content effects the volume change behavior of the clay liner and with its increase, the volume shrinkage also increases.

In this phase of study, optimum moisture content for compaction of test specimens was determined and then samples were compacted at three different moisture contents i.e.,2% above and below the optimum moisture content, to study the volume change behavior of the clay liner with varying moisture content.

4.2 Methodology

With sole aim of the research work to reduce shrinkage potential of the clay liner without effecting its permeability in arid/semi arid regions, it was considered to study the behavior of high plastic soil at constant compaction effort (Modified Proctor) and varying moisture content. It was decided arbitrarily to compact the samples at 2 % dry of optimum, at optimum and 2 % wet of optimum. Soil procured from Kashmore was air dried and lumps were broken and sieved through No. 10 Sieve. Moisture – density curve was then established by compacting samples at varying moisture content and corresponding densities and plotting graph of moisture content versus dry densities. Optimum moisture content was determined i.e water content corresponding to max dry density. Three samples each at three different moisture contents as described above were compacted, to evaluate the shrinkage potential, unconfined compression strength and permeability of each specimen. Since critical condition for the behavior of clay liner is when it is saturated. So, saturated UCC strength of the specimen were found out.

4.2.1 Compaction Test

For Modified Proctor compaction test, Standard ASTM procedure (ASTM D 1557) was followed. Mechanical compactor (Figure 4.1) was used and the dry preparation procedure was adopted. Moisture-Dry density curve for standard modified protor was plotted to establish optimum moisture content and maximum dry density (Figure 4.2). Optimum moisture content of the soil at Standard and Modified compactive efforts were 22% & 15.57% and maximum dry densities were 15.25 & 17.95 KN / m^3 respectively. So, for low molding water content, Modified compactive effort was used.

4.2.2 Volumetric Shrinkage Test

Three samples at each moisture content were prepared at modified compactive effort, using standard proctor mold. Samples were ejected from the compaction mold with the help of sample extruder (Figure 4.3). Procedure as described by D.E Daniel and Y.K.Wu (1992) was adopted for this test with slight variation of time period for observations and number of readings to note the dimensions. Initial dimensions were recorded just after extruding the specimen from the mold. Four measurements each for length and diameter were taken and the average for length and diameter was recorded to calculate the initial volume of the specimen. Dimensions were taken with the Electronic Vernier Caliper accurate to 0.01 mm (1/2540 in.). Samples were placed in the laboratory for three days to get them air dried at room temperature. Samples were then placed in the oven for drying at temperature of 110°C for three days. After three days samples were taken out and after some bearable cooling down of the samples the dimensions were again recorded and final volume and volume shrinkage of the samples in percent of initial volume was determined. It was noticed that after three days of oven drying the specimens got completely dried. Test results indicate a trend of increase in volume shrinkage as the moisture content increases from dry of optimum to wet of optimum. Table 4.1 summarizes the test results for volumetric shrinkage test.

Figure 4.1 : Mechanical Compactor that can be used for Standard as well as Modified Compactive Effort.

Figure 4.3 : Sample Extruder for 4” Dia Sample

Table 4.1 : Summary of Volumetric Shrinkage Test Results with Varying Moisture Content

MOISTURE CONTENT	SAMPLE NO	INITIAL VOLUME cm^3	DRY DENSITY KN/m^3	WATER CONTENT (%)	FINAL VOLUME cm^3	VOLUME CHANGE (%)
1	2	3	4	5	6	7
2 % DRY OF OMC	1	948.14	17.15	13.38	840.15	5.52
	2	945.07	17.27	13.15	841.87	5.28
	3	945.07	17.23	13.23	845.19	5.34
AT OMC	1	948.86	17.58	15.70	890.10	6.19
	2	943.47	17.47	16.35	883.05	6.40
	3	941.28	17.27	15.71	879.19	6.60
2 % WET OF OMC	1	945.72	17.10	17.37	877.73	7.19
	2	948.01	17.38	16.81	884.64	6.68
	3	948.66	17.12	16.47	880.13	7.22

4.2.3 Unconfined Compression Test

Three samples for each moisture content (i.e.,2% dry of optimum, at optimum, 2% wet of optimum) were extracted from 4" dia sample using 3.83 cm (1.5 inch) sample extruder (Fig 4.4). Samples were then cut to the size of 3.83 cm dia split sampler to have the ratio of height to diameter of 2. The average heights and diameters were 7.761 cm and 3.83 cm respectively. The samples were then saturated for three days by initially sprinkling the water for one day and then placing in small container and sand filled around the samples and water poured in the container. After three days, the samples were taken out and tested in an unconfined compression testing device (fig 4.5) as per the ASTM procedure (ASTM D 2166-85), to find out unconfined compressive strength. Table 4.2 shows the test results for unconfined compression test. Results indicate that the strength decreases with increase in moisture content.

Figure 4.4 : 1.5” Dia Sample Extruder

Figure 4.5 : Unconfined Compression Strength Testing Device

Figure 4.6 : Photograph of Specimen compacted 2 % Wet of Optimum

Table 4.2 : Summary of Unconfined Compression Test Results with Varying Moisture Content

SER	MOISTURE CONTENT	SAMPLE	DRY DENSITY KN/m^3	WATER CONTENT (%)	STRENGTH KN/m^2
A	b	c	D	e	f
1	2 % DRY OF OMC	1 2 3	17.36 17.18 17.61	13.21 13.43 13.64	261.94 278.46 272.93
2	AT OMC	1 2 3	17.51 17.63 17.29	15.70 16.35 15.71	239.42 214.63 226.58
3	2 % WET OF OMC	1 2 3	17.11 17.17 17.45	16.14 16.36 16.98	171.27 148.81 168.47

4.2.4 Falling Head Permeability Test.

As permeability of High Plastic clays can not be determined directly, an indirect method as described in “Soil Properties: Testing, Measurement and Evaluation, Cheng Liu and Jack B. Evett, chapter 15 (ASTM 2436-68)” was employed for determination of permeability of pure clay. Three samples were prepared in the permeameter at three different reduced densities. These samples were saturated and after permeameters were connected with the burette of Falling Head Permeability apparatus (Figure 4.7), Falling Head Permeability Test was performed and Permeability was determined. The void ratios of those samples were also determined and a graph was plotted between permeability and the void ratio. Permeability being the function of void ratio, the graph gave a straight trend line. From the dry densities of actual samples at each of three moisture contents i.e.,dry of OMC,at OMC and wet of OMC, the void ratios were determined and extrapolating the above referred trend line, the permeabilities against the desired void ratios were determined. Table 4.3 shows the summary of results. Results indicate that with increase in moisture content permeability of high plastic clay decreases.

Figure 4.7 : Falling Head Permeability Test Apparatus

Table 4.3 : Summary of Falling Head Permeability Test Results with varying Moisture Content

SER	SAMPLE TYPE	MOISTURE CONTENT %	VOID RATIO	PERMEABILITY 10^{-9} m/s
1	2 % Dry of OMC	13.25	0.630	0.00167 (Max)
2	At OMC	15.92	0.606	0.00109
3	2% Wet of OMC	16.65	0.579	0.0007 (Min)

4.3 Statistical Analysis

All the data obtained from various laboratory tests have been statistically analyzed to discard the various data points, to find out value of true mean with 90 percent confidence range, to find out the scatter or variability of data in terms of statistical functions such as “Range”, “Standard Deviation” and “Variance” and to carryout correlation – regression analysis in order to find the significance of correlation, if any, existing between the variables involved. In order to have clear understanding of the statistical terms involved, these terms are discussed briefly in subsequent paragraphs.

4.3.1 Statistics

The number calculated from a sample of data in general is called a statistic (Volk, 1969). Statistics refers to a body of methods by which useful conclusions can be drawn from numerical data (Grant and Leavenworth, 1972).

4.3.2 Significance Level (α)

Significance level of the statistical test represents the symmetrical probability for results in range (Volk 1969). It shows the probability of the occurrence of the test results.

4.3.3 Average

Any number which tends to represent the grouping tendency is called an average (Volk, 1969).

4.3.4 Mean (x)

This term is used for arithmetic average and is the value from which the sum of the squares of deviation is minimum.

4.3.5 Standard Deviation (σ)

It is the measure of the variability of the data. As the magnitude of the variation of data increases, standard deviation will also increase. It has same units as that of original data (Volk, 1969). It can be conveniently calculated from range (R) as explained in subsequent paragraphs.

4.3.6 Variance (σ^2)

Variance is the square of the standard deviation.

4.3.7 Range (R)

Range is the difference between the largest and the smallest value. Like variance, range is the measure of spread of observations. For small groups of data i.e. less than 8 samples, it is as efficient as the variance. Table to calculate standard deviation from range is also available. Table 4.4 gives the factors for various sample sizes to be multiplied with range in order to find standard deviation. Range can also be used to determine value of true mean in order to find standard deviation. Range can also be used to determine value of true mean (m) by equation.

$$m = x \pm u\,(R) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.1$$

Where, m = True Mean

x = Mean (Arithmetic Average)

u = Difference between mean of sample and the true mean of population (Value can be found out from Table 4.5 for various significance levels).

R = Range

4.3.8 Degree of Freedom

The number of independent measurements that are available for estimating a statistical parameter is called the degree of freedom of that estimate. If "N" is the number of observations, then "N-1" would be the degree of freedom (Volk, 1969).

4.3.9 Correlation – Regression Analysis

4.3.9.1 Correlation

It refers to degree of association between one variable and another or between one variable and several others (Yolk, 1969). There can be a linear or curvilinear correlation between the variables.

Table 4.4: Factors to Calculate Standard Deviation from Range (After Volk,1969)

SAMPLE SIZE	FACTOR
2	0.886
3	0.591
4	0.486
5	0.430
6	0.395
7	0.370
8	0.351
9	0.337
10	0.325
12	0.307
14	0.294
16	0.283
18	0.275
20	0.268

Table 4.5: Critical Values of "U" for Various Sample Sizes and Probability Level (After Volk 1969)

PROBABILITY LEVEL / SAMPLE SIZE	0.1	0.05	0.02	0.01
2	3.157	6.351	15.91	31.828
3	0.885	1.304	2.111	3.008
4	0.529	0.717	1.023	1.316
5	0.388	0.507	0.685	0.843
6	0.312	0.399	0.523	0.628
7	0.263	0.333	0.429	0.507
8	0.23	0.288	0.366	0.429
9	0.205	0.255	0.322	0.374
10	0.186	0.23	0.288	0.333
11	0.17	0.21	0.262	0.302
12	0.158	0.194	0.241	0.277
13	0.147	0.181	0.224	0.256
14	0.138	0.17	0.209	0.239
15	0.131	0.16	0.197	0.224
16	0.124	0.151	0.186	0.212
17	0.118	0.144	0.177	0.201
18	0.113	0.137	0.168	0.191
19	0.108	0.131	0.161	0.182
20	0.104	0.126	0.154	0.175

4.3.9.2 Regression

It deals with nature of relation between variables. With two variables that can be correlated linearly, the regression coefficient is the slope of line used to correlate the variable (Volk, 1969).

4.3.9.3 Linear Correlation and Least Square Line

For the data that can be correlated by a straight line, there is one straight line from which the sum of squares of deviations of one of variables is a minimum. This is "least Squares line".

If the pairs of values of the variables associated with each data point are designated as x_i and y_i with 'y' as dependent variable, a straight line through the data is expressed as:

$$\bar{y} = a + b\,x \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.2$$

Where, $\bar{y}$ = Estimated value of 'y' from an observed value of x.

a = Intercept, giving estimated value of y at x = 0.

b= Slope of line or regression coefficient.

The values of 'a' and 'b' corresponding to the line with minimum squared deviations of 'y' from $\bar{y}$ are :

$$a = \bar{y} - b\,\bar{x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.3$$

$$b = \frac{\Sigma(x-\bar{x})(y-\bar{y})}{\Sigma(x-\bar{x})^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.4$$

or
$$b = \frac{\Sigma xy - (\Sigma x\ \Sigma y)\ /\ N}{\Sigma x^2 - (\Sigma x)^2\ /\ N} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.5$$

Where N = Number of data points.

4.3.9.4 Correlation Coefficient (r)

It is a measure of the amount of relation between variables. When there is a perfect correlation between x and y, there is no residual deviation of y from $\bar{y}$ and "r^2" (Coefficient of Determination) equals to 1.0. When there is no correlation and none of the sum of squares of deviation is removed by linear relationship, 'r^2' equals to zero. It ranges from +1.0 to 0.0 (Chapra and Canale). Value of 'r^2' can be calculated as:

$$r^2 = \frac{\Sigma' y^2 - \Sigma'\,\bar{y}^2}{\Sigma' y^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.6$$

Where
$$\Sigma' y^2 = \Sigma\,(\,y - y'\,)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.7$$

$$\Sigma'\,\bar{y}^2 = \Sigma' y^2 - b\ \Sigma' xy \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.8$$

$$\Sigma' xy = \Sigma\,xy - \bar{X}\,\Sigma y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.9$$

$\bar{X}$ = Mean value of 'x' variables

Y' = Mean value of 'y' variables

b = Regression coefficient

4.3.9.5 Significance of Correlation

If the hypothesis is made that the correlation coefficient is equal to zero , 'r' is related to t-test and is possible to tabulate the values of 'r' corresponding to various probability levels and the degrees of freedom based on the hypothesis that there is no correlation between the two variables involved (Table 4.6). Values of 'r' in table are the maximum values of correlation coefficient that can be expected by chance for the amount of data involved if there is no correlation. The probability level indicates the chance of getting a value of ‘r’ as large as the tabulated value when there is no correlation. The 0.10 level means there is only a 10 percent chance of getting a value of ‘r’ as large as those in 0.10 column when no correlation exists.

4.3.9.6 Non Linear or Curvilinear Regression

Relation ships between engineering variables are not always linear or may not always be adequately described by the linear models. Experimental data for such variables may show a non linear trend between the observed values of the variables. Although a linear relation may be used to describe the general trend, but the predictions based on such linear relationships may overestimate or underestimate the expected results (Ang and Tang, 1975).

Analytic expression relating the variables in such cases is given by adding second and third degree terms of the independent variables to improve the correlation (Volk,1969):

$$\bar{y} = a + bx^2 + cx \quad \ldots\ldots \quad 4.10$$

where
$$b = \frac{(\Sigma' x^2 y)(\Sigma' x^2) - (\Sigma' xy)(\Sigma' xx^2)}{(\Sigma' x^2)\,\Sigma'(x^2)^2 - (\Sigma' xx^2)^2} \quad \ldots\ldots \quad 4.11$$

$$c = \frac{(\Sigma' xy)(\Sigma' x^2)^2 - (\Sigma' x^2 y)(\Sigma' xx^2)}{(\Sigma' x^2)\,\Sigma'(x^2)^2 - (\Sigma' xx^2)^2} \quad \ldots\ldots \quad 4.12$$

$$a = y' - c\bar{x} - b(\bar{x}^2) \quad \ldots\ldots \quad 4.13$$

$$\Sigma' x^2 = y'(\Sigma x)^2/N \quad \ldots\ldots \quad 4.14$$

$$\Sigma' x^2 y = \Sigma x^2 y - (\Sigma x^2 \Sigma y)/N \quad \ldots\ldots \quad 4.15$$

4.3.9.7 Computer Software (Statistical Analysis)

Use of computer software (Excel, Microsoft Windows) makes it very convenient to carry out the correlation – regression analysis for both the linear and curvilinear analysis. X and Y variables are plotted with the help of “Chart Wizard” and option is given for the display of correlation equation along with the value of coefficient of determination (r^2). Option can be set for linear or polynomial equation. Computer would display the equation and value of ‘r^2’ along with the curve and trend line (the best fitted curve or line).

4.3.10 Discarding Data

The method given by Dixon (1951) can be used to discard data. The observations in the sample are identified as X_1, X_2,……, X_n. It is immaterial whether ranking proceeds from high values to low or in reverse order. For data points less than 8, the ratio $(x_2-x_1)/(x_n-x_1)$ be found out and compared with corresponding values in Table 4.7. If the ratio exceeds values in Table 4.7, the extreme value may be rejected with risk of error set by probability level (Volk, 1969).

Table 4.6 : Correlation Coefficient "r" (After Volk,1969)

DEGREE OF FREEDOM	PROBABILITY OF LARGER VALUE OF 'r'				
	0.1	0.05	0.02	0.01	0.001
1	0.988	0.997	1	1	1
2	0.9	0.95	0.98	0.99	1
3	0.805	0.878	0.934	0.959	0.991
4	0.729	0.811	0.882	0.917	0.974
5	0.669	0.745	0.833	0.874	0.951
6	0.622	0.707	0.789	0.834	0.925
7	0.582	0.666	0.75	0.78	0.898
8	0.549	0.632	0.716	0.765	0.872
9	0.521	0.602	0.685	0.735	0.847
10	0.497	0.576	0.658	0.708	0.823
12	0.458	0.532	0.612	0.661	0.78
14	0.426	0.497	0.574	0.623	0.742
16	0.4	0.468	0.542	0.59	0.708
18	0.378	0.444	0.516	0.561	0.679
20	0.36	0.423	0.492	0.537	0.652
25	0.323	0.381	0.445	0.487	0.597
30	0.296	0.349	0.409	0.449	0.554
35	0.275	0.325	0.381	0.418	0.519
40	0.257	0.304	0.358	0.393	0.49
45	0.243	0.288	0.338	0.372	0.465
50	0.231	0.273	0.322	0.354	0.443

Table 4.7 : Maximum Ratio of Extreme Ranking Observations (After Volk, 1969)

Recommended for Sample Size	Rank Difference Ratio	Sample Size n	Maximum Ratio Probability Level 0.1	0.05	0.01
'n' less than 8	$\frac{X_2-X_1}{X_n-X_1}$	3	0.886	0.941	0.988
		4	0.679	0.765	0.889
		5	0.557	0.642	0.780
		6	0.482	0.560	0.698
		7	0.434	0.507	0.637
'n' between 8-14	$\frac{X_3-X_1}{X_{n-1}-X_1}$	8	0.650	0.710	0.829
		9	0.594	0.657	0.776
		10	0.551	0.612	0.726
		11	0.517	0.576	0.679
		12	0.490	0.546	0.642
		13	0.467	0.521	0.615
		14	0.448	0.501	0.593
'n' more than 14	$\frac{X_3-X_1}{X_{n-2}-X_1}$	15	0.472	0.525	0.616
		16	0.454	0.507	0.595
		17	0.438	0.490	0.577
		18	0.424	0.475	0.561
		19	0.412	0.462	0.547
		20	0.401	0.450	0.535

4.4 Analysis of Test Results

Results obtained from volume shrinkage test and unconfined compressive strength test were statistically analyzed to find out their significance and confidence range. Illustration of how statistical analysis was carried out is given in the subsequent paragraphs.

4.4.1 Illustration (Statistical Analysis)

To start with, results obtained from volume shrinkage test with varying moisture content are being analyzed statistically. Illustration of the procedure adopted for the statistical analysis is given in subsequent paragraphs.

4.4.1.1 Laboratory Test Data (Volume Shrinkage Test Result)

Table 4.1 summarizes the test data.

4.4.1.2 Step 1-Discarding Extreme Scattered Data Points by Dixon Method

a. Table 4.8 shows the calculations.

b. First three columns give test data taken from Table 4.1

c. In column 4, the data is arranged in ascending or descending order for each sample of observations. Point to note is that the extreme scattered point that is to be considered for discarding should be taken as X_1 and accordingly the other two points should be arranged in ascending or descending order.

   For No 1 set of samples 1 sequence would be 5.52, 5.34, 5.28

   For No 2 set of samples 2, sequence would be 6.60, 6.40, 6.19

   For No 3 set of samples 3, sequence would be 6.68, 7.19, 7.22

d. In column 5, the ratio $(X_I - X_2)/(X_1 - X_3)$ is determined;

   For No 1 set of samples, ratio would be 0.750

   For No 2 set of samples, ratio would be 0.488

   For No 3 set of samples, ratio would be 0.944

e. In column 6, the ratio calculated in column 5 is compared with the maximum allowable ratios of extreme ranking observations in Table 4.5 for the particular sample size. If the calculated ratio for a particular size of observations exceeds that given in Table 4.5, the extreme scattered data point may be discarded with the risk of error set by the probability level (maximum of 0.10). If Column 6 shows "Yes" the data point X_1 can be discarded and if "No", the discarding test fails. From the ratios computed above, we notice that only in third set of samples the extreme scattered data point can be discarded, as the calculated ratio in that case (i.e.,0.944) exceeds the maximum allowable ratio in Table 4.5, i.e., 0.886 for the risk or probability of error of 0.10 level for sample size 3.

f. Column 7 shows the final modified data after having discarded the points that qualify the test. From this point onwards, modified data would be used for the rest of the analysis.

Table 4.8: Discarding Data Points for Volumetric Shrinkage Test with varying moisture content by Dixon Method

SAMPLE TYPE	M.C %	V.C %	ARRANGED DATA X_1 , X_2 , X_3	RATIO $(X_1-X_2)/(X_1-X_3)$	DISCARD POINT (TABLE 4.7)	MOD DATA
1	2	3	4	5	6	7
2% DRY OF OMC						
1	13.38	5.52				5.52
2	13.15	5.28	5.52,5.34,5.28	0.750	NO	5.28
3	13.23	5.34				5.34
AT OMC						
1	15.70	6.19				6.19
2	16.35	6.40	6.60 , 6.40 , 6.19	0.488	NO	6.40
3	15.71	6.60				6.60
2 % WET OF OMC						
1	17.37	7.19				7.19
2	17.18	6.68	7.22 , 7.19 , 6.68	0.944	YES	X
3	16.47	7.22				7.22

4.4.1.3 Step 2-Computation of Statistical Parameters and Value of True Mean

a. Table 4.9 shows the calculations.

b. First two columns show the modified data (same as in Column 7 of the Table 4.6.

c. In third Column the Range 'R' is calculated. It is difference of two extreme data points in a sample ;

Sample 1:	5.52 – 5.28	=	0.24
Sample 2:	6.60 - 6.19	=	0.41
Sample 3:	7.22 - 7.19	=	0.10

d. Column 4 shows the calculation of Standard Deviation (SD) from the range. Factor for calculation of standard deviation from range is taken from Table 4.4. Factor for sample size 2 and 3 is 0.886 and 0.591respectively. This factor is to be multiplied with range to get standard deviation :

Sample 1 :	0.591 x 0.24	=	0.1418
Sample 2:	0.591 x 0.41	=	0.2423
Sample 3:	0.886 x 0.10	=	0.0886

e. Column 5 shows the calculation of Variance (VAR) which is the square of standard deviation;

Sample 1 : 0.0201

Sample 2 : 0.0587

Sample 3 : 0.0078

f. Column 6 shows the calculation of Mean ($\bar{x}$) which is the arithmetic average of the data points in a sample;

Sample 1: 5.38

Sample 2: 6.40

Sample 3: 7.21

g. Column 6 shows the calculation of Mean ($\bar{x}$) which is the arithmetic average of the data points in a sample;

Sample 1: 5.38

Sample 2: 6.40

Sample 3: 7.21

h. Column 7 shows the calculation of True Mean (m) for 90% confidence range. Equation 4.1 is used for the computation of 'm'. The value of "u" in equation 4.1 is obtained from the table 4.5 for the particular sample size and probability level.In our case the value of 'u' for sample size 3 and probability level 0.1 happens to be 0.885. Using the equation, the value 'm' for each sample is calculated in Column 7. This True mean has two values.

4.4.1.4 Step 3 – Correlation-Regression Analysis

a. Correlation – Regression analysis was carried out to see whether the correlation exists between the moisture content and the volume shrinkage property, and if it exists, then with what significance. Modified data after discarding the extreme scattered points was used for the purpose.

b. First, the data was analyzed to check the linear correlation as explained in paragraph 4.3.9.3. Equation of the best fitted curve given in equation 4.2 can be established by computing values of "a" and "b" using equations 4.3 and 4.5 respectively. The value of "r^2" computed by equations 4.6 to 4.9 when compared with the values in Table 4.6 would dictate the significance of the correlation.

Table 4.9 : Computation of statistical parameters for Volume Change test results with varying moisture content

SAMPLE	V.C %	RANGE R ($X_3 - X_1$)	STD DEV SD R X 0.591	VARIANCE VAR $(SD)^2$	MEAN $\bar{X} = \frac{(X_1+X_2+X_3)}{3}$	TRUE MEAN FOR 90% CONFIDENCE RANGE [m = $\bar{X} \pm 0.885$ (R)] (%)
1	2	3	4	5	6	7
2% DRY						
1	5.52					5.54
2	5.28	0.24	0.142	0.0201	5.38	
3	5.34					5.22
AT OMC						
1	6.19					6.76
2	6.40	0.41	0.2423	0.0587	6.40	
3	6.60					6.04
2% WET OF OMC						
1	7.19					7.23
2	x	0.1	0.0886	0.0078	7.21	
3	7.22					7.18

If the linear correlation was less significant or the linear correlation did not hold then the non linear or curvilinear regression was resorted to, as explained in paragraph 4.3.9.6.

c. Computations for the linear correlation equation are :

	X	Y	XY	X^2	Y^2	Y-Y`	$(Y-Y`)^2$
	13.38	5.52	73.86	179.02	30.47	-0.75	0.56
	13.15	5.28	69.43	172.92	27.88	-2.78	7.73
	13.23	5.34	70.65	175.03	28.52	-2.72	7.40
	15.70	6.19	97.18	246.49	38.32	-1.87	3.50
	16.35	6.40	104.64	267.32	40.96	-1.66	2.76
	15.71	6.60	103.69	246.80	43.56	-1.46	2.13
	17.37	7.19	124.89	301.72	51.70	-0.87	0.76
	16.47	7.22	118.91	271.26	52.13	-0.84	0.71
MEAN	15.17	6.22	95.41	232.57	39.19		
Σ	121.36	49.74	763.25	1860.58	313.53	-12.95	25.54

$a = -0.5316$ [Using Equation 4.5, N = 8]

$b = 0.4449$ [Using Equation 4.3]

$r^2 = 0.9067$

$r = 0.9522$

d. If we compare the value of "r" with those of Table 4.6 for N-1 degree of freedom (Volk, 1969), we notice that the value is greater than 0.05 probability level and is negligibly smaller than the 0.02 probability level. It shows that the hypothesis, that there is no correlation can be rejected with only 0.05 to 0.02 chance of being wrong or in other words the correlation has between 95 and 98% significance level.

e. Computations for the curvilinear regression:

	X	Y	X^2	Y^2	XY	XX^2	$(X^2)^2$	X^2Y
	13.38	5.52	179.02	30.47	73.86	2395.35	32049.74	988.2147
	13.15	5.28	172.92	27.88	69.43	2273.93	29902.19	913.0308
	13.23	5.34	175.03	28.52	70.65	2315.69	30636.52	934.6757
	15.70	6.19	246.49	38.32	97.18	3869.89	60757.32	1525.773
	16.35	6.40	267.32	40.96	104.64	4370.72	71461.32	1710.864
	15.71	6.60	246.80	43.56	103.69	3877.29	60912.26	1628.907
	17.37	7.19	301.72	51.70	124.89	5240.82	91033.09	2169.345
	16.47	7.22	271.26	52.13	118.91	4467.67	73582.48	1958.504
$\sum$	121.36	49.74	1860.58	313.53	763.25	28811.36	450334.91	11829.31
MEAN	15.17	6.22	232.57	39.19	95.41	3601.42	56291.86	1478.66

$c = 0.0136$ [Using equation 4.12]

$b = 0.0144$ [Using equation 4.11]

$a = 2.6688$ [Using equation 4.13]

$r^2 = 0.9076$

$r = 0.953$

f. If we compare the value of "r" i.e., 0.953 with that of Table 4.6 for N-1 degree of freedom (Volk, 1969), we notice that it is same as of linear correlation. It shows that the hypothesis, that there is no correlation can be rejected with only

0.05 chance of being wrong or in other words the correlation has between 95% and 98 % significance level. The results obtained from the correlation-regression analysis have been summarized in Table 4.10 and the Figure-4.8 reflects the correlation in graphical form. Same results can be obtained directly by using statistical software (Excel) which adopts the procedure explained in paragraph 4.3.9.7.

Table 4.10 : Correlation – Regression Analysis for Volume Shrinkage Test Results with varying Moisture Content

SAMPLE	M.C %	V.C %	INTERCEPT 'a'	SLOPE 'b'& 'c'	r^2	r	SIGNIFICANCE OF CORRELATION (%) (TABLE 4.6)
1	2	3	4	5	6	7	8
2% DRY OF OMC							
1	13.38	5.52					
2	13.15	5.28					
3	13.23	5.34		0.0144			
AT OMC			2.6688		0.9076	0.953	95
1	15.70	6.19		0.0136			
2	16.35	6.40					
3	15.71	6.60					
2 % WET OF OMC							
1	17.37	7.19					
2	x	x					
3	16.47	7.22					

4.4.2 Unconfined Compression Test

Results from the tests summarized in Table 4.2 were statistically analyzed adopting the same procedures as in case of data for volume shrinkage test. Extreme data points were discarded (Table 4.11), statistical parameters were calculated from modified data (Table 4.12) and correlation –regression analysis was carried out to determine significance level of correlation (Table 4.13). The graph indicates 90 percent significance level of correlation between moisture content and Unconfined Compression strength. Figure 4.9 depicts the trend of variation of strength with the change in molding moisture content along with their correlation. The trend line indicates decrease in Unconfined Compression strength with increase in molding water content i.e., wet of optimum.

Table 4.11 : Discarding Data Points for Unconfined Compression Test Results with varying Moisture Content by Dixon Method

M.C	Sample	Water Content	Strength (KPa)	Arranged Data $x_1, x_2, x_3, .., x_n$	Ratio	Discard Data Point	Modified Data
2% dry of omc	1 2 3	13.21 13.43 13.64	261.94 278.46 272.93	278.46 272.93 261.94	0.335	NO	261.94 278.46 272.93
At omc	1 2 3	15.48 15.01 14.76	239.42 214.63 226.58	239.42 226.58 214.63	0.518	NO	239.42 214.63 226.58
2%wet of omc	1 2 3	16.14 16.36 16.98	171.27 148.81 168.47	148.81 168.47 171.27	0.875	NO	171.27 148.81 168.47

Table 4.12 : Computation of Statistical Parameters for Unconfined Compression Test Results with varying Moisture Content by Dixon Method

Water Content %	Strength q_u , kpa	Range R X_3-X_1	Std Dev SD= Rx 0.591	Variance VAR= $(SD)^2$	Mean, X'= $\frac{(X_1+X_2+X_3)}{3}$	True Mean for 90 % Confidence Range m = X' ± 0.885(R)
2 % dry of omc	261.94 278.46 272.93	16.52	9.763	95.32	271.11	285.73 261.38
At omc	239.42 214.63 226.58	24.79	14.651	214.65	226.88	248.82 216.30
2% wet of omc	171.27 148.81 168.47	22.46	13.274	176.20	162.85	182.73 160.37

Table 4.13 : Correlation – Regression Analysis for Unconfined Compression Strength Test Results with varying Moisture Content

SAMPLE	M.C %	STR KPa	INTERCEPT 'a'	SLOPE 'b'& 'c'	r^2	r	SIGNIFICANCE OF CORRELATION (%) (TABLE 4.6)
1	2	3	4	5	6	7	8
2% DRY OF OMC							
1	13.21	261.94					
2	13.43	278.46					
3	13.64	272.93					
AT OMC							
1	15.48	239.42	709.91	-32.639	0.855	0.926	90 to 95
2	15.01	214.63					
3	14.76	226.58					
2 % WET OF OMC							
1	16.14	171.27					
2	16.36	148.81					
3	16.98	168.47					

4.4.3 Falling Head Permeability Test

As permeability of high plastic soil can not be determined directly through Falling Head Permeability Test Apparatus so using indirect approach i.e., by extrapolating the trend line of graph drawn between void ratio and the corresponding permeabilities for reduced density samples, permeabilities were determined which were far below the desired value. Hence permeability test results were not subjected to statistical analysis.

CHAPTER 5

EFFECT OF VARIATION OF SAND / CEMENT / POLYTHENE CONTENT ON VOLUME CHANGE BEHAVIOR OF COMPACTED CLAY

5.1 General

In order to reduce volume change potential, materials like sand, cement, lime and polythene chips can be used. Sand is expected to reduce shrinkage potential and increase permeability. Cement reduces shrinkage potential and permeability, and increases strength due to its fine grained nature and excellent bonding characteristics. Recycled polythene is expected to reduce the volume change potential.

To analyze the effect of addition of varying amount of sand,cement and polythene chips on the volume change potential of high plastic clay, volume shrinkage tests were carried out with constant compactive effort (modified proctor) and constant moisture content (2% wet of optimum).

5.2 Material used

5.2.1 Cement

Ordinary portland cement from Askari Cement Ltd was used.

5.2.2 Sand

Medium sand procured from Lawrencepur (District Attock) was used.

5.2.3 Polythene

Ordinary polythene bags of black color locally available were cut to chips size desired as per experimental matrix. Polythene used was a low density polythene (0.91 g/cm^3) and approximate tensile strength of 8965 KN/m^2.

5.3 Methodology

Samples were prepared for the tests as explained in paragraph 4.2. Three samples for each proportion of cement, sand and polythene chips were prepared using Modified compaction effort and moisture constant at 2 % wet of optimum for each test. Only ± 1 % variation was allowed as the effect of additives on various properties can only be analyzed by keeping the moisture content constant. The specimens were tested for volumetric shrinkage only and other tests were to be performed once shrinkage criterion was met.

Results indicate that sand caused reduction in volume shrinkage potential of High plastic clay up to average of 4.67 % of initial volume. Cement reduced shrinkage up to average of 6.0 %. Reduction in the shrinkage potential due to Polythene chips, in one of the sizes, was observed. For instance, 0.5 x 1.0 cm size polythene chips reduced volume shrinkage up to 6.0 % whilst other sizes did not work. It was therefore considered to use 0.5 x 1.0 cm chips size for composite clay liner.

5.3.1 Volume Shrinkage Test

Varying quantities (% by weight) of cement, sand and polythene chips were mixed in soil thoroughly. Three samples were prepared for each percentage of cement, sand and polythene in the compaction mold at 2 % wet of optimum. Samples were ejected from the compaction mold by using sample extruder. The procedure as outlined in paragraph 4.2.4 was adopted to find out the percent volume change of sample. Figure 5.1 to 5.4 shows the photographs of specimen with various sizes of polythene and there various contents . Table 5.1 summarizes the volumetric shrinkage test results.

5.4 Statistical Analysis

Results obtained from the three tests were statistically analyzed adopting the same procedure as in paragraph 4.4, to find out the significance of the test results and the correlation between varying amount of cement, sand, polythene and volume change potential. The moisture content was kept constant at 2% wet of optimum. The only difference in analysis from that of paragraph 4.4 is that of variation in “x” variables. “x” variable in this case would be the amount (by weight) of cement, sand and polythene added in soil instead of varying moisture content.

5.4.1 Volumetric Shrinkage Test

Test results summarized in Table 5.1 were analyzed. All data points in each sample were analyzed to discard extreme data points and to modify the data (Table 5.2). Statistical quantities were computed (Table 5.3) and correlation regression analysis was carried out to find significance of correlation (Table 5.4). Variation of volume change with varying amount of sand, cement and polythene chips content were plotted (Fig 5.5 to Fig 5.8).

Fig 5.1 : Photograph of Sample Mixed with 1.0 % Polythene Chips of Size 0.5 x 0.5 cm

Fig 5.2 : Photograph of Sample Mixed with 0.5% Polythene Chips of Size 0.5 x 1.0 cm

Fig 5.3 : Photograph of Sample Mixed with 0.5% Polythene Chips of Size 1.0 x 1.0 cm

Fig 5.4 : Photograph of Sample Mixed with 1.0% Polythene Chips of Size 1.0 x 1.0 cm

Table 5.1 : Summary of Volumetric Shrinkage Test Results with Varying Amount of Cement, Sand and Polythene Chips

TYPE OF SAMPLE	SAMPLE NO	INITIAL VOLUME cm^3	M.C %	DRY DENSITY gm/cc	FINAL VOLUME cm^3	VOLUME CHANGE %
CEMENT						
3 %	1	944.74	15.44	1.793	883.81	6.45
	2	944.74	15.12	1.787	884.68	6.36
	3	944.74	15.73	1.765	889.92	5.80
6 %	1	945.50	17.65	1.761	882.82	6.63
	2	943.48	18.35	1.754	878.61	6.87
	3	944.00	16.73	1.775	876.30	7.17
9 %	1	938.55	18.24	1.730	877.00	6.56
	2	938.95	17.23	1.789	877.91	6.50
	3	938.00	15.74	1.806	886.09	5.53
SAND						
15 %	1	944.00	14.24	1.843	889.62	5.75
	2	944.00	15.82	1.579	885.63	6.18
	3	918.29	15.85	1.880	864.08	5.81
30 %	1	941.89	16.50	1.868	864.42	7.80
	2	941.41	16.31	1.873	865.66	8.05
	3	915.26	15.78	1.928	862.54	5.76
45 %	1	946.36	11.85	1.997	864.97	8.60
	2	939.54	14.20	1.938	866.35	7.79
	3	919.28	15.71	1.927	866.08	5.74
60 %	1	942.30	15.17	1.844	857.60	8.99
	2	942.30	13.67	1.941	872.47	7.41
	3	943.39	13.53	1.942	899.39	4.66

Continued…..

Table 5.1: Continued …..

TYPE OF SAMPLE	SAMPLE NO	INITIAL VOLUME cm^3	M.C %	DRY DENSITY gm/cc	FINAL VOLUME cm^3	VOLUME CHANGE %
POLYTHENE						
a. 0.5x0.5 cm	1	946	18.55	1.476	860.97	8.79
(1) 0.5 %	2	946	19.38	1.488	846.51	10.33
	3	946	19.74	1.495	847.63	10.21
(2) 0.75 %	1	949.31	14.28	1.810	866.62	8.71
	2	946.36	15.89	1.783	861.61	8.95
	3	938.95	16.73	1.851	861.30	8.27
(3) 1.0 %	1	953.85	16.73	1.763	866.38	9.17
	2	943.47	18.72	1.744	857.92	9.07
	3	944	17.85	1.495	849.12	10.05
b. 0.5x1.0 cm						
(1) 0.5 %	1	942.46	20.09	1.743	861.01	8.64
	2	941.57	19.59	1.759	857.96	8.88
	3	954.35	15.67	1.792	891.83	6.55
(2) 0.75 %	1	941.73	17.98	1.733	852.68	9.46
	2	942.13	16.54	1.784	857.62	8.97
	3	943.48	17.23	1.796	868.47	7.95
(3) 1.0 %	1	942	17.03	1.752	887.22	5.82
	2	942	17.95	1.718	883.37	6.22
	3	942	18.09	1.674	872.20	7.41
c. 1.0x1.0 cm	1	942	17.96	1.751	866.19	8.05
(1) 0.5 %	2	942	18.00	1.751	861.52	8.54
	3	942	20.32	1.717	864.53	8.96
(2) 0.75 %	1	978.18	17.81	1.700	893.14	8.69
	2	950.46	16.40	1.834	869.10	8.56
	3	956.37	16.79	1.856	873.93	8.62
(3) 1.0 %	1	939.24	16.33	1.823	864.88	7.92
	2	945.64	16.46	1.829	876.26	7.34
	3	951.88	15.53	1.799	873.39	8.25

Table 5.2 : Discarding Data Points for Volumetric Shrinkage Test Results with Varying Cement, Sand and Polythene Chips Content by Dixon Method

TYPE OF SAMPLE	SAMPLE NO	V. C %	ARRANGED DATA X_1,X_2,X_3	RATIO $\frac{(X_1-X_2)}{(X_1-X_3)}$	DISCARD POINT X_1 (TABLE 4.7)	MOD DATA V.C %
CEMENT 3 %	1 2 3	6.45 6.36 5.80	5.80,6.36,6.45	0.862	NO	6.45 6.36 5.80
6 %	1 2 3	6.63 6.87 7.17	6.63,6.87,7.17	0.444	NO	6.63 6.87 7.17
9 %	1 2 3	6.56 6.50 5.53	5.53,6.50,6.56	0.942	YES	6.56 6.50 X
SAND 15 %	1 2 3	5.75 6.18 5.81	6.18,5.81,5.75	0.860	NO	5.75 6.18 5.81
30 %	1 2 3	7.80 8.05 5.76	5.76,7.80,8.05	0.879	NO	7.80 8.05 5.76
45 %	1 2 3	7.60 7.79 5.74	5.74,7.60,7.79	0.907	YES	7.60 7.79 X
60 %	1 2 3	8.99 7.41 4.66	4.66, 7.41,8.99	1.575	YES	8.99 7.41 X

Continued…..

Table 5.2: Continued …..

TYPE OF SAMPLE	SAMPLE NO	V. C %	ARRANGED DATA X_1,X_2,X_3	RATIO $\frac{(X_1-X_2)}{(X_1-X_3)}$	DISCARD POINT X_1 (TABLE 4.7)	MOD DATA V.C %
POLYTHENE a. 0.5x0.5 cm (1) 0.5 %	1 2 3	8.79 10.33 10.21	8.79, 10.21,10.33	0.916	YES	X 10.33 10.21
(2) 0.75 %	1 2 3	8.71 8.95 8.27	8.27,8.71,8.95	0.353	NO	8.71 8.95 X
(3) 1.0 %	1 2 3	9.17 9.07 10.05	10.05,9.07,9.17	0.898	YES	9.17 9.07 X
b. 0.5x1.0 cm (1) 0.5 %	1 2 3	8.64 8.88 6.55	6.55,8.64,8.88	0.897	YES	8.64 8.88 X
(2) 0.75 %	1 2 3	9.46 8.97 7.95	7.95, 8.97,9.46	0.675	NO	9.46 8.97 7.95
(3) 1.0 %	1 2 3	5.82 6.22 7.41	5.82,6.22,7.41	0.252	NO	5.82 6.22 7.41
c. 1.0x1.0 cm (1) 0.5 %	1 2 3	8.05 8.54 8.96	8.05,8.54,8.96	0.538	NO	8.05 8.54 8.96
(2) 0.75 %	1 2 3	8.69 8.56 8.62	8.56,8.69, 8.62	0.538	NO	8.69 8.56 8.62
(3) 1.0 %	1 2 3	7.92 7.34 8.25	7.34,7.92,8.25	0.637	NO	7.92 7.34 8.25

Table 5.3 : Computation of Statistical Parameters for Volumetric Shrinkage Test Result with varying Cement, Sand and Polythene Chips Content

TYPE OF SAMPLE	SAMPLE NO	V. C %	RANGE R ($X_3 - X_1$)	STD DEV SD R X 0.591	VAR $(SD)^2$	MEAN, X\` $\frac{X_1+X_2+X_3}{3}$	TRUE MEAN FOR 90% CONFIDENCE RANGE [m =X ± 0.885 (R)] (%)
CEMENT 3 %	1 2 3	6.45 6.36 5.80	0.65	0.384	0.147	6.20	6.779 5.628
6 %	1 2 3	6.63 6.87 7.17	0.54	0.319	0.102	6.89	7.368 6.412
9 %	1 2 3	6.56 6.50 X	0.06	0.053	0.003	6.53	6.583 6.477
SAND 15 %	1 2 3	5.75 6.18 5.81	0.43	0.254	0.065	5.91	6.294 5.533
30 %	1 2 3	7.80 8.05 5.76	1.14	0.674	0.454	8.26	9.272 7.254
45 %	1 2 3	7.60 7.79 X	0.19	0.168	0.028	7.70	7.868 7.532
60 %	1 2 3	8.99 7.41 X	1.58	1.4	1.960	8.20	9.598 6.802

Continued…..

Table 5.3: Continued …..

TYPE OF SAMPLE	SAMPLE NO	V.C %	RANGE R (X_3–X_1)	STD DEV SD R X 0.591	VAR $(SD)^2$	MEAN, X` $\underline{(X_1+X_2+X_3)}$ 3	TRUE MEAN FOR 90% CONFIDENCE RANGE [m =X ± 0.885 (R)] (%)
POLYTHENE a. 0.5x0.5 cm (1) 0.5 %	1 2 3	X 10.33 10.21	0.12	0.106	0.011	10.27	10.376 10.164
(2) 0.75 %	1 2 3	8.71 8.95 8.27	0.68	0.402	0.162	8.64	9.245 8.042
(3) 1.0 %	1 2 3	9.17 9.07 X	0.1	0.089	0.008	0.912	9.209 9.032
b. 0.5x1.0 cm (1) 0.5 %	1 2 3	8.64 8.88 X	0.24	0.213	0.045	8.76	8.972 8.548
(2) 0.75 %	1 2 3	9.46 8.97 7.95	1.51	0.892	0.796	8.79	10.130 7.457
(3) 1.0 %	1 2 3	5.82 6.22 7.41	1.59	0.94	0.884	6.48	7.890 5.076
c. 1.0x1.0 cm (1) 0.5 %	1 2 3	8.05 8.54 8.96	0.91	0.538	0.289	8.52	9.322 7.711
(2) 0.75 %	1 2 3	8.69 8.56 8.62	0.13	0.077	0.006	8.62	8.738 8.508
(3) 1.0 %	1 2 3	7.92 7.34 8.25	0.91	0.538	0.289	7.84	8.642 7.031

Table 5.4 : Correlation – Regression Analysis of Volumetric Shrinkage Test Results with Varying Cement, Sand and Polythene Chips Content

TYPE OF SAMPLE	SAMPLE NO	V.C %	INTERCEPT 'a'	SLOPE 'b' & 'c'	r^2	r	Significance of Correlation (%) (Table 4.6)
CEMENT							
3 %	1 2 3	6.45 6.36 5.80	4.47	-0.0581 0.7522	0.641	0.80	< 90 %
6 %	1 2 3	6.63 6.87 7.17					
9 %	1 2 3	6.56 6.50 X					
SAND							
15 %	1 2 3	5.75 6.18 5.81	3.2795	-0.0024 0.2223	0.6771	0.82	< 90 %
30 %	1 2 3	7.80 8.05 5.76					
45 %	1 2 3	7.60 7.79 X					
60 %	1 2 3	8.99 7.41 X					

Continued …..

Table 5.4: Continued …..

TYPE OF SAMPLE	SAMPLE NO	V.C %	INTERCEPT 'a'	SLOPE 'b' & 'c'	r^2	r	Significance of Correlation (%) (Table 4.6)
POLYTHENE a. 0.5x0.5 cm (1) 0.5 %	1 2 3	X 10.33 10.21	19.833	16.827 -27.54	0.9279	0.96	> 95 %
(2) 0.75 %	1 2 3	8.71 8.95 8.27					
(3) 1.0 %	1 2 3	9.17 9.07 X					
b. 0.5x1.0 cm (1) 0.5 %	1 2 3	8.64 8.88 X	1.6633	-18.747 23.567	0.7929	0.89	< 90 %
(2) 0.75 %	1 2 3	9.46 8.97 7.95					
(3) 1.0 %	1 2 3	5.82 6.22 7.41					
c. 1.0x1.0 cm (1) 0.5 %	1 2 3	8.05 8.54 8.96	5.6233	-7.1467 9.36	0.5631	0.75	< 90 %
(2) 0.75 %	1 2 3	8.69 8.56 8.62					
(3) 1.0 %	1 2 3	7.92 7.34 8.25					

CHAPTER 6

EFFECT OF VARIATION OF SAND / CEMENT / POLYTHENE CONTENT ON BEHAVIOR OF COMPACTED HIGH PLASTIC CLAY OF COMPOSITE MATERIAL

6.1 General

Addition of sand, cement or polythene content did not reduce the volume change behavior to desired level. To reduce the volume shrinkage potential of high plastic clay it was decided to try composite sample having varying percent of cement, sand and polythene chips in high plastic clay. Therefore varying compositions, as given in experiment test matrix , were prepared and tested for volume change behavior

6.2 Material used

6.2.1 Cement

Ordinary Portland cement from Askari Cement Ltd was used.

6.2.2 Sand

Medium sand procured from Lawrencepur (District Attock) was used. Sieve analysis was carried out to determine the gradation of sand.

6.2.3 Polythene

Ordinary polythene bags of black color locally available were cut to chips size desired as per experimental matrix. Polythene used was a low density polythene (0.91 g/cm^3) and approximate tensile strength of 8965 KN/m^2.

6.3 Methodology

6.3.1 Volume Shrinkage Test

Three samples for volume shrinkage tests for each combination mentioned in experimental matrix table 1.2 were compacted at modified compactive effort with molding water content of 2% wet of optimum. Initial weight, initial volume and compaction moisture content were determined and after 7 days of curing/ drying at room temperature of laboratory, the samples were then placed for oven drying for three days at 110C° temperature. After oven drying, the final volume was determined by getting the dimensions of the sample diameter and lengths. Percent change in volume shrinkage was determined. Table 6.1 summarizes the results.

6.3.2 Unconfined Compression Test

This test was performed on samples of composite material i.e., samples with 30% sand, 6% cement and 0.5, 0.75, 1.0 % of polythene chips, showing desired volume shrinkage potential of less than 4 %. Three samples of average diameter of 3.83 cm and average height 7.761 cm of each composition were extracted from 4” diameter molded sample using 3.83 cm diameter sample extruder. Samples were then cured and saturated for three days and tested as per ASTM D 1633-84. Table 6.2 summarizes the results of unconfined compression test.

6.3.3 Consolidation Test

Samples having cement and high plastic clay could not be tested with Falling Head Permeability test for permeability determination. Therefore consolidation test was performed for determination of permeability using consolidation test data. Samples were extracted from 4˝ dia compaction mold. Consolidation sample ring was weighed. Samples were placed in the ring and along with the ring were weighed and cured for three days, then placed in oedometer and incremental load applied. Data for each load increment was noted and using the consolidation test data, permeability was calculated using the formula. Table 6.3 summarizes the permeability results. The consolidation test data and permeability determination details are given at Appendix “A”

6.4 Statistical Analysis

Results obtained were statistically analyzed adopting the same procedure as in paragraph 4.4, to find out the significance of the test results and also the correlation between varying amount of polythene and volume change potential, unconfined compression strength. The only difference in analysis from that of paragraph 4.4 is that of variation in “x” variables. “x” variable in this case would be the particular amount (by weight) of polythene added in soil instead of varying moisture content. Permeability test results analyses were not carried out because of insufficient data points.

6.4.1 Volumetric Shrinkage Test

Test results summarized in table 6.1 were analyzed. All data points in each sample were analyzed to discard extreme data points and to modify the data (Table 6.4).

Figure 6.1 : Photograph of Failed Specimen in UCC Strength Test.

Figure 6.2 : Photograph of Consolidation Test Apparatus

TABLE 6.1 : Summary of Volume Shrinkage Test Results of Composite Material

SER	SAMPLE COMPOSITION (BESIDES CLAY)			SAMPLE NO	INITIAL VOL cm^3	DRY DENSITY g/cc	MOISTURE CONTENT (%)	FINAL VOL cm^3	% VOLUME CHANGE
	P %	S %	C %						
1	0.5	15	3	1	944.33	1.81	15.67	895.46	5.17
				2	944.33	1.81	15.56	897.22	4.99
				3	944.33	1.82	15.20	896.03	5.11
2	0.75	15	3	1	943.97	1.79	16.87	890.82	5.63
				2	944.94	1.79	17.12	890.32	5.78
				3	946.12	1.79	16.54	893.23	5.59
3	1.0	15	3	1	794.00	1.78	17.18	744.0	6.29
				2	950.80	1.79	15.43	905.49	4.76
				3	947.29	1.81	14.11	902.70	4.71
4	0.5	30	3	1	950.51	1.86	16.25	901.47	5.16
				2	946.12	1.85	16.59	900.34	4.84
				3	948.37	1.84	16.14	897.59	5.35
5	0.75	30	3	1	945.12	1.83	17.23	892.76	5.54
				2	946.77	1.84	16.64	896.75	5.28
				3	945.06	1.84	16.79	893.46	5.46
6	1.0	30	3	1	945.25	1.88	14.84	901.22	4.66
				2	948.30	1.86	15.76	903.57	4.72
				3	947.69	1.83	17.18	903.87	4.62
7	0.5	15	6	1	950.00	1.88	14.71	893.87	5.91
				2	957.54	1.86	15.16	902.68	5.73
				3	941.98	1.85	15.59	898.82	4.58
8	0.75	15	6	1	946.56	1.78	17.21	903.02	4.60
				2	949.84	1.77	18.05	905.35	4.68
				3	948.12	1.78	16.99	904.89	4.56
9	1.0	15	6	1	945.28	1.83	17.05	898.90	4.96
				2	948.26	1.83	17.23	902.83	4.79
				3	940.06	1.83	16.88	901.42	4.11
10	0.5	30	6	1	947.46	1.85	16.21	916.99	3.22
				2	946.41	1.84	16.43	915.55	3.26
				3	946.72	1.83	16.83	918.79	2.95
11	0.75	30	6	1	948.11	1.83	16.68	914.83	3.51
				2	952.32	1.84	16.34	922.78	3.12
				3	949.53	1.82	16.86	915.44	3.59
12	1.0	30	6	1	946.00	1.84	16.85	908.55	3.96
				2	946.00	1.83	17.66	913.95	3.39
				3	946.00	1.84	16.83	912.60	3.53

Table 6.2 : UCC Strength (saturated) of Composite material

SER	COMPOSITION OF SAMPLE	SAMPLE	DRY DENSITY KN/m^3	WATER CONTENT (%)	UCC STRENGTH KN/m^2
a	b	c	d	e	f
1	0.5 % POLYTHENE 30 % SAND 6 % CEMENT	1 2 3	18.29 18.29 18.29	20.30 19.93 19.76	354.67 331.24 418.76
2	0.75 % POLYTHENE 30 % SAND 6 % CEMENT	1 2 3	18.96 18.67 19.43	16.87 16.94 17.23	482.51 501.29 509.10
3	1.0 % POLYTHENE 30 % SAND 6 % CEMENT	1 2 3	18.61 18.61 18.61	15.57 16.20 16.72	432.44 478.60 508.19

Table 6.3 : Permeability Test Results of Composite Material

SER	COMPOSITION	DRY DENSITY KN/m^3	MOISTURE CONTENT (%)	VOID RATIO	PERMEABILITY 10^{-9} m/s
a	b	c	d	e	f
1	0.5 % POLYTHENE 6 % CEMENT 30 % SAND	18.90	15.25	0.443	0.0155
2	0.75 % POLYTHENE 6 % CEMENT 30 % SAND	18.84	15.10	0.491	0.0785
3	1.0 % POLYTHENE 6 % CEMENT 30 % SAND	18.54	13.54	0.484	0.0114

Table 6.4 : Discarding Data Points for Volumetric Shrinkage Test Results with Varying Cement, Sand and Polythene Chips Content in Composite material for Sample by Dixon Method

SAMPLE TYPE CLAY PLUS			SAMPLE NO	V. C %	ARRANGED DATA X_1,X_2,X_3	RATIO $\frac{(X_1-X_2)}{(X_1-X_3)}$	DISCARD POINT X_1 (TABLE 4.7)	MOD DATA V.C %
P%	S%	C%						
0.5	15	3	1 2 3	5.17 4.99 5.11	4.99,5.11,5.17	0.667	NO	5.17 4.99 5.11
0.75	15	3	1 2 3	5.63 5.78 5.59	5.59,5.63,5.78	0.21	NO	5.63 5.78 5.59
1.0	15	3	1 2 3	6.29 4.76 4.71	6.29,4.76,4.71	0.968	YES	X 4.76 4.71
0.5	30	3	1 2 3	5.16 4.84 5.35	4.84,5.16,5.35	0.627	NO	5.16 4.84 5.35
0.75	30	3	1 2 3	5.54 5.28 5.46	5.28,5.46,5.54	0.692	NO	5.54 5.28 5.46
1.0	30	3	1 2 3	4.66 4.72 4.62	4.62,4.66,4.72	0.4	NO	4.66 4.72 4.62
0.5	15	6	1 2 3	5.91 5.73 4.58	4.58,5.73,5.91	0.865	NO	5.91 5.73 4.58
0.75	15	6	1 2 3	4.60 4.68 4.56	4.56,4.60,4.68	0.333	NO	4.60 4.68 4.56
1.0	15	6	1 2 3	4.96 4.79 4.11	4.96,4.79,4.11	0.200	NO	4.96 4.79 4.11
0.5	30	6	1 2 3	3.22 3.26 2.95	3.26,3.22,2.95	0.129	NO	3.22 3.26 2.95

Continued…..

Table 6.4: Continued …..

SAMPLE TYPE CLAY PLUS			SAMPLE NO	V. C %	ARRANGED DATA X_1,X_2,X_3	RATIO $\frac{(X_1-X_2)}{(X_1-X_3)}$	DISCARD POINT X_1 (TABLE 4.7)	MOD DATA V.C %
P%	S%	C%						
0.5	15	3	1 2 3	5.17 4.99 5.11	4.99,5.11,5.17	0.667	NO	5.17 4.99 5.11
0.75	15	3	1 2 3	5.63 5.78 5.59	5.59,5.63,5.78	0.21	NO	5.63 5.78 5.59
1.0	15	3	1 2 3	6.29 4.76 4.71	6.29,4.76,4.71	0.968	YES	X 4.76 4.71
0.5	30	3	1 2 3	5.16 4.84 5.35	4.84,5.16,5.35	0.627	NO	5.16 4.84 5.35
0.75	30	3	1 2 3	5.54 5.28 5.46	5.28,5.46,5.54	0.692	NO	5.54 5.28 5.46
1.0	30	3	1 2 3	4.66 4.72 4.62	4.62,4.66,4.72	0.4	NO	4.66 4.72 4.62
0.5	15	6	1 2 3	5.91 5.73 4.58	4.58,5.73,5.91	0.865	NO	5.91 5.73 4.58
0.75	15	6	1 2 3	4.60 4.68 4.56	4.56,4.60,4.68	0.333	NO	4.60 4.68 4.56
1.0	15	6	1 2 3	4.96 4.79 4.11	4.96,4.79,4.11	0.200	NO	4.96 4.79 4.11
0.5	30	6	1 2 3	3.22 3.26 2.95	3.26,3.22,2.95	0.129	NO	3.22 3.26 2.95

TYPE OF SAMPLE			SAMPLE NO	V. C %	ARRANGED DATA X_1,X_2,X_3	RATIO $\frac{(X_1-X_2)}{(X_1-X_3)}$	DISCARD POINT X_1 (TABLE 4.7)	MOD DATA V.C %
P%	S%	C%						
0.75	30	6	1 2 3	3.51 3.12 3.59	3.59,3.51,3.12	0.170	NO	3.51 3.12 3.59
1.0	30	6	1 2 3	3.96 3.39 3.53	3.96,3.53,3.39	0.754	NO	3.96 3.39 3.53

Table 6.5 : Computation of Statistical Parameters for Volumetric Shrinkage Test Result with varying Cement, Sand and Polythene Chips Content in Composite Material for Sample

TYPE OF SAMPLE [CLAY WITH % OF POLYTHENE (P), SAND (S) AND CEMENT (C)]			SAMPLE NO	V. C %	RANGE R ($X_3–X_1$)	STD DEV (SD) = R x 0.591	VAR $(SD)^2$	MEAN, X\` $\frac{(X_1+X_2+X_3)}{3}$	TRUE MEAN FOR 90% CONFIDENCE RANGE [m =X\`± 0.885 (R)] (%)
P%	S%	C%							
0.5	15	3	1 2 3	5.17 4.99 5.11	0.18	0.106	0.011	5.09	5.25 4.93
0.75	15	3	1 2 3	5.63 5.78 5.59	0.19	0.112	0.013	5.67	5.84 5.50
1.0	15	3	1 2 3	X 4.76 4.71	0.05	0.030	0.001	4.74	4.78 4.70
0.5	30	3	1 2 3	5.16 4.84 5.35	0.51	0.301	0.091	5.12	5.57 4.67
0.75	30	3	1 2 3	5.54 5.28 5.46	0.26	0.154	0.024	5.43	5.66 5.20
1.0	30	3	1 2 3	4.66 4.72 4.62	0.10	0.059	0.004	4.67	4.76 4.58

Table 6.5: Continued …..

TYPE OF SAMPLE [CLAY WITH % OF POLYTHENE (P), SAND (S) AND CEMENT (C)]			SAMPLE NO	V. C %	RANGE R $(X_3 – X_1)$	STD DEV (SD)= R x 0.591	VAR $(SD)^2$	MEAN, X` $\frac{(X_1+X_2+X_3)}{3}$	TRUE MEAN FOR 90% CONFIDENCE RANGE [m =X ± 0.885 (R)] (%)
P%	S%	C%							
0.5	15	6	1 2 3	5.91 5.73 4.58	1.33	0.786	0.618	5.41	6.59 4.23
0.75	15	6	1 2 3	4.60 4.68 4.56	0.12	0.071	0.056	4.61	4.72 4.50
1.0	15	6	1 2 3	4.96 4.79 4.11	0.85	0.502	0.252	4.62	5.37 3.87
0.5	30	6	1 2 3	3.22 3.26 2.95	0.31	0.183	0.033	3.14	3.41 2.87
0.75	30	6	1 2 3	3.51 3.12 3.59	0.47	0.278	0.077	3.41	3.83 2.99
1.0	30	6	1 2 3	3.96 3.39 3.53	0.57	0.337	0.114	3.63	4.13 3.13

Statistical quantities were computed (Table 6.5) and correlation regression analysis was carried out to find out significance of correlation (Table 6.6). Variation of volume change with varying amount of polythene chips content were plotted (Fig 6.3 to Fig 6.6).

Table 6.6 : Correlation – Regression Analysis of Volumetric Shrinkage Test Results with Varying Cement, Sand and Polythene Chips Content in Composite Material for Sample

TYPE OF SAMPLE [CLAY WITH % OF POLYTHENE (P), SAND (S) AND CEMENT (C)]			SAMPLE NO	V.C %	INTERCEPT 'a'	SLOPE 'b' & 'c'	r^2	r	SIGNIFICANCE OF CORRELATION (%) (Table 4.6)
P %	S %	C %							
0.5	15	3	1 2 3	5.17 4.99 5.11	-0.5883	-12.07 17.39	0.967	0.983	< 90 %
0.75	15	3	1 2 3	5.63 5.78 5.59					
1.0	15	3	1 2 3	X 4.76 4.71					
0.5	30	3	1 2 3	5.16 4.84 5.35	1.2867	-8.56 11.94	0.835	0.921	90 %
0.75	30	3	1 2 3	5.54 5.28 5.46					
1.0	30	3	1 2 3	4.66 4.72 4.62					
0.5	15	6	1 2 3	5.91 5.73 4.58	9.3933	6.4 -11.17	0.462	0.680	< 90 %
0.75	15	6	1 2 3	4.60 4.68 4.56					
1.0	15	6	1 2 3	4.96 4.79 4.11					

Continued …..

Table 6.6: Continued …..

TYPE OF SAMPLE [CLAY WITH % OF POLYTHENE (P), SAND (S) AND CEMENT (C)]			SAMPLE NO	V.C %	INTERCEPT 'a'	SLOPE 'b' & 'c'	r^2	r	SIGNIFICANCE OF CORRELATION (%) (Table 4.6)
P%	S%	C%							
0.5	30	6	1 2 3	3.22 3.26 2.95	2.4867	-0.3467 1.4867	0.494	0.703	< 90 %
0.75	30	6	1 2 3	3.51 3.12 3.59					
1.0	30	6	1 2 3	3.96 3.39 3.53					

6.4.2 Unconfined Compression Test

Test results summarized in Table 6.2 were analyzed. All data points in each sample were analyzed to discard extreme data points and to modify the data (Table 6.7). Statistical quantities were computed (Table 6.8) and correlation regression analysis was carried out to find out significance of correlation (Table 6.9). Variation of unconfined compression strength with varying amount of polythene chips content was plotted (Fig 6.7).

6.4.3 Permeability Test

Statistical analysis was not carried out because of insufficient number of data points.

Table 6.7 : Discarding Data Points for UCC Strength Test Results with Varying Polythene Chips Content in Sample of Composite material with 30 % Sand and 6 % Cement by Dixon Method

SAMPLE TYPE CLAY PLUS			SAMPLE NO	UCC STR, 'q_u' KPa	ARRANGED DATA X_1,X_2,X_3	RATIO $\frac{(X_1-X_2)}{(X_1-X_3)}$	DISCARD POINT X_1 (TABLE 4.7)	MOD DATA q_u
P%	S%	C%						
0.5	30	6	1 2 3	354.67 331.24 418.76	331.24 354.67 418.76	0.2677	NO	354.67 331.24 418.76
0.75	30	6	1 2 3	482.51 501.29 509.10	482.51 501.29 509.10	0.7063	NO	482.51 501.29 509.10
1.0	30	6	1 2 3	432.44 478.60 508.19	432.44 478.60 508.19	0.6094	NO	432.44 478.60 508.19

Table 6.8 : Computation of Statistical Parameters for UCC Strength Test Results with Varying Polythene Chips Content in Sample of Composite material with 30 % Sand and 6 % Cement

TYPE OF SAMPLE [CLAY WITH % OF POLYTHENE (P), SAND (S) AND CEMENT (C)]			SAMPLE NO	UCC STR, 'q_u' KPa	RANGE R $(X_3–X_1)$	STD DEV (SD) = R x 0.591	VAR $(SD)^2$	MEAN, X` $\frac{(X_1+X_2+X_3)}{3}$	TRUE MEAN FOR 90% CONFIDENCE RANGE [m =X`± 0.885 (R)] (%)
P%	S%	C%							
0.5	30	6	1 2 3	354.67 331.24 418.76	64.09	37.877	1434.67	368.22	424.94 311.50
0.75	30	6	1 2 3	482.51 501.29 509.10	26.59	15.715	246.96	497.63	521.16 474.10
1.0	30	6	1 2 3	432.44 478.60 508.19	75.99	44.910	2016.96	473.08	540.33 405.83

Table 6.9 : Correlation – Regression Analysis of UCC Strength Test Results with Varying Cement, Sand and Polythene Chips Content in Composite Material for Samples fulfilling Volume Shrinkage criterion

<table>
<tr><th colspan="3">TYPE OF SAMPLE [CLAY WITH % OF POLYTHENE (P), SAND (S) AND CEMENT (C)]</th><th rowspan="2">SAMPLE NO</th><th rowspan="2">UCC STR, ‘q_u’ KPa</th><th rowspan="2">INTERCEPT ‘a’</th><th rowspan="2">SLOPE ‘b’ & ‘c’</th><th rowspan="2">r^2</th><th rowspan="2">r</th><th rowspan="2">SIGNIFICANCE OF CORRELATION (%) (Table 4.6)</th></tr>
<tr><th>P%</th><th>S%</th><th>C%</th></tr>
<tr><td>0.5</td><td>30</td><td>6</td><td>1
2
3</td><td>354.67
331.24
418.76</td><td rowspan="3">2.4867</td><td rowspan="3">-0.3467
1.4867</td><td rowspan="3">0.494</td><td rowspan="3">0.703</td><td rowspan="3">< 90 %</td></tr>
<tr><td>0.75</td><td>30</td><td>6</td><td>1
2
3</td><td>482.51
501.29
509.10</td></tr>
<tr><td>1.0</td><td>30</td><td>6</td><td>1
2
3</td><td>432.44
478.60
508.19</td></tr>
</table>

CHAPTER 7

DISCUSSION ON RESULTS

7.1 Effect of Moisture Content Variation on Volume Change, Strength and Permeability Behavior of Compacted Clays

Test Results for samples compacted at varying moisture content were analyzed statistically. Table 7.1 shows the summary of results of three parameters i.e., unconfined compressive strength, volumetric shrinkage and permeability. Results show that a correlation exist between moisture content and unconfined compressive strength, volumetric shrinkage and permeability and reflects a clear trend of variation of unconfined compressive strength, volumetric shrinkage and permeability with increase in molding water content. As the molding water content exceeds optimum limit, i.e., wet of optimum, decrease in unconfined compressive strength and permeability of the samples takes place. It is due to the reason that the soils compacted dry of optimum have a flocculated structure due to which there is considerable inter-particle contact between positively charged clay mineral edges and their negative faces, producing strong bonds. Also, considerable free water is trapped in the large voids between the particles in addition to the adsorbed water. Strong bonds resist displacement and results in higher strength and the large voids between the particles make the structure more permeable. As the soil compacted dry of optimum is partially saturated, it would have low volume change potential. On the other hand, the soils compacted wet of optimum would form dispersed structure where the repulsion between the particles dominates and the particles position themselves for the maximum grain-to-grain distance. The structure is dense and watertight with weak inter-particle bonds resulting in low strength and low permeability. The soil compacted wet of optimum is saturated or nearly saturated and as it dries, a meniscus develops in each void at the soil surface which produces tension in the soil water and a corresponding compression in the soil structure resulting in the pronounced volume change. Test results confirm the researches on the subjects carried out by S.Ahmed, C.W.Lovell, and S.Diamond (1974), H .B Seed and C.K.Chan(1959) and by L.Barden and G.R.Sides (1970) that have been already described in paragraphs 2.17 and 2.16 respectively. Fig 7.1 represents these results graphically.

Table 7.1: Summary of Test Results With Varying Moisture Content

SER	MOISTURE CONTENT	SAMPLE	AVERAGE COMPACTION		V.C (%)	UCC STR KN/m^2	PERMEABILITY 10^{-9} m/s
			DENSITY KN/m^3	M.C (%)			
a	b	c	d	e	f	g	h
1	2 % DRY OF OMC	1 2 3	17.23	13.23	5.52 5.28 5.34	261.94 278.46 272.93	0.00167
		Average	**17.23**	**13.23**	**5.38**	**271.11**	**0.00167**
2	AT OMC	1 2 3	17.46	15.92	6.19 6.40 6.60	239.42 214.63 226.58	0.00109
		Average	**17.46**	**15.92**	**6.40**	**226.87**	**0.00109**
3	2 % WET OF OMC	1 2 3	17.22	16.69	7.19 6.68 7.22	171.27 148.81 168.47	0.0007
		Average	**17.22**	**16.69**	**7.03**	**162.85**	**0.0007**

The variation of permeability, the most important parameter for the clay liner, confirms the logic for general practice of compacting the clay liners wet of optimum rather than dry of optimum. Normally on dry of optimum, the permeability exceeds the minimum limit laid down for the clay liner, which is $1x10^{-9}$ m/s (Evans, 1991). But in case of high plastic clay of Kashmore, permeability i.e., 1.67 X 10^{-12}m/s has been with in limits specified. However, the fact is that it is normal practice to compact clay liners on wet of optimum to achieve minimum possible permeability. Therefore, all the samples in subsequent study were compacted wet of optimum. The percent volume change in case of high plastic clay, increases on wet of optimum, and exceeds the permissible limits laid down for the compacted clay liner i.e., 4 % volume change in arid/semi arid regions (Kleppe and Olson, 1985). In order to reduce this volume change on wet of optimum, it was decided to try and find out most suitable amount of cement, sand and recycled polythene chips while keeping moisture content 2 % wet of optimum for further study.

7.2 Effect of Varying Sand / Cement / Polythene Chips Content on Behavior of Compacted High Plastic Clays

Table 7.2 summarizes the statistically analyzed test results for high plastic clay mixed with varying amounts of cement, sand and polythene chips. These results were obtained from volumetric shrinkage test only because volume shrinkage was the essential criterion and other tests were to be conducted on those samples, which fulfilled this requirement. Test results show change in the volume shrinkage behavior of high plastic clay when mixed with cement, sand and polythene. Figure 7.2 gives the variation of volumetric shrinkage with the varying amount of polythene chips and its sizes, sand and cement. With the addition of sand, maximum dry density of the soil increases and the optimum moisture content drops down. It is due to the reason that with addition of sand, the range of particle sizes present is increased which results in greater maximum dry density (Lambe and Whiteman, 1979). Comparison of the test results obtained with addition of sand to those obtained without sand shown that, the sand tends to reduce the volumetric shrinkage potential. The reason is that the size of the clay particles are smaller than those of sand particles so the clay particles surround the sand particles and aggregated grain structure with comparatively larger inter-particle voids and weaker bonds is formed which results in reduced shear strength and increased permeability. Logic for the reduced volumetric shrinkage is that the sand does not have affinity for the water nor does the capillary tension develop in sands. As the moisture in the voids dry out, there would be no corresponding compression and thus no significant volume change. These alterations show a positive trend as far as our required compacted clay liners are concerned. With 15 percent sand the percent volume change was reduced to average of 5.91% as compared to average value of 7.03% with the pure soil, but it was still well beyond the acceptable limits of 4%. Since the volume shrinkage criterion could not be met by adding sand even up to 60 %, so other Tests i.e., strength and permeability test were ignored.

Addition of cement in clayey soils alters its behavior and at same compactive effort the maximum dry density of the soil cement mixture reduces slightly. For soil cement mixtures, having much more strength than the soil without cement, require comparatively large compactive effort to achieve the maximum density equal to that of

Table 7.2 : Summary of Volumetric Shrinkage Test Results with varying amount of Cement, Sand and Polythene Chips

TYPE OF SAMPLE	SAMPLE NO	MOISTURE CONTENT %	DRY DENSITY gm/cc	VOLUME CHANGE %
CEMENT				
3 %	1	15.44	1.793	6.45
	2	15.12	1.787	6.36
	3	15.73	1.765	5.80
	Average			**6.20**
6 %	1	17.65	1.761	6.63
	2	18.35	1.754	6.87
	3	16.73	1.775	7.17
	Average			**6.89**
9 %	1	18.24	1.730	6.56
	2	17.23	1.789	6.50
	3			X
	Average			**6.53**
SAND				
15 %	1	14.24	1.843	5.75
	2	15.82	1.579	6.18
	3	15.85	1.880	5.81
	Average			**5.91**
30 %	1	16.57	1.862	7.80
	2	16.50	1.868	8.05
	3	16.31	1.873	5.76
	Average			**7.20**
45 %	1	13.59	1.948	7.60
	2	11.85	1.997	7.79
	3			X
	Average			**7.70**
60 %	1	15.17	1.844	8.99
	2	13.67	1.941	7.41
	3			X
	Average			**8.20**

Continued…..

Table 7.2: Continued …..

TYPE OF SAMPLE	SAMPLE NO	MOISTURE CONTENT %	DRY DENSITY gm/cc	VOLUME CHANGE %
POLYTHENE a. 0.5x0.5 cm (1) 0.5 %	1 2 3	 19.38 19.74	 1.488 1.495	X 10.33 10.21
	Average			**10.27**
(2) 0.75 %	1 2 3	14.28 15.89 16.73	1.810 1.783 1.851	8.71 8.95 8.27
	Average			**8.64**
(3) 1.0 %	1 2 3	16.73 18.72	1.763 1.744	9.17 9.07 X
	Average			**9.12**
b. 0.5x1.0 cm (1) 0.5 %	1 2 3	20.09 19.59	1.743 1.759	8.64 8.88 X
	Average			**8.76**
(2) 0.75 %	1 2 3	17.98 16.54 17.23	1.733 1.784 1.796	9.46 8.97 7.95
	Average			**8.79**
(3) 1.0 %	1 2 3	17.03 17.95 18.09	1.752 1.718 1.674	5.82 6.22 7.41
	Average			**6.48**

Continued…..

Table 7.2: Continued …..

TYPE OF SAMPLE	SAMPLE NO	MOISTURE CONTENT %	DRY DENSITY gm/cc	VOLUME CHANGE %
c. 1.0x1.0 cm (1) 0.5 %	1 2 3	17.96 18.00 20.32	1.751 1.751 1.717	8.05 8.54 8.96
	Average			**8.52**
(2) 0.75 %	1 2 3	17.81 16.40 16.79	1.700 1.834 1.856	8.69 8.56 8.62
	Average			**8.62**
(3) 1.0 %	1 2 3	16.33 16.46 15.53	1.823 1.829 1.799	7.92 7.34 8.25
	Average			**7.84**

same soil without cement. If both the soils were compacted at same compactive effort, soil cement mixtures would result in comparatively lower maximum density. Addition of cement tends to increase the strength (q_u) , reduces permeability and the volume change potential. With increase in amount of cement content, this tendency becomes more pronounced. The reason for this change in behavior is because the particle sizes of the Portland cement (0.5 to 80 μm) are smaller than clay particles so the cement particles surround soil particles. As the cement hydrates, a gel is formed. Upon hardening this gel forms a cellular matrix that encapsulates the soil particles and stronger bonds are formed. The cement particles interact with the clay fractions to deprive them of their water affinity resulting in an improved soil with high strength, low permeability and low volume change potential. Figure 7.2 shows volume change behavior with varying cement content. With the addition of 3 percent cement (by weight), the percent volume change reduced to average 6.20 percent and addition of cement upto 9 percent did not reduce volume shrinkage to 4%. This shows that, to get the volume shrinkage potential to 4 %,

amount of cement content will have to be increased 12 to 15 %, which will make the proposition uneconomical.

Addition of polythene chips had already been tried in previous study (Waqar,1995). Based on that study, various sizes and proportions were tried to see their effects on volume shrinkage behavior of high plastic clays. The results show that although volume shrinkage reduces with the increase in chips content but if compared with the one without chips, the values of volume change are rather more instead of being lower. Figure 7.2 shows the variation of volumetric shrinkage with varying amount of polythene chips of various sizes i.e., 0.5 x 0.5, 1.0 x 0.5 and 1.0 x 1.0 cm size. Figure 5.7 to Fig 5.9 show that non-linear correlation exists between volume shrinkage and chips content. With the addition of varying content of polythene chips of each size tried, the volume change potential showed a downward trend, but it did not work to bring the volumetric shrinkage within the permissible limits of 4 %. Thus it reflects that polythene chips alone is not at all, a useful material that can be used with the high plastic soils in clay liners for reducing volume shrinkage.

7.3 Effect of Varying Sand / Cement and Polythene Content on Behavior of Compacted High Plastic Clay of composite material

Results are summarized in Table 7.3. Figure 7.3 shows the volume shrinkage behavior of all the combinations tried in this study. Results show that sand and cement content played contributory role in all the combinations and polythene chips contributed insignificantly. Since desired volume shrinkage occurred in samples having 30 % sand, 6% cement, so strength and permeability tests were also conducted on those samples only. Table 7.4 summarizes the results of three test parameters on above-mentioned samples. Figure 7.4 shows the behavior of this set of composition in volume shrinkage, unconfined compression and permeability. Volume shrinkage in samples varying percentage of polythene chip combined with 30 % sand and 6 % cement got reduced below 4% hence fulfilled one of the criterions of clay liner. Strength results showed a trend of increase in the strength with increase in chips content. The strength was sufficient to meet the criterion. However permeability increased with increase in chips content and was more for samples with chips 0.75% but comparatively reduced with 1.0% chip content. This increase in permeability is attributed to chips as well as sand.

TABLE 7.3 : Summary of Volume Shrinkage Test Results of Composite Material

SER NO	SAMPLE COMPOSITION (BESIDES CLAY)			SAMPLE NO	DRY DENSITY g/cm^3	MOISTURE CONTENT (%)	VOLUME CHANGE (%)
	Poly %	Sand %	Cement %				
1	0.5	15	3	1	1.81	15.67	5.17
				2	1.81	15.56	4.99
				3	1.82	15.20	5.11
2	0.75	15	3	1	1.79	16.87	5.63
				2	1.79	17.12	5.78
				3	1.79	16.54	5.59
3	1.0	15	3	1	1.78	17.18	6.29
				2	1.79	15.43	4.76
				3	1.81	14.11	4.71
4	0.5	30	3	1	1.86	16.25	5.16
				2	1.85	16.59	4.84
				3	1.84	16.14	5.35
5	0.75	30	3	1	1.83	17.23	5.54
				2	1.84	16.64	5.28
				3	1.84	16.79	5.46
6	1.0	30	3	1	1.88	14.84	4.66
				2	1.86	15.76	4.72
				3	1.83	17.18	4.62
7	0.5	15	6	1	1.88	14.71	5.91
				2	1.86	15.16	5.73
				3	1.85	15.59	4.58
8	0.75	15	6	1	1.78	17.21	4.60
				2	1.77	18.05	4.68
				3	1.78	16.99	4.56
9	1.0	15	6	1	1.83	17.05	4.96
				2	1.83	17.23	4.79
				3	1.83	16.88	4.11
10	0.5	30	6	1	1.85	16.21	3.22
				2	1.84	16.43	3.26
				3	1.83	16.83	2.95
11	0.75	30	6	1	1.83	16.68	3.51
				2	1.84	16.34	3.12
				3	1.82	16.86	3.59
12	1.0	30	6	1	1.84	16.85	3.96
				2	1.83	17.66	3.39
				3	1.84	16.83	3.53

Table 7.4 : Summary of Results of Test Parameters with varying polythene content in Composite Material having volume shrinkage less than 4 %

SER	COMPOSITION	SAMPLE NO	V.C %	STRENGTH (Kpa)		PERMEABILITY (10^{-9}m / sec)
				UNSAT	SAT	
1	0.5 % CHIPS + 30 % SAND + 6 % CEMENT	1 2 3	3.22 3.26 2.95	604 588 691	354.67 331.24 418.76	0.0152
		Average	**3.14**	**627.67**	**368.22**	**0.0152**
2	0.75 % CHIPS + 30 % SAND + 6 % CEMENT	1 2 3	3.08 3.13 3.15	787.23 698.45 785.11	469.73 426.65 418.19	0.0785
		Average	**3.12**	**756.93**	**438.19**	**0.0785**
3	1.0 % CHIPS + 30 % SAND + 6 % CEMENT	1 2 3	3.53 3.39 3.96	942.67 929.49 899.54	432.44 478.60 508.19	0.0114
		Average	**3.63**	**923.80**	**473.07**	**0.0114**

This increased permeability is also because of the reason that the samples were tested after 7 days curing and the curing was done by placing samples in odeometer and immersed in water. If the samples were cured for longer period of time, cement would get more time to form gel around sand particles reducing the voids and resultantly reducing the permeability of the mix. However, in our case, the permeability was well with in limits i.e; less than 1 x 10^{-9} m/s.

CHAPTER 8
SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary

The objective of this research work was to investigate the use of recycled/ waste polythene chips, cement and sand to reduce the shrinkage potential of High Plastic Compacted Clay in arid/ semi arid regions. This was accomplished by conducting volume shrinkage, Unconfined Compression, Falling Head permeability test for untreated soil and consolidation test for cement treated soil samples of High Plastic Clay (PI 46). Variation of volume shrinkage potential and strength were analyzed on samples compacted dry of optimum, at optimum and wet of optimum water content at constant compactive effort i.e., modified compactive effort. Variation of volume shrinkage with varying amount of cement, sand and polythene chips content compacted 2 % wet of OMC was then analyzed. In addition to this, various compositions of composite material were prepared and volume shrinkage behavior was assessed and the composition, which gave desired volume shrinkage was further tested for strength and permeability behavior.

Chapter 1 introduces the subject and describes the research objectives. Chapter 2 describes the review of literature pertaining to previous research on the subject and behavior and properties of compacted clays. This facilitates to establish the requirements of the ‘ compacted clay liner’ for arid/semi arid regions; permeability less than 1 x 10^{-9} m/s, volume change potential to be less than 4 % and strength about 200 Kpa (however it varies from project to project). Chapter 3 covers the methodology for the selection of soil for the research work. High plastic clay of Kashmore (south of Punjab) was selected and procured for the study. Chapter 4 describes the effect of variation of moisture content on strength, volume shrinkage and permeability of the compacted clays. The strength and volume shrinkage increased with increase in molding moisture content and the permeability further reduced. Permeability of High plastic clay was well with in limits even dry of optimum. Chapter 5 describes the effect of variation of sand, cement and polythene chips content on volume shrinkage potential of high plastic clays. 15 % sand reduced the shrinkage potential of the clay significantly whilst even 60 % of sand content did not decrease the shrinkage potential much. Cement content alone also did not give desired results and 3 to 9 % cement content gave volume shrinkage between 6 and7 %. Polythene chips of three different sizes were tried but none of them gave encouraging results, however 0.5x 1.0 cm sized chips gave a clear trend of variation of volume change with increase in chips content reducing it up to 6.5 %. Chapter 6 describes the effect of various compositions of composite clay consisting High plastic clay, sand, cement and polythene chips on volume change potential of clay. In this the results obtained from the unconfined compression test and permeability results from consolidation test for the combination meeting the volume shrinkage criterion are also described. Chapter 7 gives detailed discussion on the results obtained from the tests. Field Methodology to use the results obtained from this research is described in Appendix ‘B’.

Salient conclusions and recommendations in the light of detailed analysis and discussion on the results are enumerated in the subsequent paragraphs.

8.2 Salient Conclusions

a. For compacted clay liners, high plastic clay should be compacted at low moisture contact and modified compactive effort.

b. Optimum moisture content for maximum dry density of Kashmore soil i.e., high plastic clay only is 15 % and test parameters range, against moisture content ranging from 13 to 17 %, is as follow: -

	Test Parameter	Result Range
(1)	Volume shrinkage	5-7 %
(2)	UCC strength	271-163 KN/m^2
(3)	Permeability	0.00167-0.0007 x 10^{-9} m/s

(Except volume shrinkage, both strength and permeability are within Specified limit)

c. Volume shrinkage of soil mixed with sand (15 – 60%) was ranging from 6 to 8 %.

d. Addition of cement (3 – 9%) in high plastic clay gave volume shrinkage from 6 to 7 %.

e. Addition of varying sizes and proportions of polythene chips had following effects on volume shrinkage:-

Size of Chips	Range of Proportion	Test Results
(1) 0.5 x 0.5 cm	0.5 to 1.0 %	10 to 9 %
(2) 0.5 x 0.1 cm	0.5 to 1.0 %	9 to 7 %
(3) 1.0 x 1.0 cm	0.5 to 1.0 %	8 %

f. Volume shrinkage test results from samples of composite material with polythene chips (0.5 – 1.0%), Sand (15-30%) and Cement (3 – 6%) were from 5 to 3 %.

g. Sample having 30% sand, 6% cement and 0.5 to 1.0% polythene chips had 3 to 3.5% volume shrinkage, 368 to 473 Kpa as UCC strength and 0.0785×10^{-9} to 0.0114×10^{-9} m/s permeability. Composite sample with 0.5 chips, 30% sand and 6% cement in high plastic clay met the criterion for compacted clay liner. The results were, volume shrinkage 3%, UCC strength 368 Kpa and permeability 0.0152×10^{-9} m/s.

8.3 Recommendations

a. For arid / semi arid regions, high plastic clay, if used as clay liner, should be compacted dry of /or at OMC using modified compactive effort.

b. Instead of pure/untreated high plastic clay, composite soil having high plastic clay mixed with 30% sand, 6% cement and 0.5 % polythene chips of 0.5 x 0.5 cm size should be used.

8.4 Recommendations for Future Research

a. Study should be carried out to evaluate the behavior of high plastic clay compacted dry of optimum moisture content.

b. Study should also be carried out on composite soil having high plastic clay mixed with sand and cement only to evaluate its behavior in volume shrinkage, strength and permeability tests.

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