EFFECT OF SOIL STABILIZERS ON THE STRUCTURAL DESIGN OF FLEXIBLE PAVEMENTS

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DECLARATION

The author hereby declares that this thesis entitled **"EFFECT OF SOIL STABILIZERS ON THE STRUCTURAL DESIGN OF FLEXIBLE PAVEMENTS"**, in whole or part, has not been used to obtain any degree in this, or any other institute, except where references have been given in text. It is entirely the authors own work. The author confirms that the library may lend or copy this thesis upon request for academic purposes.

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CERTIFICATE

This is to certify that the thesis entitled **"Effect Of Soil Stabilizers On The Structural Design Of Flexible Pavements"**, being submitted by **Mr. NAVTAJ SINGH DHANOA, Roll No 801123005** in partial fulfilment for the award of degree of **Masters of Engineering in Civil - Infrastructure Engineering** at **Thapar University, Patiala** is a bonafide work carried out by him under our guidance and supervision and that no part of this thesis has been submitted for the award of any other degree.

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ABSTRACT

Poor subgrade soil conditions can result in inadequate pavement support and reduce pavement life. Soils may be improved through the addition of chemical or cementitious additives. These chemical additives range from waste products to manufactured materials and include lime, Class C fly ash, Portland cement, cement kiln dust, RBI Grade 81. These additives can be used with a variety of soils to help improve their native engineering properties. The effectiveness of these additives depends on the soil treated and the amount of additive used.

Design of the various pavement layers is very much dependent on the strength of the subgrade soil over which they are going to be laid. The subgrade strength is mostly expressed in terms of California Bearing Ratio (CBR). Weaker subgrade essentially requires thicker layers whereas stronger subgrade goes well with thinner pavement layers. The pavement and the subgrade mutually must sustain the traffic volume. The Indian Road Congress (IRC) encodes the exact design strategies of the pavement layers based upon the subgrade strength which is primarily dependant on CBR value for a laboratory or field sample soaked for four days. For an engineer, it's important to understand the change of subgrade strength. This project is an attempt to understand the strength of subgrade in terms of CBR values subjected to different types of stabilizers.

Treatment with cement and lime was found to be an effective option for improvement of soil properties, based on the testing conducted as a part of this work. It was found that with the addition of stabilizers i.e. cement and lime, the C.B.R. increased upto a certain limit but after that the C.B.R. decreased even on the further addition of stabilizers.

LIST OF CONTENTS

CONTENT

LIST OF TABLES

LIST OF FIGURES

Chapter 1
Introduction

1.1. Subgrade

Subgrade layer is the lowest layer in the pavement structure underlying the base course or surface course, depending upon the type of pavement. Generally, subgrade consists of various locally available soil materials that sometimes might be soft and/or wet that cannot have enough strength/stiffness to support pavement loading. A sound knowledge of performance of the subgrade soil under prevailing in-situ condition is necessary prior to the construction of the pavement. The better the strength/stiffness quality of the materials the better would be the long-term performance of the pavement. Hence, the design of pavement should be focused on the efficient, most economical and effective use of existing subgrade materials to optimize their performance. In case of soft and wet subgrades, proper treatment might be needed in order to make the subgrade workable for overlying layers (e.g., creating working platform) for pavement construction.

In the past, the strength quality of the subgrade soil used in pavement construction had been determined by various laboratory tests such as the California bearing ratio (CBR). However, neither of these methods considers the effect of cyclic loading of the vehicular load on the pavement due to static nature of their loading conditions. The recent development in pavement design includes the introduction of stiffness based modulus, called the resilient modulus, which deals with the repeated loading condition on the materials to be tested, thus simulating the actual vehicular loading in the field. The repeated loading triaxial test is performed within the elastic range of the soil in order to determine the resilient modulus. On the other hand, the permanent deformation deals with the cyclic loading of materials beyond the elastic limit or sometimes up to failure of the specimens in order to evaluate the rutting performance (single-stage tests) and different shakedown stages (or limits) of the materials (multi-stage tests). Despite the more precise results from resilient modulus and permanent deformation tests, some designers and contractors still prefer using CBR value or any other conventional method in the design of pavement rather than the use of resilient modulus due to associated low cost and lesser time compared to the repeated loading triaxial tests.

1.1.1. Desirable Properties of Subgrade Soil

The advantageous properties of sub grade soil as a highway material are

- Stability
- Incompressibility
- Permanency of strength
- Minimum changes in volume due to climate
- Superior drainage, and
- Ease of compaction

1.1.2. Objectives

The objective of this work can be summarized as follows:

i. The CBR test was performed on the soil samples and value of CBR was recorded. ii.

The different kind of soil stabilizers were added in these soil samples and the CBR test was performed again.

iii. Cement and Lime were added in the soils for the purpose of stabilization and the changes in the value of CBR were recorded.

iv. Subsequently, on the basis of results obtained, the savings in crust thicknesses were calculated.

1.2. Soil Subgrade Stabilization

Soil Subgrade Stabilization has proved to be very economical as it provides cheap materials for the construction of low cost roads. Local materials can be used effectively. There are many techniques of soil stabilization. Cement stabilization is an important method of stabilization. It has proved very much effective in case of sandy soil due to the ease of pulverization and mixing and the smaller quantity of cement required. Cement stabilization refers to stabilizing soils with Portland cement. The primary reaction is with the water in the soil that leads to the formation of a cementitious material. These reactions occur almost independently of the nature of the soil and for this reason Portland cement can be used to stabilize a wide range of materials. Although there are several types of cement stabilized soils, there are two types associated with highway construction.

Soil stabilization occurs when lime, fly ash, cement or bentonite clay is added to a reactive soil. The resulting pozzolanic reaction between these materials and the soil develops a durable and stable bond between molecules in the soil. This reaction can provide for long lasting stabilization of clay based soils.

Soil Stabilization is a simple process involving in-place mixing where an appropriate amount of lime, fly ash, cement or bentonite clay is spread over the ground surface, mixed to an appropriate depth. Pulverization by our mixers thoroughly combines the lime and soil to depths of 12 to 18 inches. For heavy clays, it is typical to complete a preliminary mixing, spreading lime and passing over the entire area, followed by 24 to 48 hours (or more) of moist curing. This is followed by a second spreading of fly ash or lime, followed by final mixing. During the final mixing faze the soil is compacted to develop the proper and intended soil strength and durability.

1.2.1. Mechanisms of Stabilization

The stabilization mechanism may vary widely from the formation of new compounds binding the finer soil particles to coating particle surfaces by the additive to limit the moisture sensitivity. Therefore, a basic understanding of the stabilization mechanisms involved with each additive is required before selecting an effective stabilizer suited for a specific application. Chemical stabilization involves mixing or injecting the soil with chemically active compounds such as Portland cement, lime, fly ash, calcium or sodium chloride or with viscoelastic materials such as bitumen. Chemical stabilizers can be broadly divided in to three groups: Traditional stabilizers such as hydrated lime, Portland cement and Fly ash; Non-traditional stabilizers comprised of sulfonated oils, ammonium chloride, enzymes, polymers, and potassium compounds; and By-product stabilizers which include cement kiln dust, lime kiln dust etc. Among these, the most widely used chemical additives are lime, Portland cement and fly ash. Although stabilization with fly ash may be more economical when compared to the other two, the composition of fly ash can be highly variable. The mechanisms of stabilization of the different stabilizers are detailed below.

1.2.2. Materials

Materials used to stabilize soils including lime, fly ash and cement, are strong alkali and a caustic material. They can burn the skin and are considered dangerous to the eye.

I. **Lime**: It is prepared by decomposing limestone at elevated temperatures. Lime-soil reactions are complex and primarily involve a two step process. The primary reaction involves cation exchange and flocculation/agglomeration that bring about rapid textural and plasticity changes (*2*). The altered clay structure, as a result of flocculation of clay particles due to cation exchange and short-term pozzolanic reactions, results in larger particle agglomerates and more friable and workable soils. Although pozzolanic reaction processes are slow, some amount of pozzolanic strength gain may occur during the primary reactions, cation exchange and flocculation/agglomeration. Extent of this strength gain may vary with soils depending on differences in their mineralogical composition. Therefore, mellowing periods, normally about one-day in length but ranging up to about 4-days, can be prescribed to maximize the effect of short term reactions in reducing plasticity, increasing workability, and providing some initial strength improvement prior to compaction. The second step, a longer-term pozzolanic based cementing process among flocculates and agglomerates of particles, results in strength increase which can be considerable depending on the amount of pozzolanic product that develops, and this, in turn depends on the reactivity of the soil minerals with the lime or other additives used in stabilization. The pozzolanic reaction process, which can either be modest or quite substantial depending on the mineralogy of the soil, is a long term process. This is because the process can continue as long as a sufficiently high pH is maintained to solubilize silicates and aluminates from the clay matrix, and in some cases from the fine silt soil. These solubilized silicates and aluminates then react with calcium from the free lime and water to form calcium-silicate-hydrates and calciumaluminate-hydrates, which are the same type of compounds that produce strength development in the hydration of Portland cement. However, the pozzolanic reaction process is not limited to long term effects. The pozzolanic reaction progresses relatively quickly in some soils depending on the rate of dissolution from the soil matrix. In fact, physio-chemical changes at the surface of soil particles due to pozzolanic reactions result in changes in plasticity, which are reflected in textural changes that may be observed relatively rapidly just as cation exchange reactions are. Lime stabilization refers to the process of adding burned limestone products either calcium oxide (i.e. quicklime) or calcium hydroxide (Ca $(OH)_2$) to soil in order to improve its properties. This process is similar to cement stabilization except that according to Bell (1993); lime stabilization is suitable for soils with high clay contents. Lime was used throughout the world by the ancient civilization as a binding agent for brick and stone.

II. **Portland cement**: It is comprised of calcium-silicates and calcium-aluminates that hydrate to form cementitious products. Cement hydration is relatively fast and causes immediate strength gain in stabilized layers. Therefore, a mellowing period is not typically allowed between mixing of the components (soil, cement, and water) and compaction. In fact it is general practice to compact soil cement before or shortly after initial set, usually within about 2 hours. Unless compaction is achieved within this period traditional compaction energy may not be capable of developing target density. However, Portland cement has been successfully used in certain situations with extended mellowing periods, well beyond 2 to 4 hours.

Generally, the soil is remixed after the mellowing periods to achieve a homogeneous mixture before compaction. Although the ultimate strength of a soil cement product with an extended mellowing period may be lower than one in which compaction is achieved before initial set, the strength achieved over time in the soil with the extended mellowing period may be acceptable and the extended mellowing may enhance the ultimate product by producing improved uniformity. Nevertheless, the conventional practice is to compact soil cement within 2 hours of initial mixing. During the hydration process, free lime, $Ca(OH)_2$ is produced. In fact up to about 25 percent of the cement paste (cement and water mix) on a weight basis is lime. This free lime in the high pH environment has the ability to react pozzolanically with soil, just as lime does and this reaction continues as long as the pH is high enough, generally above about 10.5. Cement stabilization involves the addition of small amount of cement to modify the soil properties. The amount of cement needed to stabilize soil may range from 3 to

16% by dry weight of soil, depending on the soil type and properties required. Any type of cement may be used for soil stabilization but ordinary Portland cement is mostly used according to Bell (1993).

III. **RBI Grade-81**: RBI Grade-81 is a unique, cost-effective, environment friendly technological breakthrough in soil stabilization, waste binding and pavement layer design for the road and highway building world. RBI Grade-81 is a unique and highly effective natural inorganic soil stabilizer for infrastructure development and repair. RBI Grade-81 meets the requirement for a well-proven, reliable and very cost-effective method by creating a strong and irreversible impermeable layer which is resistant to adverse climatic conditions, from very high temperatures to permafrost conditions, and accommodating all types of roads and load requirements. RBI Grade-81 is environment friendly and emphasizes the use of recycled material, recognizing the lack of readily available resources. It reduces the Carbon Footprint of any project by reducing transportation requirements and carbon emissions. This makes it eligible for Carbon Credits in the environment friendly sensitive global marketplace.

RBI Grade-81 was originally developed by RBI for South African Army Road Building International for the in the beginning of 1990's for pavement engineering applications. RBI Grade-81 is a natural inorganic soil-stabilizer which re-engineers & modifies the properties of soil to strengthen it for roads, paving and roads and pavement. Alchemist Technology is the exclusive manufacturer and distributor of RBI Grade-81 in India.RBI Grade-81 is patented worldwide including India.

The Benefits of RBI Grade-81 are that it:

- Reduces time of construction by up to 40%
- Drastically increases the strength of roads
- Makes soil water-resistant & prevents damage to road foundations.
- Reduces the requirement of Aggregates
- Reduces transport & earth-moving costs by up to 60%
- Has longer durability

Figure 1.1: Laying of RBI Grade-81 (Source www.rbigrade81.com)

- Reduces maintenance costs.
- Has a small Carbon Footprint and is environment friendly

IV. **Fly ash**: It is also generally considered as a traditional stabilizer. While lime and Portland cement are manufactured materials, fly ash is a by-product from burning coal during power generation. As with other by-products, the properties of fly ash can vary significantly depending on the source of the coal and the steps followed in the coal burning process. These by-products can broadly be classified into class C (self-cementing) and class F (non-self cementing) fly ash based on AASHTO M 295 (ASTM C 618). Class C fly ash contains a substantial amount of lime, CaO, but almost all of it is combined with glassy silicates and aluminates. Therefore upon mixing with water, a hydration reaction similar to that which occurs in the hydration of Portland cement occurs. As with Portland cement, this hydration reaction produces free lime. This free lime can react with other unreacted pozzolans, silicates and aluminates, available within the fly ash to produce a pozzolanic reaction, or the free lime may react pozzolanically with soil silica and/or alumina. Class F ash, on the other hand, contains very little lime and the glassy silica and/or alumina exists almost exclusively as pozzolans. Therefore, activation of these pozzolans requires additives such as Portland cement or lime, which provide a ready source of free lime. The hydration or “cementitious” reactions and the pozzolanic reactions that occur when fly ash is blended with water forms the products that bond soil grains or agglomerates together to develop strength within the soil matrix. As discussed previously, maintenance of a high system pH is required for long term strength gain in fly ash-soil mixtures.

1.3 Subgrade Stability

Subgrade stability is a function of a soil's strength and its behaviour under repeated loading. Both properties significantly influence pavement construction operations and the long-term performance of the pavement.

The subgrade should be sufficiently stable to:

1. Prevent excessive rutting and shoving during construction;
2. Provide good support for placement and compaction of pavement layers;
3. Limit pavement rebound deflections to acceptable limits; and
4. Restrict the development of excessive permanent deformation (rutting) in the sub grade during the service life of the pavement. When the subgrade does not possess these attributes, corrective action in the form of a subgrade treatment is needed.

1.4. Modern Methods

The life of road depends on strength of the subgrade soil and traffic density. The subgrade soil is not uniform throughout the alignment of the road. Generally the poor subgrade soil having soaked California Bearing Ratio (CBR) value less than 2% is replaced by good quality subgrade material. The additive like RBI Grade 81is used to improve the properties of subgrade soil. The cost of construction of road increases, if only RBI Grade 81 is used as a stabilizer. The CBR value of subgrade soil can be improved by using moorum with RBI Grade 81and cost of construction can be reduced to certain extent. From CBR test, it is found that the soaked CBR value of soil is improved by 476.56% i.e. 2.56% to 14.76% by stabilizing soil with 20% moorum and 4% RBI Grade 81. The various mixes of soil: moorum: RBI Grade 81 for the different proportions were tested for maximum dry density (MDD), optimum moisture content (OMC) and soaked CBR value.RBI Grade-81 has been invented to provide comprehensive and irreversible soil stabilization specifically for road construction. Treated soil is water resistant & prevents damage to the road foundation and provides better ride-ability & longer durability leading to reduced pavement maintenance cost.

1.5. Weak subgrade and treatment

The subgrade in flexible pavement is more vulnerable to failure under the vehicular traffic loading due to non- uniform distribution of the load from overlying layers and the presence of high moisture contents. This layer gets less emphasis compared to other layers in pavement, despite the fact that most of the pavement failure is being caused due to the bearing capacity failure of the subgrade layer. Some subgrade soils, especially clayey soils, have great strength at low moisture content; however they become very weak and less workable with the increase in water content beyond the optimum value. Such soil should be either replaced with superior quality fill material or treated with suitable treatment process (Prusinski and Bhattacharja, 1999). The replacement of the subgrade soil might not always be the best option due to associated hauling cost of the excavated materials as well as the imported quality materials. In some developing regions or even urban areas, the unviability of the aggregate or the shortage of the suitable fill materials makes replacement of weak subgrade soil uneconomical. In such conditions, the strength/stiffness properties of the existing weak subgrade soil can be improved by the use of proper compaction technique as well as by using some chemical stabilizers. Portland cement, lime and fly ash are the most common types of chemical stabilizers used by most of states to stabilize the weak subgrades; thus creating a proper working platform and/or subbase layer for pavement construction.

1.6. Flexible Pavement

Flexible pavements are so named because the total pavement structure deflects, or flexes, under loading. A flexible pavement structure is typically composed of several layers of material. Each layer receives the loads from the above layer, spreads them out and then passes these loads to the next layer below. Typical flexible pavement structure shown in plate 1.1 consisting of:-

a) Surface course: This is the top layer and the layer that comes in contact with traffic. It may be composed of one or several different HMA sub-layers. HMA is a mixture of coarse and fine aggregates and asphalt binders with or without additives.

b) Base course: This is the layer directly below the HMA layer and generally consists of aggregate (either stabilized or un-stabilized).

c) Sub-base course: This is the layer (or layers) under the base layer. A sub-base is not always needed.

d) Subgrade course: The subgrade is the material upon which the pavement structure is placed. Although there is a tendency to look at pavement performance in terms of pavement crust structure material, mix design and thickness but the sub-grade can often be the overriding factor in the overall pavement performance. The CBR value of the subgrade material is generally used to design the total pavement crust thickness as per IRC: 37-2012 guidelines.

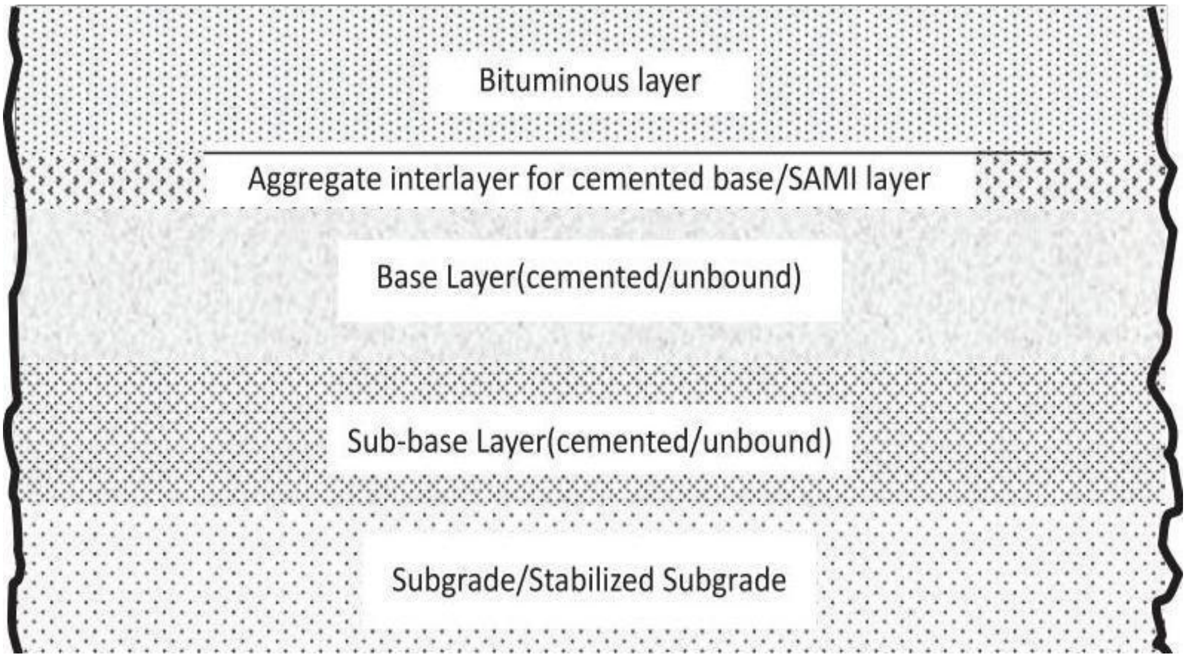

Figure 1.2: Different layers of the Flexible Pavement (Source: IRC: 37-2012)

A flexible pavement structure is typically composed of several layers (as shown in Figure no 1.1) of material with better quality materials on top where the intensity of stress from traffic loads is high and lower quality materials at the bottom where the stress intensity is low. Flexible pavements can be analyzed as a multilayer system under loading and are constructed by using different layers such as Bituminous concrete (BC), Dense Bituminous Macadam (DBM), Bituminous Macadam (BM), Wet Mix Macadam (WMM) and Granular Subbase (GSB) as per the MORTH specifications with the designed thickness as per the IRC: 37-2012 have shown in Figure 1.3.

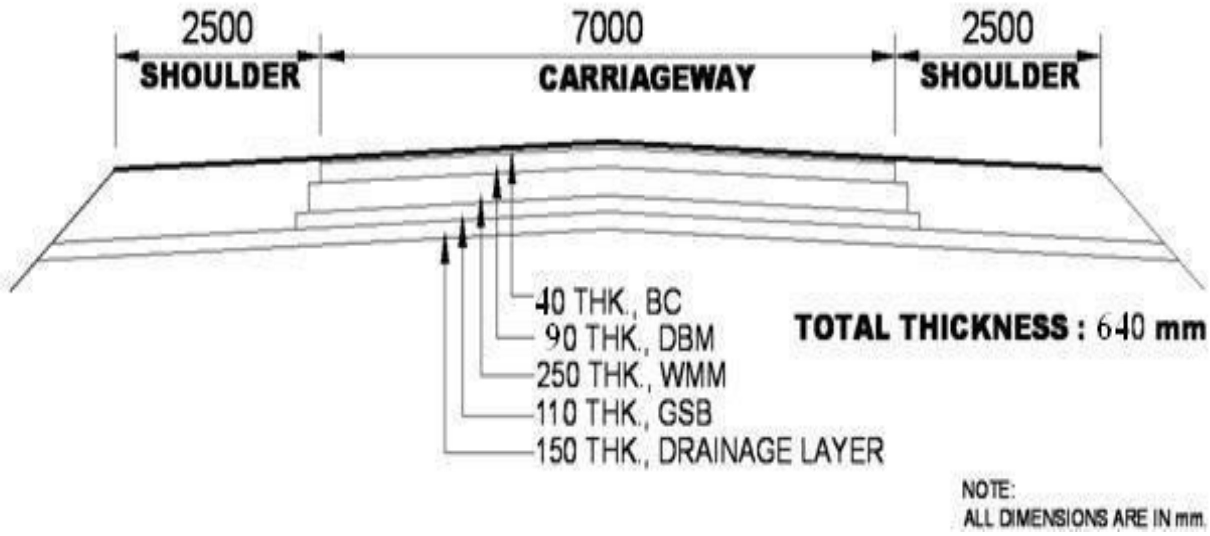

Figure 1.3: - Section of Flexible Pavement designed as per IRC: 37-2012

Chapter 2
Literature Review

2.1. General

Soil Stabilization has proved to be very economical as it provides cheap materials for the construction of low cost roads. Local materials can be used effectively. There are many techniques of soil stabilization. Cement stabilization is an important method of stabilization. it has proved very much effective in case of sandy soil due to the ease of pulverization and mixing and the smaller quantity of cement required. Cement stabilization refers to stabilizing soils with Portland cement. The primary reaction is with the water in the soil that leads to the formation of a cementitiuos material.

2.2. Use of Cement As Stabilizer

Little (1995): Laboratory testing indicates that cement reacts with medium, moderately fine, and fine-grained soils to produce decreased plasticity, increased workability, and increased strength. Strength gain is primarily due to the chemical reactions that occur between the cement and soil particles. These chemical reactions occur in two phases, with both immediate and long-term benefits. The first phase of the chemical reaction involves immediate changes in soil texture and soil properties caused by cation exchange. The free calcium of the lime exchanges with the adsorbed cations of the clay mineral, resulting in reduction in size of the diffused water layer surrounding the clay particles. This reduction in the diffused water layer allows the clay particles to come into closer contact with one another, causing flocculation/agglomeration of the clay particles, which transforms the clay into a more silt-like or sand-like material. Overall, the flocculation and agglomeration phase of lime stabilization results in a soil that is more readily mixable, workable, and, ultimately, compactable.

2.3. Use of Cement Kiln Dust as Stabilizer

Robert L. Parsons (2004): Poor subgrade soil conditions can result in inadequate pavement support and reduce pavement life. Soils may be improved through the addition of chemical or cementitious additives. These chemical additives range from waste products to manufactured materials and include lime, Class C fly ash, Portland cement, cement kiln dust from pre-calciner and long kiln processes, and proprietary chemical stabilizers. These additives can be used with a variety of soils to help improve their native engineering properties. The effectiveness of these additives depends on the soil treated and the amount of additive used. This report contains a summary of the performance of a wide range of soils treated with pre-calciner cement kiln dust (CKD), and is intended to be viewed as a companion report to the previously published Kansas Department of Transportation report, Performance of Soil Stabilization Agents. CKD has been used as a soil additive to improve the texture, increase strength and reduce swell characteristics. CKD was combined with a total eight different soils with classifications of CH, CL, ML, SM, and SP. Durability testing procedures included freeze-thaw, wet-dry, and leach testing. Atterberg limits and strength tests were also conducted before and after selected durability tests. Changes in pH were monitored during leaching. Relative values of soil stiffness were also tracked over a 28-day curing period using the soil stiffness gauge. Treatment with cement kiln dust was found to be an effective option for improvement of soil properties, based on the testing conducted as a part of this research. Strength and stiffness were improved and plasticity and swell potential were substantially reduced. Durability of CKD treated samples in wet-dry testing was comparable to that of soil samples treated with the other additives, while performance was not as good in freeze thaw testing. CKD treated samples performed very well in leaching tests and in many cases showed additional reductions in plasticity and some strength gains after leaching. It is recommended based on the results of this research that cement kiln dust be considered a viable option for the stabilization of subgrade soils. As with all additives, it is recommended that a mix design be conducted prior to selection to confirm the CKD selected and the amount specified will provide satisfactory performance.

2.4. Use of Lime as Stabilizer

Eades and Grim (1960), described that practically all 4 fine-grained soils undergo this rapid cation exchange and flocculation/agglomeration reactions when treated with lime in the presence of water. The second phase of the chemical reaction involves pozzolanic reactions within the lime-soil mixture, resulting in strength gain over time. When the cement is combined with a sandy soil, the pH of the pore water increases. When the pH reaches 12.4, the silica and alumina from the clay become soluble and are released from the clay mineral. In turn, the released silica and alumina react with the calcium from the lime to form cement, which strengthens in a gradual process that continues for several years. As long as there is sufficient calcium from the lime to combine with the soluble silica and alumina, the pozzolanic reaction will continue as long as the pH remains high enough to maintain the solubility of the silica and alumina. Strength gain also largely depends on the amount of silica and alumina available from the clay itself; thus, it has been found that cement stabilization is more effective for montmorillonitic soils than for kaolinitic soils (Lees et. al, 1982). In addition to pozzolanic reactions, carbonation can also lead to long-term strength increases for soils stabilized with lime. Carbonation occurs when lime reacts with carbon dioxide from the atmosphere to produce a relatively insoluble calcium carbonate. This can be advantageous since after mixing, the slow process of carbonation and formation of cementitious products can lead to long-term strength increases (Arman and Munfakh, 1970). However, prior to mixing, exposure of cement to air should be avoided through proper handling methods and expedited construction procedures in order to avoid premature carbonation of the lime (Chou, 1987).

2.5. Use of Calcium Rich Chemicals as Stabilizer

Oklahoma DOT (2009): The study shows that the Soil stabilization or modification refers to the improvement of the soil physically or chemically by using various techniques including mechanical compaction and the use of various calcium rich chemicals. The selection of proper stabilization technique depends on the soil type and its condition. Mechanical stabilization is best suited for coarse grained soils or aggregates at optimum or below optimum moisture contents. However, clayey soils are more effective under chemical stabilization. If the clayey soil is mixed with the specific stabilizer just enough to make it workable, better in texture and compatibility regardless the strength and durability, then it is referred to as modification (Indiana DOT, 2002); modification is restricted to the soil having AASTHO designation A-4, A-5, A-6 and A-7 . On the other hand, stabilization refers to the selection of the stabilizer in order to achieve certain target strength/stiffness values in addition to modification. In conclusion, creating working platform for construction purpose only is part of modification/treatment; whereas stabilization is essential if we are dealing with construction of sub base in pavements.

2.6. Use of Fly Ash as Stabilizer

Louisiana standard specification (2006): Various states established their own criteria for modification and stabilization. The LADOTD recommends the criteria for the selection of the stabilizer based on the soil characteristic. Furthermore, the Texas DOT has a wide range of selection of stabilizers for subgrade and subbase soils, which describes the selection of various stabilizers based on the properties of the subgrade soils. Fly ashes produced by power plants in the United States occasionally contain significant amounts of unburned carbon due to common use of low nitrogen-oxide and sulphur-oxide burners in recent years. This ash cannot be reused in concrete production due to its reactivity with air entrainment admixtures and is being land filled at large percentages. A study was conducted to stabilize low stiffness road surface material with high carbon fly ash. The non-cementitious fly ash was activated with another recycled material, lime kiln dust (LKD). California bearing ratio (CBR) and resilient modulus tests were conducted to determine the strength and stiffness, respectively, of the stabilized materials. Addition of LKD and curing of specimens generally increased CBR and summary resilient modulus (SMR), and lowered plastic strains, CBR increased with increasing CaO content as well as with $CaO/SiO2$ and $CaO/(SiO2 + Al2O3)$ ratio of the mixtures; however, these parameters could not be correlated with the SMR. The unpaved road materials stabilized with LKD and fly ash are expected to lose 31–67% of their initial moduli after 12 cycles of freezing and thawing. Lower base thicknesses and reduction in construction costs can be expected by stabilizing road surface materials with high carbon fly ash.

Cetin Bora et. al. (2010): Roadways are one of the largest construction fields, and reuse of suitable waste materials in their construction can provide significant cost savings while meeting the objectives of the United States Federal Highway Administration Green Highways Partnerships initiative. A laboratory study was conducted to investigate the feasibility of reusing chemically stabilized road surface material in construction of highway bases. Non-cementitious off-spec high carbon fly ash was activated with lime kiln dust and used to stabilize an unpaved road material (URM) collected from Maryland. The effects of lime kiln dust (LKD) and fly ash addition, and curing time on strength and stiffness of highway bases were studied. The effects of winter conditions on stiffness were examined by performing resilient modulus tests on the specimens after a series of freeze–thaw cycles. The base thicknesses were calculated for all mixture designs by using their CBR and summary resilient moduli (SMR) values.

S. Kolias (2004): The effectiveness of using high calcium fly ash and cement in stabilising fine-grained clayey soils (CL,CH) was investigated in the laboratory. Strength tests in uniaxial compression, in indirect (splitting) tension and flexure were carried out on samples to which various percentages of fly ash and cement had been added. Modulus of elasticity was determined at 90 days with different types of load application and 90-day soaked CBR values are also reported. Pavement structures incorporating subgrades improved by in situ stabilisation with fly ash and cement were analyzed for construction traffic and for operating traffic. These pavements are compared with conventional flexible pavements without improved subgrades and the results clearly show the technical benefits of stabilising clayey soils with fly ash and cement. In addition TG–SDTA and XRD tests were carried out on certain samples in order to study the hydraulic compounds, which were formed. This work shows that the potential benefit of stabilising clayey soils with high calcium fly ash but this depends on the type of soil, the amount of stabilising agent and the age. The study of the formation of the hydraulic products during the curing of clay containing as a stabilising agent high calcium fly ash shows that a significant amount of tobermorite is formed leading to a denser and more stable structure of the samples. A further addition of cement provides better setting and hardening and the combination of these two binders can increase the early as well the final strength of the stabilised material. The free CaO of fly ash reacts with the clay constituents (SiO2 and the other aluminium silicates) leading to the formation of tobermorites and calcium aluminium silicate hydrates as well.

Misra Anil (2004): Self-cementing class C fly ashes are being increasingly used for soil stabilization of road bases and in other civil constructions. Because of their self-cementing capability in the presence of water, they can be used for clay subgrade improvement as cement surrogates, or as road subgrade material. However, for efficient and economic utilization of self-cementing class C fly ash, the physico-mechanical characteristics of these ashes must be determined extensively. This paper focuses upon the laboratory evaluation of the (1) stabilization characteristics of clay soils blended with self-cementing class C fly ash, and (2) residual self-cementation capabilities of ponded class C fly ash. Testing carried out by the authors and other researchers have indicated that curing time, curing condition, clay mineralogy, amount of fly ash and swelling potential in the soil-fly ash mix are the important variables that control stabilization characteristics. In this paper, the stabilization characteristics were evaluated in terms of the gain in the uniaxial compressive strength and stiffness, and swelling potential. To examine these effects, 12 set of mixtures of ideal clay soils with known percentages of kaolinite and montmorillonite, self-cementing class C fly ash and appropriate amount of water were compacted and cured. In the mixed samples, amount of montmorillonite varied from 0, 2, 4 and 6%, and the amount of self-cementing class C fly ash varied from 5, 10 and 20%. To investigate the effect of curing condition, three curing environments were used. For swelling test, the cured samples were inundated and allowed to swell at the seating pressure of about 2 KPa applied by the weight of the top porous stone and load plate using the one dimensional odometer apparatus. In addition to the stabilization characteristics of clay soils-fly ash blend, the residual self-cementation capabilities of ponded class C fly ash were also investigated in terms of unconfined compression and CBR tests performed at 7 and 14 days of curing. Results obtained from these test were encouraging and compared favorably with the typical subgrade materials.

2.7. Use of Geo-Textiles As Stabilizer

Aiban. S.A. et. al. (2005): Many construction and post-construction problems have been reported in the literature when sabkha soils have been used without an understanding of their abnormal behaviour, especially their inferior loading capability in their natural conditions. The strength of these soils can be further significantly decreased if the sabkha is soaked. The main objective of this study was to upgrade the load carrying capacity of pavements constructed on sabkha soils using geo-textiles, and to assess the effect of geo-textile grade, base thickness, loading type (static and dynamic) and moisture condition (as-moulded and soaked) on the performance of soil-fabric aggregate (SFA) systems. In addition, the sabkha soil was treated with different dosages (5%, 7%, and 10%) of Portland cement and the performance of cement-stabilized sabkha was compared to that of the SFA system under different testing conditions. The ANOVA results indicated that the use of geo textile has a beneficial effect on sabkha soils, especially under wet conditions. Although the improvement in the load-carrying capacity of sabkha samples with high dosages of cement showed better results than the inclusion of geotextile, an economic analysis showed that the use of geotextiles would be superior. Moreover, mechanistic analysis was used to develop a prediction model for the percentage increase in the modulus of resilience. This investigation was conducted to assess the effect of geotextile on the performance of sabkha soil. Several parameters were investigated, including the effects of soaking, geotextile grade, subbase thickness and loading condition. In addition, the effect of cement addition on the improvement of sabkha was studied and compared with the performance of SFA systems.

2.8. Use of Fibre Reinforcement As Stabilizer

Chauhan M.S. et. al. (2008): In this paper, the effectiveness of fibre reinforcement (coir fibre and synthetic fibre) in subgrade soil has been studied from the point of view of strength. The permanent strain, resilient strain behaviour and resilient modulus of subgrade soil have been determined in the laboratory. A value of 10% (20 mm) strain is taken as the failure criterion for the subgrade for pavement in rural area. A sub grade soil of silty sand mixed with optimum content of fly ash and two different types of fibres varying in their tensile strength and coefficient of frictions were used. Repeated tri axial tests on samples, unreinforced and reinforced at the optimum content of fibre, were carried out at a confining pressure of 25, 50 and 75 KN/m2 and the stress levels of 153 and 204 kN/m2, producing six different deviator stresses. It is concluded from this study that both the permanent and resilient strains in all materials decrease with confining pressure but increase with the number of load cycles and deviator stress in reinforced and unreinforced conditions. Further, the resilient modulus decreases with the number of load cycles and deviator stress and increases with the confining pressure. Coir fibre shows better resilient response against synthetic fibre by higher coefficient of friction. Fly ash is also used in this study and for maximum dry density, the 30% fly ash and 70% sand mix is tested for various parameters.

Mekkawy M. Mohamed et. al. (2010): A recently completed field study in Iowa showed that many granular shoulders overlie clayey sub grade layer with California Bearing Ratio (CBR) value of 10 or less. When subjected to repeated traffic loads, some of these sections develop considerable rutting. Due to costly recurring maintenance and safety concerns, the authors evaluated the use of biaxial geo grids in stabilizing a severely rutted 310 m tests section supported on soft sub grade soils. Monitoring the test section for about one year, demonstrated the application of geo grid as a relatively simple method for improving the shoulder performance. The field test was supplemented with a laboratory testing program, where cyclic loading was used to study the performance of nine granular shoulder models. Each laboratory model simulated a granular shoulder supported on soft sub grade with geo grid reinforcement at the interface between both layers. Based on the research findings, a design chart correlating rut depth and number of load cycles to sub grade CBR was developed. The chart was verified by field and laboratory measurements and used to optimize the granular shoulder design parameters and better predict the performance of granular shoulders. The possibility of realistic prediction of two-layer sub grade load-settlement characteristics is discussed. The case of improvement of the soft sub grade properties using the geo synthetic reinforcement placed at the boundary between two different subgrade layers is analysed.

Krystyna Kazimierowicz-Frankowska (2007): In the first part of the paper, a short review of the main conclusions from experimental results dealing with the influence of geosynthetic reinforcement on the load-settlement characteristics of sub grade is presented. Then, the results of using the selected analytical membrane action model to describe the reinforcement action in soil are discussed. The model is verified on the basis of data obtained from previously published laboratory tests. Particular attention is devoted to influencing some basic initial parameters on the accuracy of obtained results. Important problems which need intensive investigations are identified.
The limitations and advantages of a model, based on a membrane reinforcement mechanism, were examined to predict the loaded sub grade settlements. The comparison of the two-layer sub grade behaviour (with and without reinforcement) was also enclosed. The obtained results lead to the following practical conclusions: The load settlement characteristics of two-layer sub grade with or without the geo synthetic layer do not differ much for the initial range of settlements. Therefore, a common load–settlement curve approximates the behaviour of both kinds of structures sufficiently well for settlements lower than approximately 0.1 D/2B.

2.9 Pavement Layers with Chemical Stabilized Materials (IRC: 37-2012)

- Chemically stabilized soils and aggregates may include all kinds of stabilization such as cement, lime, lime-flyash, or their combination, proprietary chemical stabilisers, enzymes, polymers and any other stabilizer provided these meet the strength and durability requirements. While cement, lime, lime- flyash stabilized materials are well known for their strength, performance and durability, the commercially produced stabilizers should meet the additional requirements of leachability and concentration of heavy metals. Where stabilized materials are used in the pavement, only mechanized method of construction for laying and compaction should be used. The equipment should be capable of administering the design doses of stabilizer and quantity of water and producing a uniform and homogeneous mix. Such materials are also termed as cemented or cementitious materials.
- IRC:SP: 89-2010 Guidelines for soil and granular material stabilisation using cement, lime & fly ash' has very comprehensively described the entire process of stabilization. High strength chemically stabilized layer having a 7-day unconfined strength of 6- 12 MPa given Table 8 of IRC:SP-89-2010 is not recommended because of wide shrinkage and thermal cracks that may occur during the service. Cementitious materials having lower strength can be used in bases and sub-bases. Since a cementitious base is required to act as an important load bearing layer, a minimum strength of 4.5 MPa for bases of cement treated aggregate at seven days is necessary for long term durability as measured by Wetting and drying/freezing and thawing tests (BIS: 4332 (Part IV) - 1968). Specimens having strength of 5 MPa and above are generally stable under durability tests. Cement stabilised aggregate specimens should be stored in a moist curing room/ curing chamber undisturbed for seven days before tests. Modulus of rupture of cementitious bases may be taken as 20 per cent of the 28 day UCS (MEPDG) for flexure strength evaluation.
- Lime-Soil and Lime-flyash-aggregate mixes develop strength at a slow rate and strength for their acceptability should be determined at 28 days. These slow setting stabilizers develop fine cracks unlike cement treated materials in which the rate of strength gain is high. These binders need less water for

compaction which indirectly reduces shrinkage also. Long term strength of lime–soil or lime-flyash- soil mixtures can be determined by curing sealed samples at 400 C (as per ASTM D5102-09 'Standard Test Methods for Unconfined Compressive Strength of Compacted Soil-Lime- Mixtures'. Accelerated curing may be use to provide a correlation between between normal and accelerated curing strengths for the material-binder combination. Three day curing of lime of lime-flyash soil at 500C is found to be equivalent to about 33 to 38 days of moist curing at ambient temperature of about 300C (16). Some typical values of unconfined compressive strength and modulus of rupture of lime-flyash concrete suitable for cemented bases extracted from IRC: SP:20-2002' Rural road Manual' are given below:

S.No.	Proportion of Lime: Fly Ash and Coarse aggregate by Weight	Water Content, per cent by Weight of Mix	28 Day Strength in MPa	
			UCS	MR
1	1:2.0:2:5.25	10.0	6.9	1.48
2	1:2.0:2.7:6.3	11.0	7.5	1.48
3	1:1.5:2.25:5.25	9.7	7.5	1.48

Table 2.1: Typical values of UCS and MR

Extracted from IRC SP:20-2002'Rural Road Manual

The proportions may vary depending upon the quality of materials and laboratory tests are required to be done prior to construction to ensure that the materials have the minimum strength. Different trials are necessary to arrive at a good mix proportion for a base and a sub-base. Construction procedure is explained in IRC: SP: 20-2002. Even for lime-fly ash stabilized materials, flexural strength can be about 20 per cent of the UCS. Published literature also (16) shows that flexural strength may be as high as 35 to 40 per cent of the UCS of lime or lime-flyash stabilized soil. The recommendation of MEPDG (3) taking the flexural strength as 20 per cent of the UCS is very reasonable if laboratory data is not available. Long term strength gain for slow setting stabilizers can be considered in design.

- Laboratory tests: A number of laboratory tests such as flexure tests,direct tension test ,longitudinal resonant frequency test, indirect tensile strength test, direct compression test etc can be used to measure elastic modulus of cementitious material. Unconfined compression tests give high values of modulus. AUSTROADS recommend flexure load test since this is considered to closer to stress/strain in the cemented base layer caused by traffic loading. Relation between UCS and elastic modulus was recommended (14) as

  E (cemented base) = k x UCS

  UCS = Unconfined strength at 28 days, MPa; k = 1000 to 1250.

  E value of the cemented bases containing 4 to 6 per cent Ordinary Portaland and, slag or Pozzolonic cements is recommended as 5000 MPa. If field evaluation by FWD indicates higher modulus, fresh estimate of pavement life can be made.

  Poisson's ratio of the cemented layer may vary from 0.2 to 0.25. A value of 0.25 may be adopted. Stresses are not very sensitive to Poisson's ratio.

  Cemented granular sub-base may have cement from 2 to 4 per cent to get a 7-day strength of 1.5 to 3.0 MPa. Its modulus as determined in laboratory may range fom 2000 MPa to 3000 MPa. Since it forms the platform for the construction traffic, it cracks and cannot retain the initial modulus. A value of 600 MPa is recommended and its fatigue behaviour is not considered because of cracks. If the stabilized soil sub-bases have 7-day UCS values in the range 0.75 to 1.5 MPa, the recommended E value for design is 400 MPa. Field tests by FWD should be routinely done to collect data for obtaining pavement design parameters. Cement requirement for a given strength is much higher for soils than for granular materials. For the commercially available propriety cementing materials, the binder contents have to be determined from laboratory tests to meet the strength requirement.

- Cemented bases should be compacted in a single layer to a maximum compacted thickness of 200 mm. Layers laid at different times may not have the strength because of lack of interface bond unless special care is taken.

- After the construction, curing as recommended in IRC:SP:89-2010 must be done immediately to aid in development of strength and prevent drying shrinkage. Spraying of bitumen emulsion is a very effective method of curing. Exposing the compacted layer to sun damages the stabilised layer and it does not develop strengths as intended.
- Cemented layers normally develop transverse and longitudinal cracks due to shrinkage and thermal stresses during hydration and during the service life. Hence a layer of Stress Absorbing Membrane Interlayer (SAMI) of elastomeric modified binder (AUSTROADS 2004) is to be provided over the cemented base to resist reflection cracking. The rate of spread of the binder is about 2 litres/m^2 followed by light coating of aggregates of size 10mm to prevent pick up of the binder by the wheels of construction machinery. Geotextile seal and many other commercially available synthetic products available commercially have the promise to retard crack propagation in the bituminous layer. SAMI is not very effective if the crack opening is more than 3 mm.
- Another method of arresting the cracks from propagating to the upper bituminous layer is to provide an interlayer of good quality aggregates between the bituminous layer and the cemented base. The aggregate layer should extend beyond the cemented base by about 0.5 m so that moisture, if any, travels down to the porous sub-base. Wet Mix Macadam (WMM) meeting the IRC/MORTH specification can form a good aggregate interlayer. Being sandwiched between two stiff layers, the aggregate behaves as layer of high modulus under heavy load while its modulus is lower when lighter loads act. Priming and tack coating are required before laying of the bituminous layer. Use of 1 to 2 per cent bitumen emulsion in WMM is needed only when the construction traffic is likely to deform the compacted WMM requiring regrading. It does not improve the crack resistance of the aggregate layer. Thickness of 75 mm to 150 mm has been used for the inter layer by different organisations. The Guidelines proposes a thickness of 100 mm.
- Behaviour of cemented base after traffic associated cracking: The cemented layers may get numerous carcks due to fatigue cracking and its modulus may be reduced drastically from 5000 MPa to 500 MPa. Falling Weight Deflectometer can be a good tool to examine the condition of the cemented base at any time during

its service life. The bituminous layer has little tensile strain before cracking of the cemented base but the value becomes very high after the cracking of the base and fatigue failure of the bituminous layer also is imminent. If the thickness of the bituminous layer is higher 175 mm (14), the bituminous layer also has considerable remaining life and the total life of the pavement is the sum of the fatigue life of the cemented layer and that of the bituminous layer. If the sum of the thickness of bituminous layer and 75 per cent of the thickness of aggregate interlayer is greater than 175 mm (14), the pavement life is again equal to the fatigue lives of the cemented and that of the bituminous layers.

- Maximum size of the aggregate of the granular base and sub-base should be 53 mm for obtaining a homogenious mass in mechanised construction. Close graded granular sub- base of MORTH can be used in the construction of cemented bases and sub-bases while coarse graded granular sub-base with percent passing 0.075 mm sieve less than 2 per cent can be used as a porous cemented drainage layer laid above the coarse graded cemented sub-base. Grading 4 can also be used.
- Procedure for the mix design method for cementitious granular bases and sub-bases can be same as adopted for Dry Lean Concrete (MORTH) except that the cementitious stabilizer content is much lower due to low strength requirement. Optimum moisture content has to be determined by trial. Since maximum size of aggregates can be 53 mm, 150 mm cubes have to be made at different moisture content and compacted by vibratory hammer with square plates. Details are given in MORTH Specifications for determination of optimum moisture content, method of curing and evaluation of strength.
- For stabilized soils, even 50 mm diameter, 100 mm high samples can be used for UCS after curing. Beam size can be 50 mm x 50 mm x 300 mm for flexure tests after curing. Field condition may be simulated during the curing.

If the soil is modified by addition of small percentage of lime/cement/other stabilizers, CBR tests can be done to evaluate the quality of the modified soil. If the cement content is 2 per cent or higher, unconfined compression strength should be determined to determine the strength of the stabilized soil.

Chapter 3
Experimental Programme

3.1. Need and Scope

The California bearing ratio test is penetration test meant for the evaluation of subgrade strength of roads and pavements. The results obtained by these tests are used with the empirical curves to determine the thickness of pavement and its component layers. This is the most widely used method for the design of flexible pavement. This Chapter covers the laboratory method for the determination of C.B.R. of undisturbed and remoulded /compacted soil specimens, both in soaked as well as unsoaked state.

3.2. Planning and Organization

Equipments and tool required.

1. Cylindrical mould with inside dia 150 mm and height 175 mm, provided with a detachable extension collar 50 mm height and a detachable perforated base plate 10 mm thick.

2. Spacer disc 148 mm in dia and 47.7 mm in height along with handle.

3. Metal rammers. Weight 2.6 kg with a drop of 310 mm (or) weight 4.89 kg a drop 450 mm.

4. Weights. One annular metal weight and several slotted weights weighing 2.5 kg each, 147 mm in dia, with a central hole 53 mm in diameter.

5. Loading machine. With a capacity of atleast 5000 kg and equipped with a movable head or base that travels at an uniform rate of 1.25 mm/min. Complete with load indicating device.

6. Metal penetration piston 50 mm dia and minimum of 100 mm in length.

7. Two dial gauges reading to 0.01 mm.

8. Sieves. 4.75 mm and 20 mm I.S. Sieves.

9. Miscellaneous apparatus, such as a mixing bowl, straight edge, scales soaking tank or pan, drying oven, filter paper and containers.

Figure 3.1: California Bearing Ratio Testing machine & equipments

3.3. Definition of C.B.R.

It is the ratio of force per unit area required to penetrate a soil mass with standard circular piston at the rate of 1.25 mm/min. to that required for the corresponding penetration of a standard material.

C.B.R. = (Test load/Standard load) X 100

The following table gives the standard loads adopted for different penetrations for the standard material with a C.B.R. value of 100%

Penetration of plunger (mm)	Standard load (kg)
2.5	1370
5.0	2055
7.5	2630
10.0	3180
12.5	3600

Table 3.1 Standard loads for different penetrations

The test may be performed on undisturbed specimens and on remoulded specimens which may be compacted either statically or dynamically.

3.4. Preparation of Test Specimen

- **Undisturbed specimen:** Attach the cutting edge to the mould and push it gently into the ground. Remove the soil from the outside of the mould which is pushed in. When the mould is full of soil, remove it from weighing the soil with the mould or by any field method near the spot. Determine the density
- **Remoulded specimen:** Prepare the remoulded specimen at Proctors maximum dry density or any other density at which C.B.R> is required. Maintain the specimen at optimum moisture content or the field moisture as required. The material used should pass 20 mm I.S. sieve but it should be retained on 4.75 mm

I.S. sieve. Prepare the specimen either by dynamic compaction or by static compaction.

3.5. Dynamic Compaction

- Take about 4.5 to 5.5 kg of soil and mix thoroughly with the required water. Fix the extension collar and the base plate to the mould. Insert the spacer disc over the base. Place the filter paper on the top of the spacer disc.
- Compact the mix soil in the mould using either light compaction or heavy compaction. For light compaction, compact the soil in 3 equal layers, each layer being given 55 blows by the 2.6 kg rammer. For heavy compaction compact the soil in 5 layers, 56 blows to each layer by the 4.89 kg rammer.
- Remove the collar and trim off soil.
- Turn the mould upside down and remove the base plate and the displacer disc.
- Weigh the mould with compacted soil and determine the bulk density and dry density.
- Put filter paper on the top of the compacted soil (collar side) and clamp the perforated base plate on to it.

3.6. Static Compaction

- Calculate the weight of the wet soil at the required water content to give the desired density when occupying the standard specimen volume in the mould from the expression.

W =desired dry density * (1+w) V

Where W = Weight of the wet soil

w = desired water content

V = volume of the specimen in the mould = (as per the mould available in Lab)

- Take the weight W (calculated as above) of the mix soil and place it in the mould.
- Place a filter paper and the displacer disc on the top of soil.
- Keep the mould assembly in static loading frame and compact by pressing the displacer disc till the level of disc reaches the top of the mould.
- Keep the load for some time and then release the load. Remove the displacer disc.
- The test may be conducted for both soaked as well as unsoaked conditions.
- If the sample is to be soaked, in cases of compaction, put a filter paper on the top of the soil and place the adjustable stem and perforated plate on the top of filter paper.
- Put annular weights to produce a surcharge equal to weight of base material and pavement expected in actual construction. Each 2.5 kg weight is equivalent to 7 cm construction. A minimum of two weights should be put.
- Immerse the mould assembly and weights in a tank of water and soak it for 96 hours. Remove the mould from tank.
- Note the consolidation of the specimen.

3.7. Procedure for Penetration Test

- Place the mould assembly with the surcharge weights on the penetration test machine.
- Seat the penetration piston at the center of the specimen with the smallest possible load, but in no case in excess of 4 kg so that full contact of the piston on the sample is established.
- Set the stress and strain dial gauge to read zero. Apply the load on the piston so that the penetration rate is about 1.25 mm/min.
- Record the load readings at penetrations of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10 and 12.5 mm. Note the maximum load and corresponding penetration if it occurs for a penetration less than 12.5 mm.
- Detach the mould from the loading equipment. Take about 20 to 50 g of soil from the top 3 cm layer and determine the moisture content.

3.8. Observation and Recording

- For Dynamic Compaction
  - Optimum water content (%)
  - Weight of mould + compacted specimen g
  - Weight of empty mould g
  - Weight of compacted specimen g
  - Volume of specimen cm^3
  - Bulk density g/cc
  - Dry density g/cc
  - For static compaction
  - Dry density g/cc
  - Moulding water content %
  - Wet weight of the compacted soil, (W)gm
  - Period of soaking 96 hrs. (4days).

If the initial portion of the curve is concave upwards, apply correction by drawing a tangent to the curve at the point of greatest slope and shift the origin. Find and record the correct load reading corresponding to each penetration.

C.B.R. = P_T/P_S X 100

Where P_T = Corrected test load corresponding to the chosen penetration from the load penetration curve.

P_S = Standard load for the same penetration taken from the table 1.1.

Penetration Dial		Load Dial		Corrected Load
Readings	Penetration (mm)	Proving ring reading	Load (kg)	

Table 3.2: Standard Corrected load

3.9. Interpretation and recording

C.B.R. of specimen at 2.5 mm penetration

C.B.R. of specimen at 5.0 mm penetration

The C.B.R. values are usually calculated for penetration of 2.5 mm and 5 mm. Generally the C.B.R. value at 2.5 mm will be greater that at 5 mm and in such a case/the former shall be taken as C.B.R. for design purpose. If C.B.R. for 5 mm exceeds that for 2.5 mm, the test should be repeated. If identical results follow, the C.B.R. corresponding to 5 mm penetration should be taken for design.

3.10 Test Results

The CBR tests were conducted using unstabilized and stabilized soil samples, these tests are presented in the tables and respective graphs as shown below.

3.11. Classification of Unstabilized Soil

The composition of soil used for cement stabilization is as follows:

Sand = 66.00 %

Silt = 18.00 %

Clay = 16.00 %

The composition of soil used for lime stabilization is as follows

Sand = 24.80 %

Silt + Clay = 75.2%

Plasticity Index: 15.26 %

3.12. Test Results of C.B.R. Test for Cement Stabilization

3.12.1. C.B.R. Test of Unstabilized Soil in Cement Stabilization

Table 3.3 Load vs. Penetration of Soil Sample used in Cement Stabilization

Penetration (mm)	Load (Kg)	
0	0.00	
0.5	4.70	
1	11.75	
1.5	28.20	
2	39.95	
2.5	60.40	
3	79.90	
4	103.40	
5	123.30	
7.5	150.40	
10	171.55	
12	183.30	
CBR (%) at 2.5mm	4.41	
CBR (%) at 5.0mm	6.00	Accepted

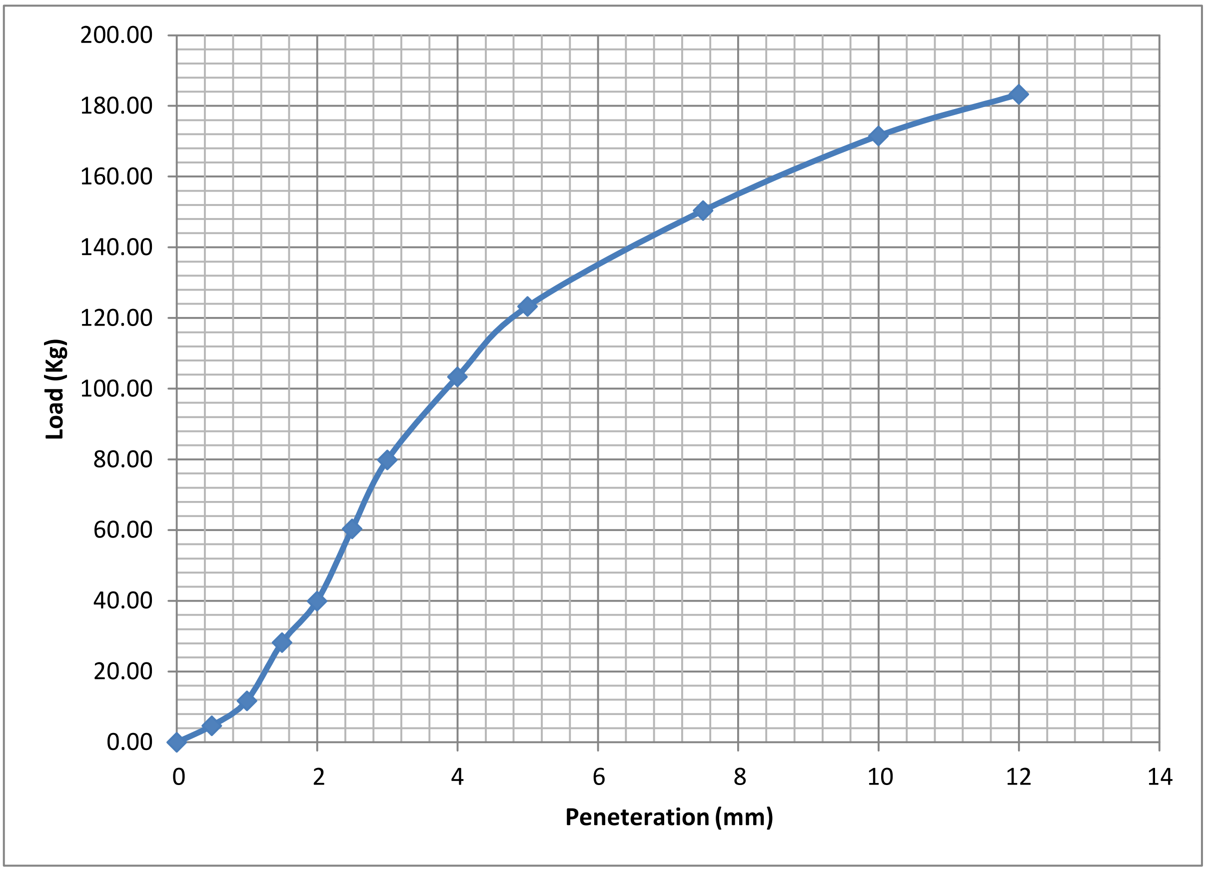

Figure 3.2 Load vs. Penetration of Soil Sample used in Cement Stabilization

3.12.2. CBR Test of Soil Stabilized with 2% Cement

Table 3.4. Load vs. Penetration of Soil Sample with 2% Cement

Penetration (mm)	Load (Kg)	
0	0.00	
0.5	6.25	
1	16.63	
1.5	39.51	
2	57.13	
2.5	80.33	
3	106.27	
4	137.52	
5	163.99	
7.5	212.03	
10	228.16	
12	243.79	
CBR (%) at 2.5mm	5.86	
CBR (%) at 5.0mm	7.98	Accepted

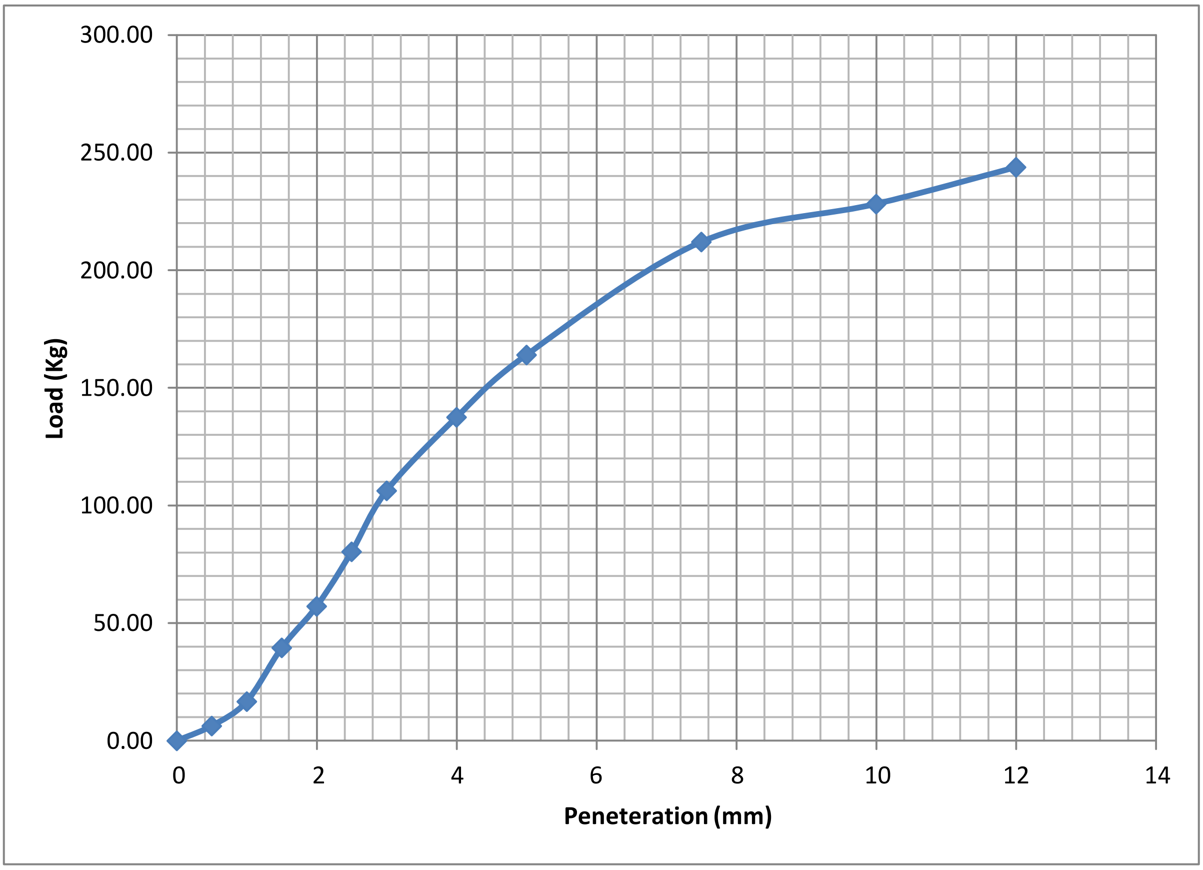

Figure 3.3 Loads *vs.* Penetration of Soil Sample with 2% Cement

3.12.3. CBR Test of Soil Stabilized With 4% Cement

Table 3.5 Load vs. Penetration of Soil Sample with 4% Cement

Penetration (mm)	Load (Kg)	
0	0.00	
0.5	7.94	
1	22.86	
1.5	47.66	
2	67.52	
2.5	102.08	
3	141.03	
4	174.75	
5	180.01	
7.5	204.46	
10	219.05	
12	224.37	
CBR (%) at 2.5mm	7.45	
CBR (%) at 5.0mm	8.76	Accepted

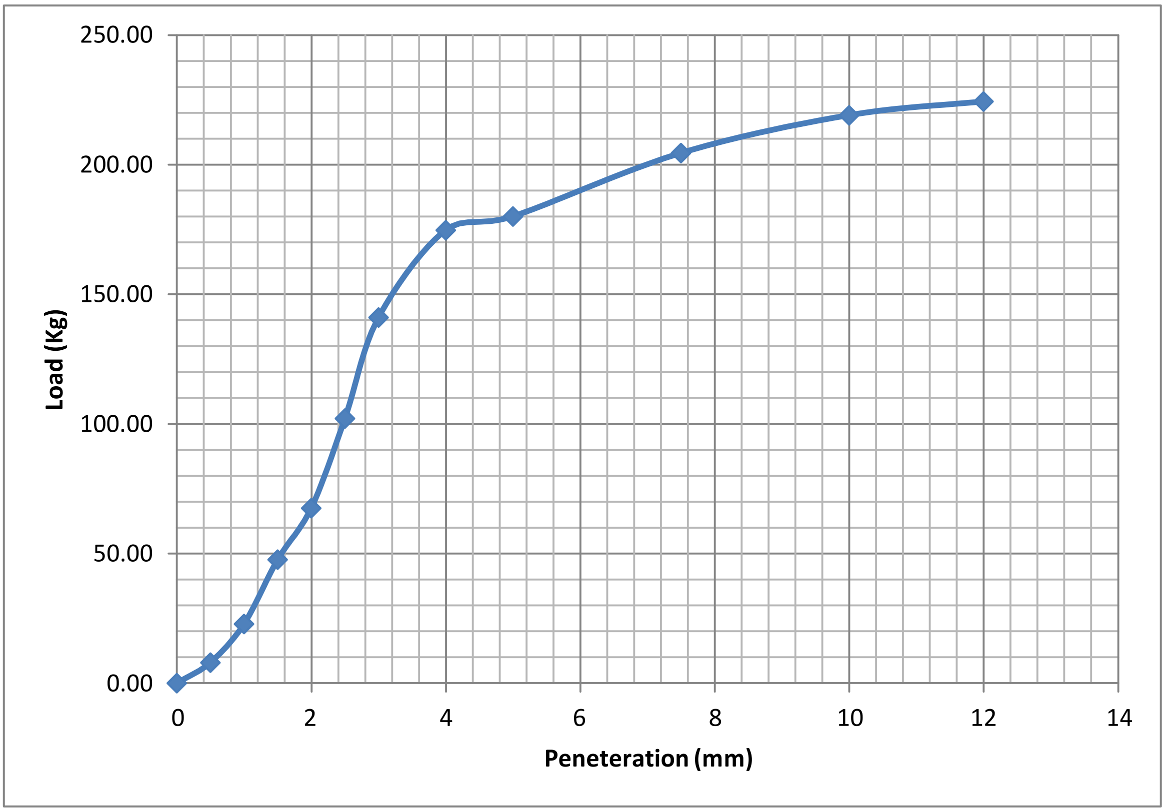

Figure 3.4 Loads vs. Penetration of Soil Sample with 4% Cement

3.12.4. Variation of MDD with Cement Content

Table 3.6: Variation of Maximum Dry Density (MDD) with cement content

Sample No.	MDD (Kg/m^3)	Percentage Increase in MDD
0% Cement	1820	-
2% Cement	1868	2.66%
4% Cement	1902	4.53%
8% Cement	1924	5.74%
12% Cement	1944	6.85%
15% Cement	1904	4.64%

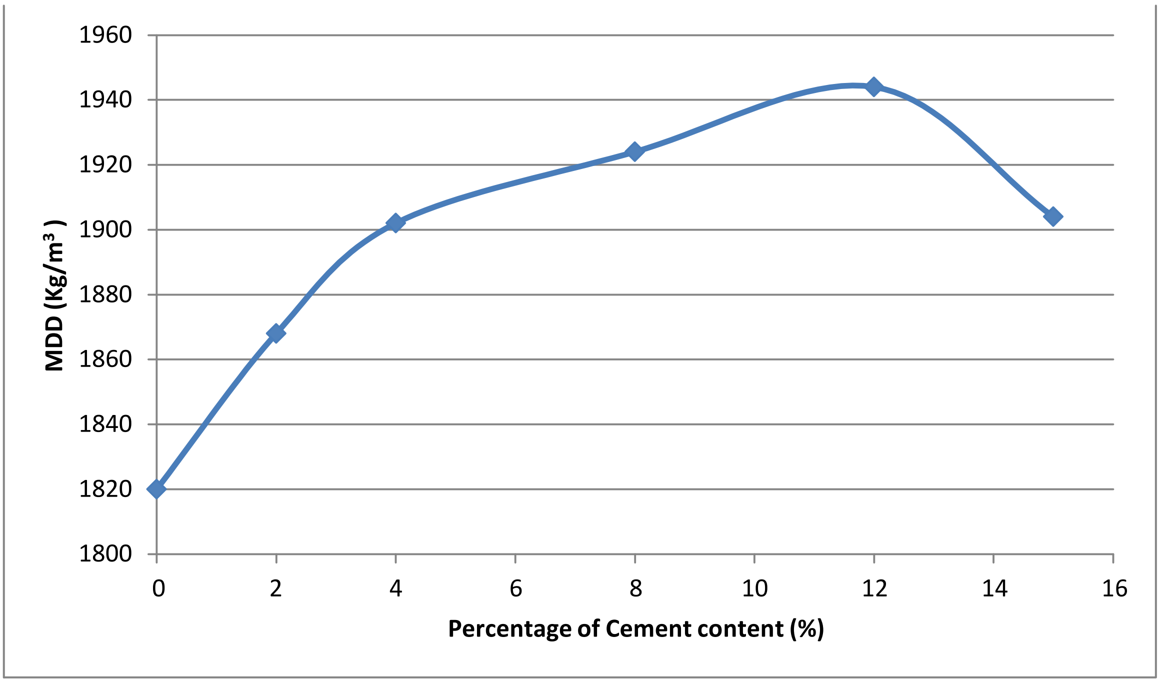

Figure 3.5: Variation of Maximum Dry Density (MDD) with cement content

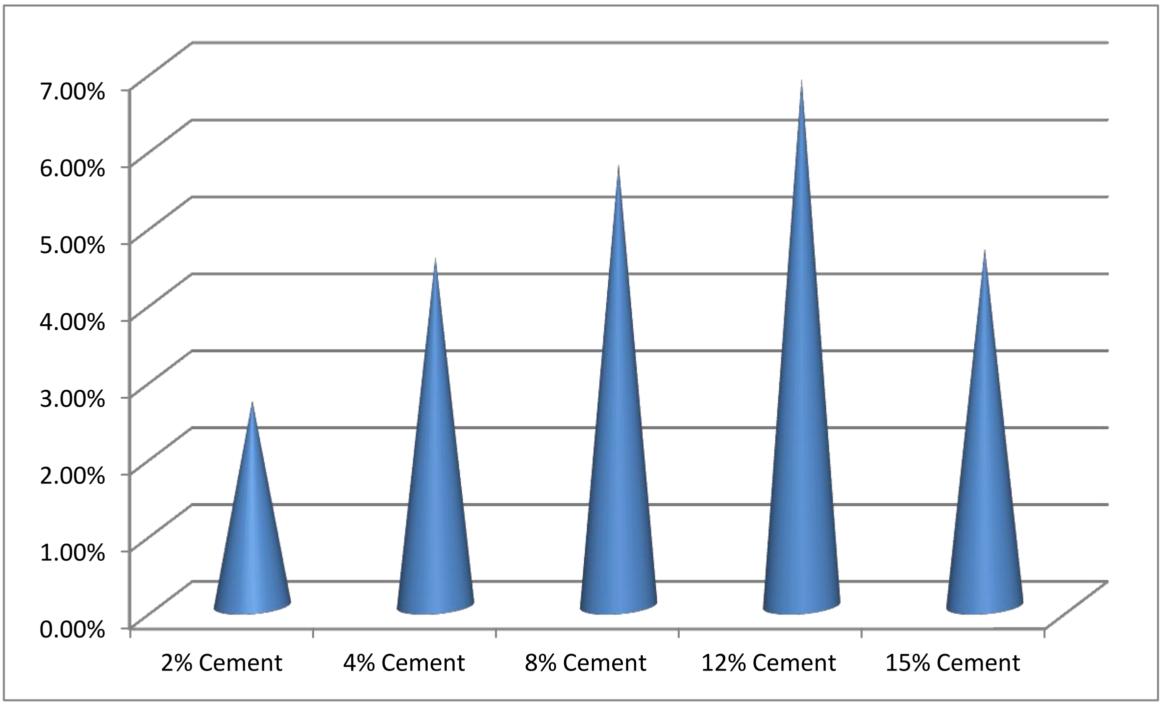

Figure 3.6: Variation of Maximum Dry Density (MDD) in Comparison to base sample

3.12.5. Variation of OMC with Cement Content

Table 3.7: Variation of Optimum Moisture Content (OMC) with cement content

Sample No.	OMC (%)	Percentage Increase in OMC
0% Cement	12.4	-
2% Cement	11.97	-3.45%
4% Cement	11.80	-4.81%
8% Cement	11.44	-7.72%
12% Cement	11.18	-9.85%
15% Cement	12.13	+2.15%

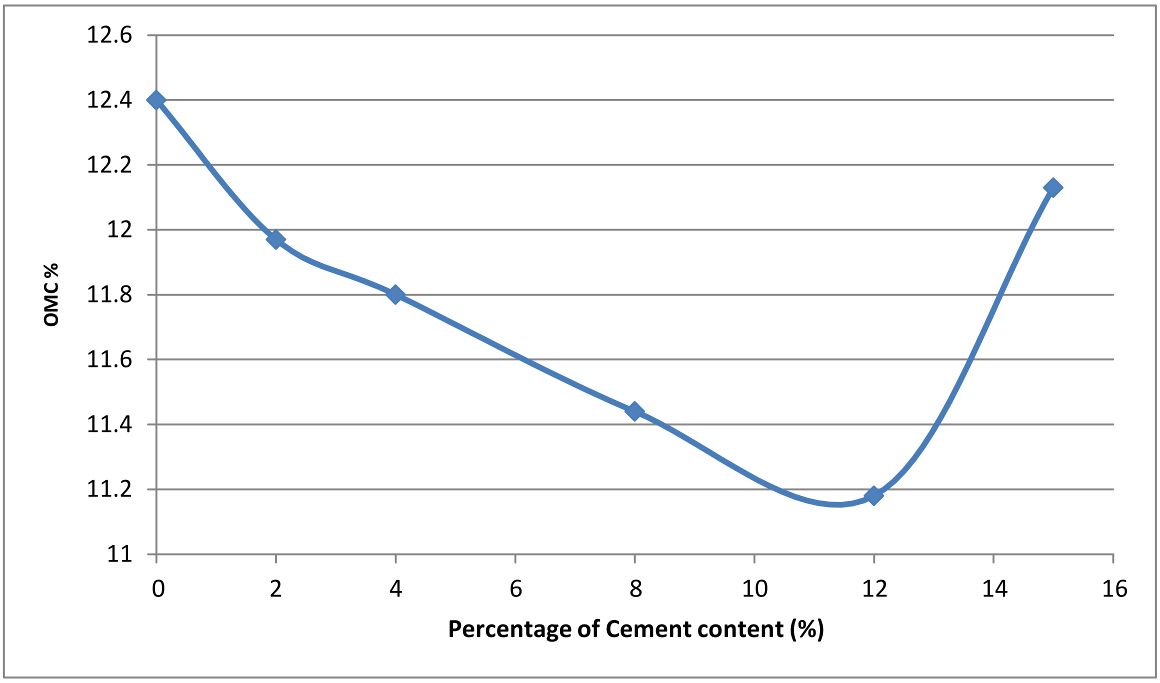

Figure 3.7: Variation of Optimum Moisture Content (OMC) with cement content

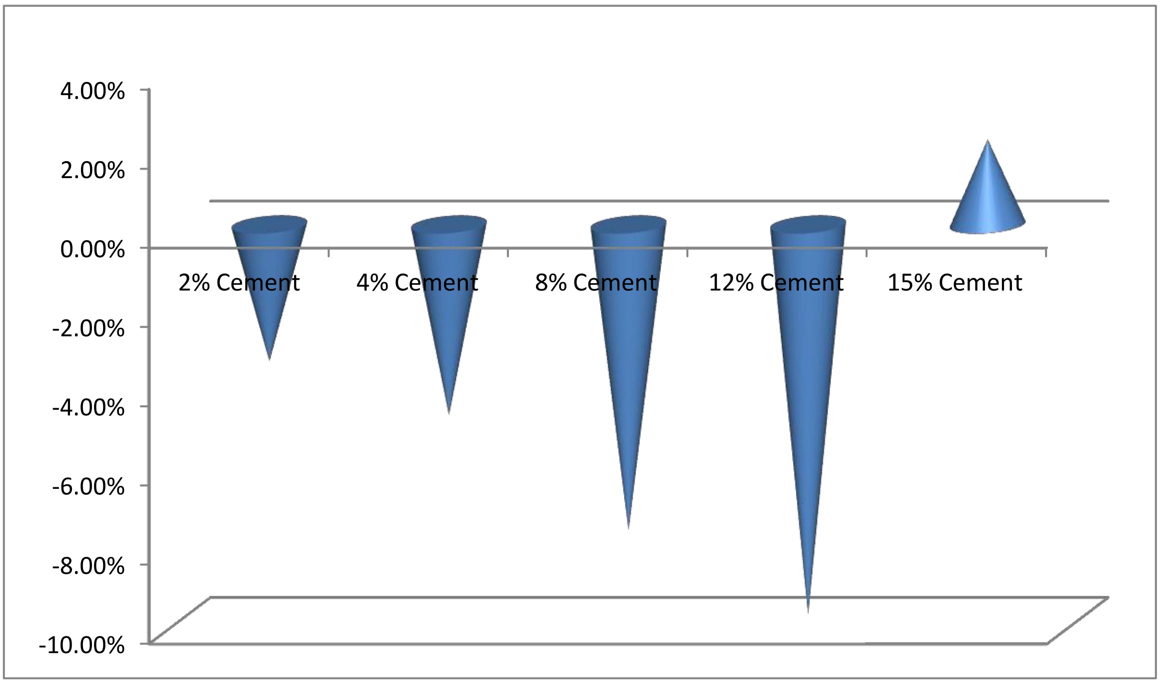

Figure 3.8: Variation of Optimum Moisture Content (OMC) in Comparison to Base sample

3.12.6. Variation of CBR with Cement Content

Table 3.8: Variation of California Bearing Ratio (Soaked) with cement content

Sample No.	CBR (%)
0% Cement	6.0
2% Cement	7.98
4% Cement	8.76
8% Cement	9.54
12% Cement	9.86
15% Cement	4.31

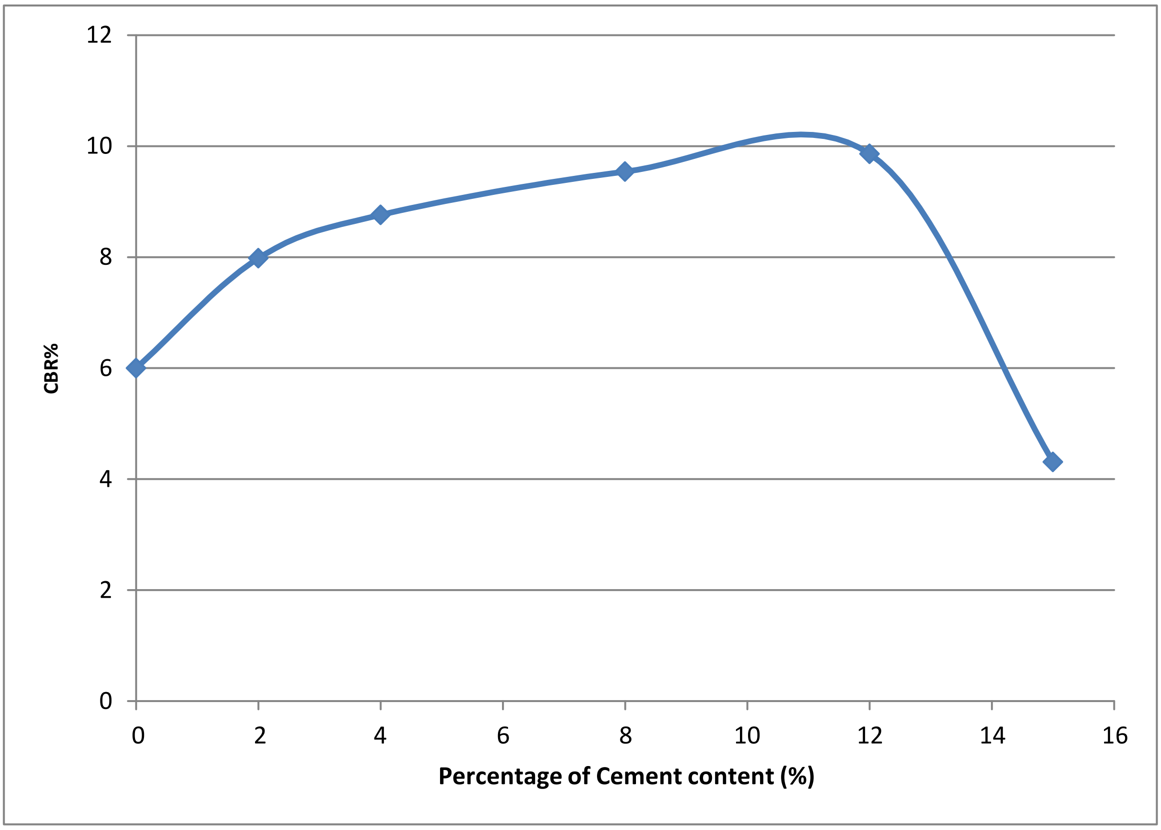

Figure 3.9: Variation of California Bearing Ratio (Soaked) with cement content

Table 3.9: Variation of California Bearing Ratio (Soaked) with cement content

Sample No.	Percentage Increase in CBR
0% Cement	-
2% Cement	33.33%
4% Cement	46.16%
8% Cement	59.2%
12% Cement	64.26%
15% Cement	-28.17%

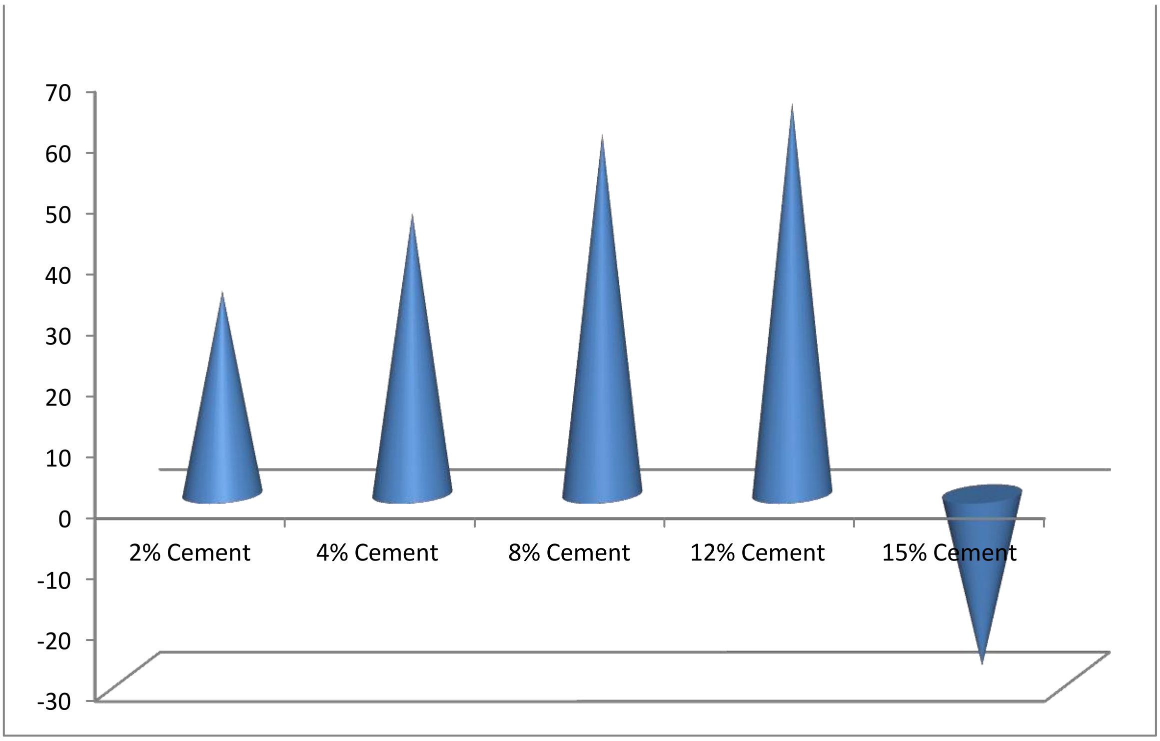

Figure 3.10: Variation of California Bearing Ratio (Soaked) with cement content

3.13. Test Results of C.B.R. Test for Lime Stabilization

3.13.1. C.B.R. Test of Unstabilized Soil In Lime Stabilization

Table 3.10 Load vs. Penetration of Soil Sample used in Lime Stabilization

Penetration (mm)	Load (Kg)	
0	0.00	
0.5	11.09	
1	20.19	
1.5	27.26	
2	38.81	
2.5	41.10	
3	50.36	
4	54.53	
5	58.01	
7.5	84.63	
10	110.88	
12	118.73	
CBR (%) at 2.5mm	3.00	Accepted
CBR (%) at 5.0mm	2.82	

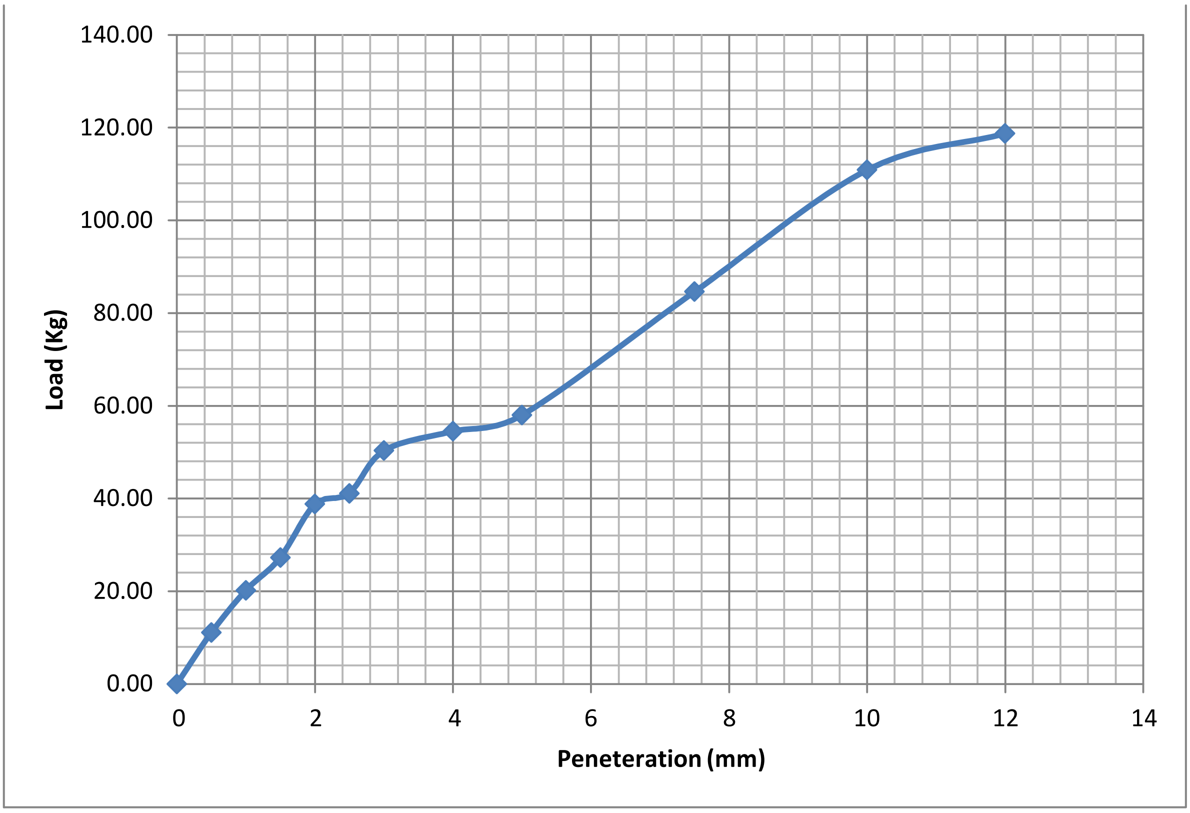

Figure 3.11 Loads vs. Penetration of Soil Sample used in Lime Stabilization

3.13.2. CBR Test of Soil Stabilized With 2% Lime

Table 3.11 Load vs. Penetration of Soil Sample with 2 % Lime

Penetration (mm)	Load (Kg)	
0	0.00	
0.5	25.61	
1	43.53	
1.5	62.97	
2	84.69	
2.5	94.94	
3	110.38	
4	125.96	
5	134.00	
7.5	184.74	
10	249.73	
12	265.86	
CBR (%) at 2.5mm	6.93	
CBR (%) at 5.0mm	6.52	Accepted

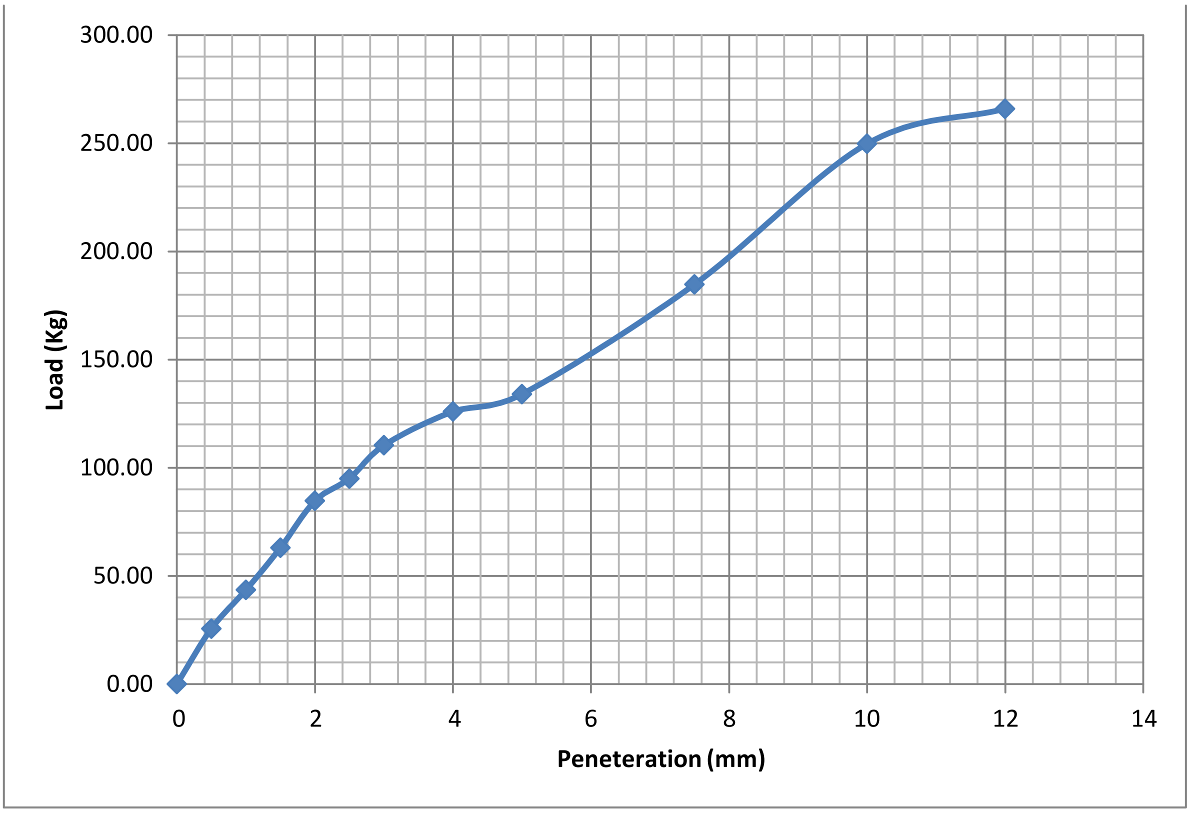

Figure 3.12 Loads vs. Penetration of Soil Sample with 2 % Lime

3.13.3. CBR Test of Soil Stabilized With 4% Lime

Table 3.12 Load vs. Penetration of Soil Sample with 4 % Lime

Penetration (mm)	Load (Kg)	
0	0.00	
0.5	35.20	
1	64.10	
1.5	86.55	
2	123.22	
2.5	136.49	
3	159.89	
4	173.13	
5	184.18	
7.5	288.70	
10	332.04	
12	356.97	
CBR (%) at 2.5mm	9.96	
CBR (%) at 5.0mm	8.96	Accepted

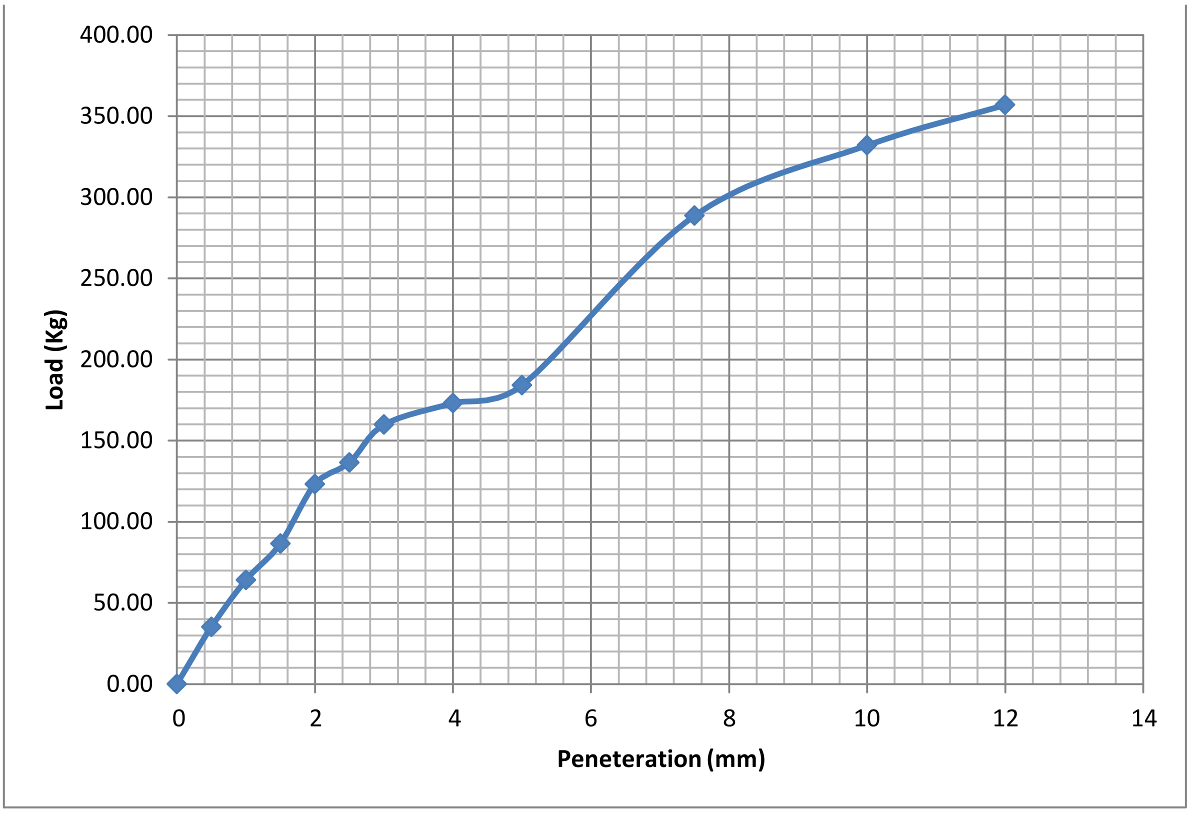

Figure 3.13 Loads vs. Penetration of Soil Sample with 4 % Lime

3.13.4. Variation of CBR with Lime Content

Table 3.13: Variation of California Bearing Ratio (Soaked) with Lime content

Sample No.	CBR (%)
0% Lime	3.0
2% Lime	6.52
4% Lime	8.96
10% Lime	9.12

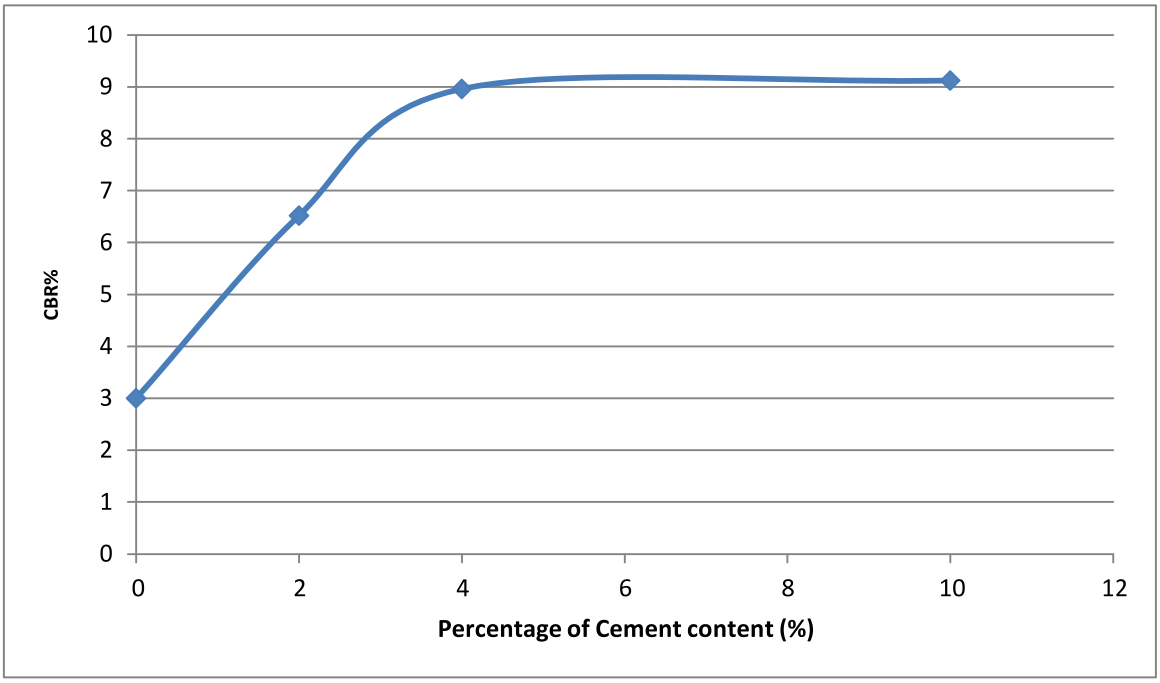

Figure 3.14: Variation of California Bearing Ratio (Soaked) with Lime content

Table 3.14: Percentage increase in CBR Value with addition of Lime

S.No	Description	Percentage Increase (%)
1	Soil Sample CBR 3%	Design with Base Sample
2	With Lime addition 2%, CBR 6.52 %	117.33%

3	With Lime addition 4%, CBR 8.96%	198.00% and 37.5% with respect to 2% trial.
4	With Lime addition 10%, CBR 9.12%	204.00% with respect to base sample and 1.78% with respect to 4% trial.

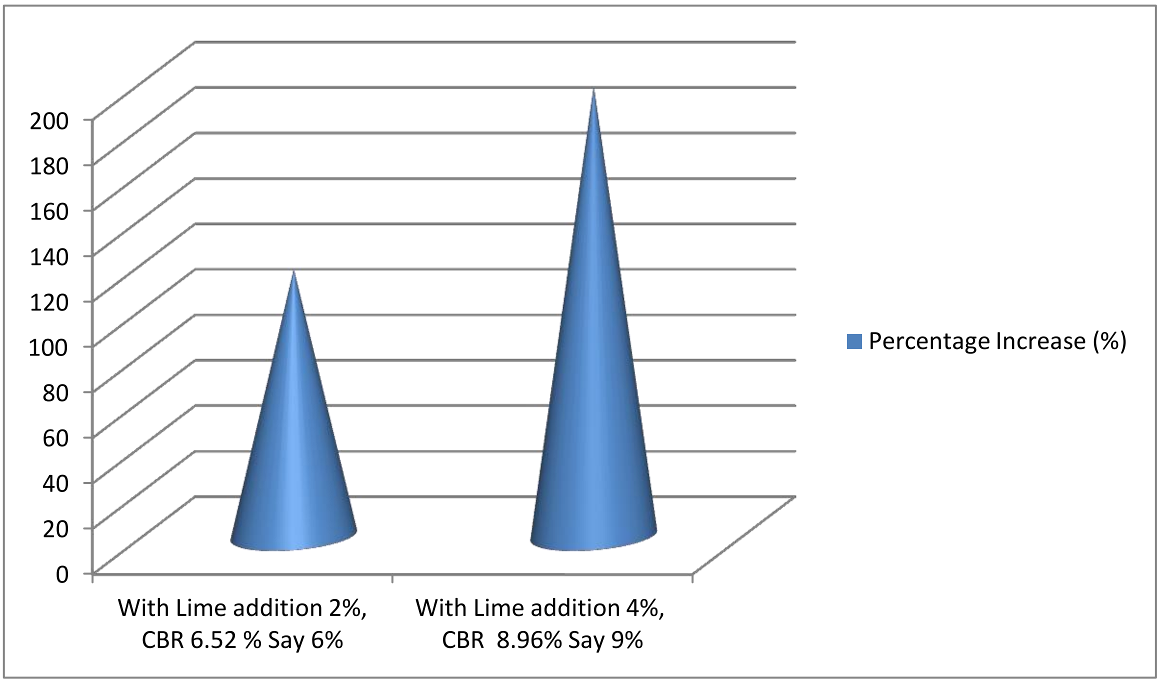

Figure 3.15: Percentage increase in CBR Value with addition of Lime

3.13.5. Variation of OMC and MDD with Lime Content

The Lime treated soil has a lower maximum density than the original untreated soil. and the OMC increases with the increase in Lime content.

Table 3.15: Variation of OMC and MDD with Lime

Sample No.	OMC (%)	MDD (Kg/m^3)	Percentage Increase in OMC	Percentage Increase in MDD
0% Lime	14.6	1930	-	-
2% Lime	15.4	1836	5.48	-4.87
4% Lime	18.8	1810	28.77	-6.22
6% Lime	21.3	1788	45.89	-7.26
10% Lime	21.4	1773	46.60	-8.13

The Plasticity index will be reduced drastically with the increase in lime content in the soil sample.

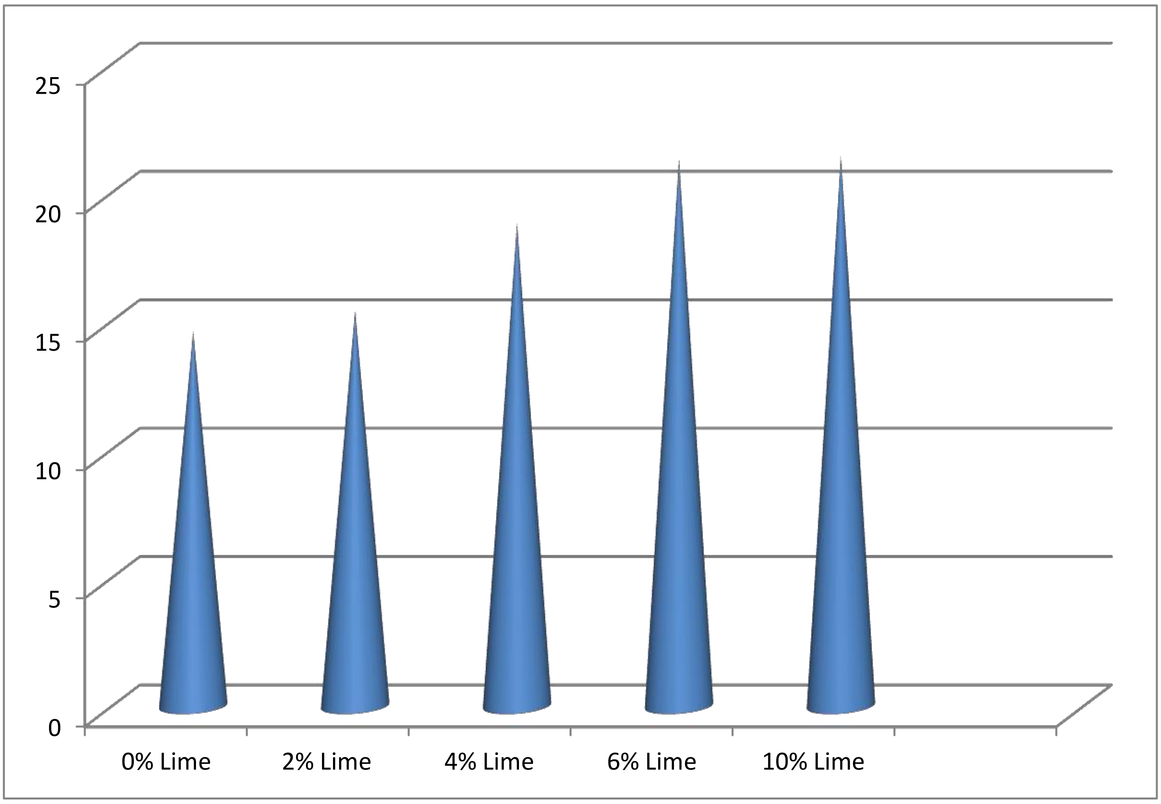

Figure 3.16: Variation of Optimum Moisture Content (OMC) with Lime content

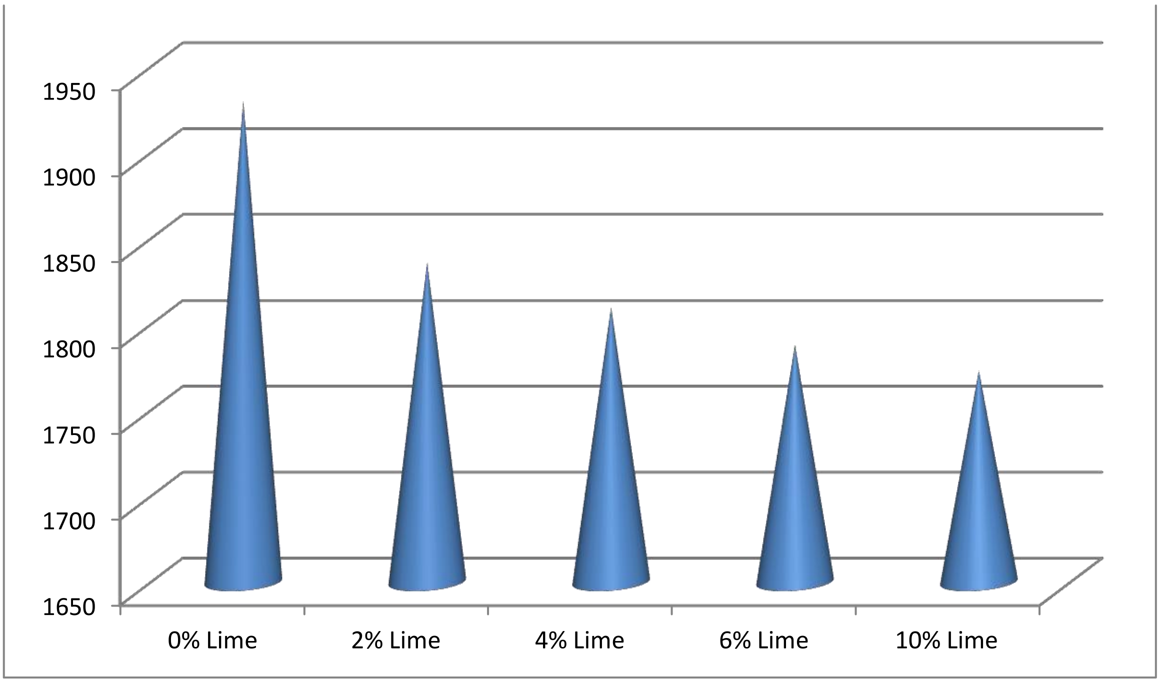

Figure 3.17: Variation of Maximum Dry Density (MDD) with Lime content

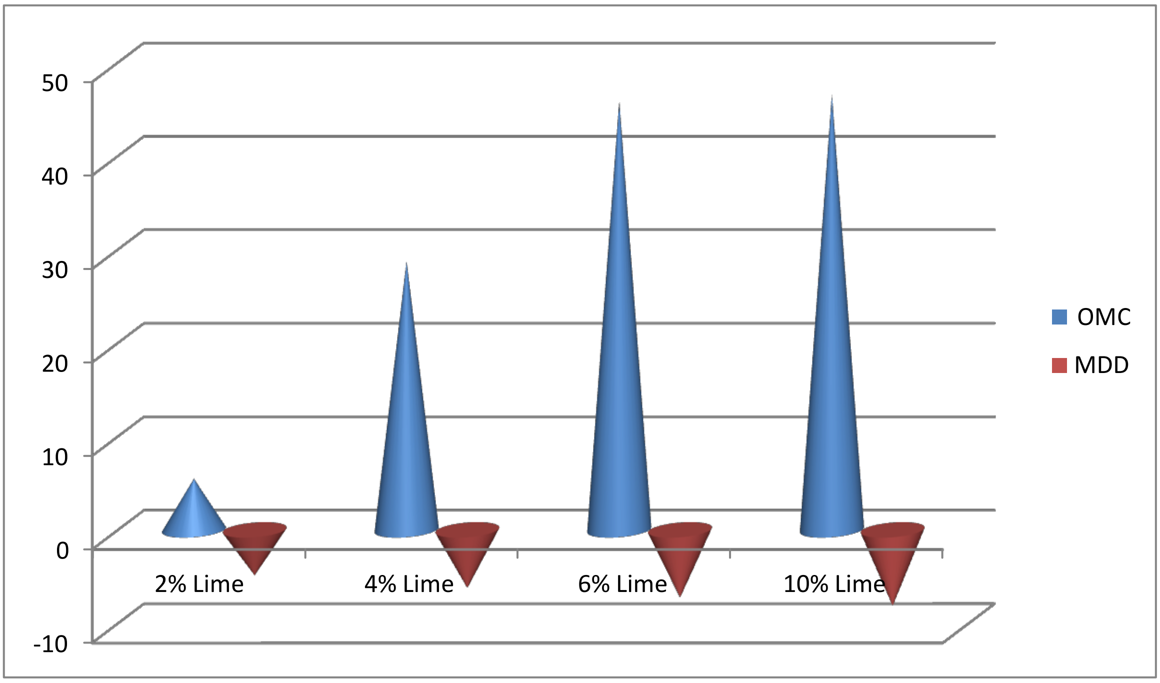

Figure 3.18: Variation of Maximum Dry Density (MDD) and Optimum Moisture Content (OMC)

Chapter 4

Design of Flexible Pavement as per IRC 37:2012

4.1. Axle load Survey

Axle load survey has been carried out on the random sampling bases. Selected categories of commercial vehicles were stopped. The enumerators guided the front wheel of the vehicle slowly on the calibrated Axle Load Pad and the readings were recorded on the specially designed format once the readings got stabilized. The vehicle was directed to move ahead slowly till the rear wheel properly mounted on the axle pad. The procedure was repeated while recording the readings on the meter. In case of tandem axles, the second, third and subsequent axle was also directed on the wheel pad in sequential manner and the measurement of weight was repeated.

Table 4.1 Axle load survey Coding Schedule

Vehicle Code	Vehicle Type	Commodity Code	Commodity Carried
1	Car / Jeep	1	Liquid Cargo
2	Taxi	2	Chemicals / Fertilizers / Medicals
3	S.T.	3	Engineering Goods
4	Private Buses	4	Automobiles Goods
5	LCV / Vans - Tempo	5	Electronics
6	Truck 2 Axle	6	Buildings Materials
7	Truck 3 Axle	7	Foods & Agriculture Produts
8	MAV Container 4 Axle	8	Container Cargo
9	MAV Container 5 Axle	9	Clothes & Textiles
10	MAV Container 6 Axle	10	Coal
		11	Misc. & Others
		12	Empty
		13	Passenger

Table 4.2 Axle load survey

S.No.	Vehicle type	Commodity type	Axle Load (Kg) - Single Sides						Axle Load (Tonnes) - Both Sides						Gross Vehicle weight
			FRONT AXLE	REAR AXLE 1	REAR AXLE 2	REAR AXLE 3	REAR AXLE 4	REAR AXLE 5	FRONT AXLE	REAR AXLE 1	REAR AXLE 2	REAR AXLE 3	REAR AXLE 4	REAR AXLE 5	
			Front Axle Single/ Tandem	Axle no 2 S-T	Axle no 3 S-T	Axle no 4 S-T	Axle no 5 S-T	Axle no 65 S-T	Front Axle Single/ Tandem	Axle no 2 S-T	Axle no 3 S-T	Axle no 4 S-T	Axle no 5 S-T	Axle no 6 S-T	
1	7	7	3300	4450	4700				6.600	8.900	9.400	0.000	0.000	0.000	**24.900**
2	8	5	2900	5050	4150	4050			5.800	10.100	8.300	8.100	0.000	0.000	**32.300**
3	6	4	2650	6050					5.300	12.100	0.000	0.000	0.000	0.000	**17.400**
4	8	3	3100	6850	6950	6800			6.200	13.700	13.900	13.600	0.000	0.000	**47.400**
5	8	6	3400	7950	8050	7850			6.800	15.900	16.100	15.700	0.000	0.000	**54.500**
6	7	7	3000	5100	4350				6.000	10.200	8.700	0.000	0.000	0.000	**24.900**
7	6	3	1650	4500					3.300	9.000	0.000	0.000	0.000	0.000	**12.300**
8	7	11	3100	4650	4800				6.200	9.300	9.600	0.000	0.000	0.000	**25.100**
9	7	7	3500	6500	6450				7.000	13.000	12.900	0.000	0.000	0.000	**32.900**
10	6	4	2450	4600					4.900	9.200	0.000	0.000	0.000	0.000	**14.100**
11	7	7	3250	3800	3650				6.500	7.600	7.300	0.000	0.000	0.000	**21.400**
12	6	5	2600	5800					5.200	11.600	0.000	0.000	0.000	0.000	**16.800**
13	7	7	2900	4550	4550				5.800	9.100	9.100	0.000	0.000	0.000	**24.000**
14	7	7	3050	3850	3900				6.100	7.700	7.800	0.000	0.000	0.000	**21.600**
15	7	11	2850	4700	4950				5.700	9.400	9.900	0.000	0.000	0.000	**25.000**
16	9	4	3000	5400	3900	4300	5100		6.000	10.800	7.800	8.600	10.200	0.000	**43.400**
17	8	4	2900	7800	4100	3300			5.800	15.600	8.200	6.600	0.000	0.000	**36.200**
18	7	7	2850	4850	4700				5.700	9.700	9.400	0.000	0.000	0.000	**24.800**
19	7	2	3450	4350	4600				6.900	8.700	9.200	0.000	0.000	0.000	**24.800**
20	7	7	3250	5000	5700				6.500	10.000	11.400	0.000	0.000	0.000	**27.900**
21	9	4	3250	6500	4050	4300	4050		6.500	13.000	8.100	8.600	8.100	0.000	**44.300**

22	9	4	3150	6600	4650	4950	5500		6.300	13.200	9.300	9.900	11.000	0.000	**49.700**
23	7	4	2900	6050	5600				5.800	12.100	11.200	0.000	0.000	0.000	**29.100**
24	7	11	2850	5050	4850				5.700	10.100	9.700	0.000	0.000	0.000	**25.500**
25	6	7	2900	5850					5.800	11.700	0.000	0.000	0.000	0.000	**17.500**
26	7	7	3050	3850	4750				6.100	7.700	9.500	0.000	0.000	0.000	**23.300**
27	6	11	2650	6100					5.300	12.200	0.000	0.000	0.000	0.000	**17.500**
28	6	6	2650	4500					5.300	9.000	0.000	0.000	0.000	0.000	**14.300**
29	7	7	3200	4650	4850				6.400	9.300	9.700	0.000	0.000	0.000	**25.400**
30	7	7	3200	4050	4350				6.400	8.100	8.700	0.000	0.000	0.000	**23.200**
31	7	7	3100	4900	5050				6.200	9.800	10.100	0.000	0.000	0.000	**26.100**
32	7	11	3050	5050	4800				6.100	10.100	9.600	0.000	0.000	0.000	**25.800**
33	6	4	3050	6350					6.100	12.700	0.000	0.000	0.000	0.000	**18.800**
34	6	6	2900	5150					5.800	10.300	0.000	0.000	0.000	0.000	**16.100**
35	7	7	2750	5100	4950				5.500	10.200	9.900	0.000	0.000	0.000	**25.600**
36	7	1	3050	4800	5050				6.100	9.600	10.100	0.000	0.000	0.000	**25.800**
37	6	1	1950	2650					3.900	5.300	0.000	0.000	0.000	0.000	**9.200**
38	7	7	3100	4050	3350				6.200	8.100	6.700	0.000	0.000	0.000	**21.000**
39	7	7	2600	3500	4300				5.200	7.000	8.600	0.000	0.000	0.000	**20.800**
40	7	7	3200	6500	6600				6.400	13.000	13.200	0.000	0.000	0.000	**32.600**
41	6	4	2700	5850					5.400	11.700	0.000	0.000	0.000	0.000	**17.100**
42	6	4	2750	5350					5.500	10.700	0.000	0.000	0.000	0.000	**16.200**
43	6	4	3300	5400					6.600	10.800	0.000	0.000	0.000	0.000	**17.400**
44	6	11	2400	4600					4.800	9.200	0.000	0.000	0.000	0.000	**14.000**
45	7	7	2950	4900	4800				5.900	9.800	9.600	0.000	0.000	0.000	**25.300**
46	7	11	3050	4850	5150				6.100	9.700	10.300	0.000	0.000	0.000	**26.100**
47	7	7	3050	4800	4300				6.100	9.600	8.600	0.000	0.000	0.000	**24.300**
48	7	11	3200	6200	5950				6.400	12.400	11.900	0.000	0.000	0.000	**30.700**
49	8	4	2650	3050	4800	4200			5.300	6.100	9.600	8.400	0.000	0.000	**29.400**
50	6	7	3550	7900					7.100	15.800	0.000	0.000	0.000	0.000	**22.900**
51	7	11	3300	4650	4450				6.600	9.300	8.900	0.000	0.000	0.000	**24.800**

52	7	7	2700	4300	4050				5.400	8.600	8.100	0.000	0.000	0.000	**22.100**
53	7	7	2950	4950	4800				5.900	9.900	9.600	0.000	0.000	0.000	**25.400**
54	7	11	2750	4700	4400				5.500	9.400	8.800	0.000	0.000	0.000	**23.700**
55	7	7	3050	4800	4600				6.100	9.600	9.200	0.000	0.000	0.000	**24.900**
56	6	4	3850	6500					7.700	13.000	0.000	0.000	0.000	0.000	**20.700**
57	6	4	2800	4550					5.600	9.100	0.000	0.000	0.000	0.000	**14.700**
58	7	7	4150	4250	4550				8.300	8.500	9.100	0.000	0.000	0.000	**25.900**
59	7	4	2800	6250	6100				5.600	12.500	12.200	0.000	0.000	0.000	**30.300**
60	6	11	3100	4500					6.200	9.000	0.000	0.000	0.000	0.000	**15.200**
61	5	11	1600	2100					3.200	4.200	0.000	0.000	0.000	0.000	**7.400**
62	4	13	2400	2900					4.800	5.800	0.000	0.000	0.000	0.000	**10.600**
63	7	11	2700	4100	3750				5.400	8.200	7.500	0.000	0.000	0.000	**21.100**
64	7	7	2250	4650	4200				4.500	9.300	8.400	0.000	0.000	0.000	**22.200**
65	7	7	3100	4650	4700				6.200	9.300	9.400	0.000	0.000	0.000	**24.900**
66	5	11	1250	1800					2.500	3.600	0.000	0.000	0.000	0.000	**6.100**
67	7	7	2950	6800	6450				5.900	13.600	12.900	0.000	0.000	0.000	**32.400**
68	6	11	2600	6050					5.200	12.100	0.000	0.000	0.000	0.000	**17.300**
69	6	4	2250	4550					4.500	9.100	0.000	0.000	0.000	0.000	**13.600**
70	8	4	2900	3950	3050	4700			5.800	7.900	6.100	9.400	0.000	0.000	**29.200**
71	7	7	2700	5250	5300				5.400	10.500	10.600	0.000	0.000	0.000	**26.500**
72	7	11	2450	3400	3150				4.900	6.800	6.300	0.000	0.000	0.000	**18.000**
73	6	4	2900	5400					5.800	10.800	0.000	0.000	0.000	0.000	**16.600**
74	7	11	3100	4800	4950				6.200	9.600	9.900	0.000	0.000	0.000	**25.700**
75	6	6	3200	5800					6.400	11.600	0.000	0.000	0.000	0.000	**18.000**
76	8	4	2800	4800	4650	4300			5.600	9.600	9.300	8.600	0.000	0.000	**33.100**
77	8	4	2600	4600	4550	4400			5.200	9.200	9.100	8.800	0.000	0.000	**32.300**
78	7	11	3200	4400	4650				6.400	8.800	9.300	0.000	0.000	0.000	**24.500**
79	6	4	4400	6050					8.800	12.100	0.000	0.000	0.000	0.000	**20.900**
80	6	4	3750	5850					7.500	11.700	0.000	0.000	0.000	0.000	**19.200**
81	7	7	3400	4900	4500				6.800	9.800	9.000	0.000	0.000	0.000	**25.600**

82	6	8	2100	3950					4.200	7.900	0.000	0.000	0.000	0.000	**12.100**
83	6	11	2850	5950					5.700	11.900	0.000	0.000	0.000	0.000	**17.600**
84	7	7	2800	4750	4600				5.600	9.500	9.200	0.000	0.000	0.000	**24.300**
85	7	7	2900	3400	3250				5.800	6.800	6.500	0.000	0.000	0.000	**19.100**
86	6	11	1800	3250					3.600	6.500	0.000	0.000	0.000	0.000	**10.100**
87	7	11	3250	5050	5000				6.500	10.100	10.000	0.000	0.000	0.000	**26.600**
88	7	7	3100	4350	4800				6.200	8.700	9.600	0.000	0.000	0.000	**24.500**
89	7	11	2400	5200	5600				4.800	10.400	11.200	0.000	0.000	0.000	**26.400**
90	7	7	2850	4900	5050				5.700	9.800	10.100	0.000	0.000	0.000	**25.600**
91	7	7	2950	4550	5050				5.900	9.100	10.100	0.000	0.000	0.000	**25.100**
92	7	11	3050	3500	3550				6.100	7.000	7.100	0.000	0.000	0.000	**20.200**
93	7	2	2900	4500	4350				5.800	9.000	8.700	0.000	0.000	0.000	**23.500**
94	7	1	2150	1550	1400				4.300	3.100	2.800	0.000	0.000	0.000	**10.200**
95	7	7	3000	4850	4800				6.000	9.700	9.600	0.000	0.000	0.000	**25.300**
96	7	11	3500	4750	4800				7.000	9.500	9.600	0.000	0.000	0.000	**26.100**
97	7	7	2700	4650	4500				5.400	9.300	9.000	0.000	0.000	0.000	**23.700**
98	7	7	3300	4500	4600				6.600	9.000	9.200	0.000	0.000	0.000	**24.800**
99	7	4	3050	6400	6150				6.100	12.800	12.300	0.000	0.000	0.000	**31.200**
100	7	7	3000	4900	4950				6.000	9.800	9.900	0.000	0.000	0.000	**25.700**
101	6	4	2850	5300					5.700	10.600	0.000	0.000	0.000	0.000	**16.300**
102	7	11	2650	4750	4800				5.300	9.500	9.600	0.000	0.000	0.000	**24.400**
103	6	7	3100	4500					6.200	9.000	0.000	0.000	0.000	0.000	**15.200**
104	6	11	3150	6050					6.300	12.100	0.000	0.000	0.000	0.000	**18.400**
105	6	11	2750	5300					5.500	10.600	0.000	0.000	0.000	0.000	**16.100**
106	7	7	2750	5600	5500				5.500	11.200	11.000	0.000	0.000	0.000	**27.700**
107	6	11	2850	5850					5.700	11.700	0.000	0.000	0.000	0.000	**17.400**
108	7	7	3250	3650	3800				6.500	7.300	7.600	0.000	0.000	0.000	**21.400**
109	7	7	3000	4700	4650				6.000	9.400	9.300	0.000	0.000	0.000	**24.700**
110	6	8	2700	5700					5.400	11.400	0.000	0.000	0.000	0.000	**16.800**
111	7	11	2650	4200	4300				5.300	8.400	8.600	0.000	0.000	0.000	**22.300**

112	7	4	2850	4950	5050				5.700	9.900	10.100	0.000	0.000	0.000	**25.700**
113	7	7	2450	4050	4100				4.900	8.100	8.200	0.000	0.000	0.000	**21.200**
114	6	7	2700	5600					5.400	11.200	0.000	0.000	0.000	0.000	**16.600**
115	6	4	2650	5400					5.300	10.800	0.000	0.000	0.000	0.000	**16.100**
116	6	4	3450	4500					6.900	9.000	0.000	0.000	0.000	0.000	**15.900**
117	7	11	2850	5600	5800				5.700	11.200	11.600	0.000	0.000	0.000	**28.500**
118	7	7	2850	5600	5800				5.700	11.200	11.600	0.000	0.000	0.000	**28.500**
119	6	4	3300	5400					6.600	10.800	0.000	0.000	0.000	0.000	**17.400**
120	6	4	3100	5200					6.200	10.400	0.000	0.000	0.000	0.000	**16.600**
121	6	11	2651	4550					5.302	9.100	0.000	0.000	0.000	0.000	**14.402**
122	7	7	3150	4200	4350				6.300	8.400	8.700	0.000	0.000	0.000	**23.400**
123	7	2	2900	4600	4750				5.800	9.200	9.500	0.000	0.000	0.000	**24.500**
124	7	7	2850	4350	4050				5.700	8.700	8.100	0.000	0.000	0.000	**22.500**
125	7	7	2950	4800	4950				5.900	9.600	9.900	0.000	0.000	0.000	**25.400**
126	7	11	3450	4250	4500				6.900	8.500	9.000	0.000	0.000	0.000	**24.400**
127	7	7	3100	5400	5250				6.200	10.800	10.500	0.000	0.000	0.000	**27.500**
128	7	7	2650	5450	5050				5.300	10.900	10.100	0.000	0.000	0.000	**26.300**
129	7	7	3300	5000	4700				6.600	10.000	9.400	0.000	0.000	0.000	**26.000**
130	6	1	1900	2450					3.800	4.900	0.000	0.000	0.000	0.000	**8.700**
131	7	7	3100	4850	4850				6.200	9.700	9.700	0.000	0.000	0.000	**25.600**
132	7	7	3100	4800	4950				6.200	9.600	9.900	0.000	0.000	0.000	**25.700**
133	6	4	3100	5400					6.200	10.800	0.000	0.000	0.000	0.000	**17.000**
134	7	7	2700	5300	4850				5.400	10.600	9.700	0.000	0.000	0.000	**25.700**
135	5	11	800	850					1.600	1.700	0.000	0.000	0.000	0.000	**3.300**
136	6	11	2900	5450					5.800	10.900	0.000	0.000	0.000	0.000	**16.700**
137	7	7	3350	4650	4500				6.700	9.300	9.000	0.000	0.000	0.000	**25.000**
138	7	4	3900	4700	4500				7.800	9.400	9.000	0.000	0.000	0.000	**26.200**
139	6	4	2750	5850					5.500	11.700	0.000	0.000	0.000	0.000	**17.200**
140	7	11	3900	6050	5850				7.800	12.100	11.700	0.000	0.000	0.000	**31.600**
141	7	7	3450	4850	4950				6.900	9.700	9.900	0.000	0.000	0.000	**26.500**

142	7	4	3300	4200	4050				6.600	8.400	8.100	0.000	0.000	0.000	**23.100**
143	6	11	2250	5400					4.500	10.800	0.000	0.000	0.000	0.000	**15.300**
144	6	4	3950	4550					7.900	9.100	0.000	0.000	0.000	0.000	**17.000**
145	5	7	1000	2350					2.000	4.700	0.000	0.000	0.000	0.000	**6.700**
146	7	11	3000	3750	3950				6.000	7.500	7.900	0.000	0.000	0.000	**21.400**
147	7	6	2950	6050	5850				5.900	12.100	11.700	0.000	0.000	0.000	**29.700**
148	7	7	2650	4800	4950				5.300	9.600	9.900	0.000	0.000	0.000	**24.800**
149	7	7	2800	4750	4450				5.600	9.500	8.900	0.000	0.000	0.000	**24.000**
150	7	11	2400	3800	3950				4.800	7.600	7.900	0.000	0.000	0.000	**20.300**
151	6	5	2850	4550					5.700	9.100	0.000	0.000	0.000	0.000	**14.800**
152	7	5	3100	5700	4950				6.200	11.400	9.900	0.000	0.000	0.000	**27.500**
153	7	4	3600	6550	6850				7.200	13.100	13.700	0.000	0.000	0.000	**34.000**
154	4	13	2400	2900					4.800	5.800	0.000	0.000	0.000	0.000	**10.600**
155	7	8	3100	5250	5550				6.200	10.500	11.100	0.000	0.000	0.000	**27.800**
156	6	4	3200	4800					6.400	9.600	0.000	0.000	0.000	0.000	**16.000**
157	7	7	2950	5200	4900				5.900	10.400	9.800	0.000	0.000	0.000	**26.100**
158	7	7	2850	4050	4100				5.700	8.100	8.200	0.000	0.000	0.000	**22.000**
159	7	7	2900	3900	3650				5.800	7.800	7.300	0.000	0.000	0.000	**20.900**
160	6	8	2700	4550					5.400	9.100	0.000	0.000	0.000	0.000	**14.500**
161	7	7	2650	4800	5050				5.300	9.600	10.100	0.000	0.000	0.000	**25.000**
162	7	7	2650	4900	4750				5.300	9.800	9.500	0.000	0.000	0.000	**24.600**
163	7	2	3200	6500	6750				6.400	13.000	13.500	0.000	0.000	0.000	**32.900**
164	6	4	2800	4500					5.600	9.000	0.000	0.000	0.000	0.000	**14.600**
165	7	2	2550	4050	3850				5.100	8.100	7.700	0.000	0.000	0.000	**20.900**
166	7	2	2850	4600	4850				5.700	9.200	9.700	0.000	0.000	0.000	**24.600**
167	6	6	2700	5400					5.400	10.800	0.000	0.000	0.000	0.000	**16.200**
168	4	13	1950	2500					3.900	5.000	0.000	0.000	0.000	0.000	**8.900**
169	7	7	3400	7100	6950				6.800	14.200	13.900	0.000	0.000	0.000	**34.900**
170	6	4	2950	5800					5.900	11.600	0.000	0.000	0.000	0.000	**17.500**
171	7	11	3200	4450	4600				6.400	8.900	9.200	0.000	0.000	0.000	**24.500**

172	7	11	2900	4700	5000				5.800	9.400	10.000	0.000	0.000	0.000	**25.200**
173	7	7	3600	7500	7350				7.200	15.000	14.700	0.000	0.000	0.000	**36.900**
174	6	4	3100	4550					6.200	9.100	0.000	0.000	0.000	0.000	**15.300**
175	4	12	2100	2750					4.200	5.500	0.000	0.000	0.000	0.000	**9.700**
176	6	6	3200	4550					6.400	9.100	0.000	0.000	0.000	0.000	**15.500**
177	4	13	1600	1950					3.200	3.900	0.000	0.000	0.000	0.000	**7.100**
178	7	11	2900	5200	4850				5.800	10.400	9.700	0.000	0.000	0.000	**25.900**
179	7	7	2750	4200	4450				5.500	8.400	8.900	0.000	0.000	0.000	**22.800**
180	4	13	1800	2300					3.600	4.600	0.000	0.000	0.000	0.000	**8.200**
181	7	8	2850	4850	4950				5.700	9.700	9.900	0.000	0.000	0.000	**25.300**
182	7	5	2650	4400	4350				5.300	8.800	8.700	0.000	0.000	0.000	**22.800**
183	7	8	3200	5300	5200				6.400	10.600	10.400	0.000	0.000	0.000	**27.400**
184	7	7	3500	5550	5350				7.000	11.100	10.700	0.000	0.000	0.000	**28.800**
185	8	4	3300	6150	6600	6350			6.600	12.300	13.200	12.700	0.000	0.000	**44.800**
186	7	7	2900	4850	4700				5.800	9.700	9.400	0.000	0.000	0.000	**24.900**
187	7	2	3100	6050	5200				6.200	12.100	10.400	0.000	0.000	0.000	**28.700**
188	7	1	2800	4850	4800				5.600	9.700	9.600	0.000	0.000	0.000	**24.900**
189	7	11	2800	5250	5300				5.600	10.500	10.600	0.000	0.000	0.000	**26.700**
190	8	8	2600	3600	3900	4050			5.200	7.200	7.800	8.100	0.000	0.000	**28.300**
191	7	6	3000	5850	5600				6.000	11.700	11.200	0.000	0.000	0.000	**28.900**
192	6	6	2900	6050					5.800	12.100	0.000	0.000	0.000	0.000	**17.900**
193	5	7	1900	3050					3.800	6.100	0.000	0.000	0.000	0.000	**9.900**
194	7	3	2950	5850	6050				5.900	11.700	12.100	0.000	0.000	0.000	**29.700**
195	6	5	3100	5450					6.200	10.900	0.000	0.000	0.000	0.000	**17.100**
196	7	9	3400	7050	6800				6.800	14.100	13.600	0.000	0.000	0.000	**34.500**
197	6	8	3100	5600					6.200	11.200	0.000	0.000	0.000	0.000	**17.400**
198	8	3	2500	4900	4300	4050			5.000	9.800	8.600	8.100	0.000	0.000	**31.500**
199	7	4	2950	5200	5550				5.900	10.400	11.100	0.000	0.000	0.000	**27.400**
200	6	4	2850	5400					5.700	10.800	0.000	0.000	0.000	0.000	**16.500**
201	5	11	2200	4400					4.400	8.800	0.000	0.000	0.000	0.000	**13.200**

202	7	2	2900	5800	5300				5.800	11.600	10.600	0.000	0.000	0.000	**28.000**
203	8	8	2300	4800	5050	5000			4.600	9.600	10.100	10.000	0.000	0.000	**34.300**
204	6	7	2850	4550					5.700	9.100	0.000	0.000	0.000	0.000	**14.800**
205	7	5	3300	6300	6450				6.600	12.600	12.900	0.000	0.000	0.000	**32.100**
206	7	1	2700	4050	3850				5.400	8.100	7.700	0.000	0.000	0.000	**21.200**
207	6	11	2800	4500					5.600	9.000	0.000	0.000	0.000	0.000	**14.600**
208	9	2	2800	6400	6350	6050	6200		5.600	12.800	12.700	12.100	12.400	0.000	**55.600**
209	7	8	3000	3500	3300				6.000	7.000	6.600	0.000	0.000	0.000	**19.600**
210	4	13	3000	5400					6.000	10.800	0.000	0.000	0.000	0.000	**16.800**
211	6	3	3050	6050					6.100	12.100	0.000	0.000	0.000	0.000	**18.200**
212	7	11	2850	5400	5550				5.700	10.800	11.100	0.000	0.000	0.000	**27.600**
213	7	1	3200	5400	5200				6.400	10.800	10.400	0.000	0.000	0.000	**27.600**
214	7	2	3050	6000	5850				6.100	12.000	11.700	0.000	0.000	0.000	**29.800**
215	8	8	3050	4050	4800	4850			6.100	8.100	9.600	9.700	0.000	0.000	**33.500**
216	8	4	2700	6300	5800	5650			5.400	12.600	11.600	11.300	0.000	0.000	**40.900**
217	6	5	3050	5800					6.100	11.600	0.000	0.000	0.000	0.000	**17.700**
218	7	4	2600	5300	5600				5.200	10.600	11.200	0.000	0.000	0.000	**27.000**
219	6	6	2950	6550					5.900	13.100	0.000	0.000	0.000	0.000	**19.000**
220	7	6	3500	7000	7050				7.000	14.000	14.100	0.000	0.000	0.000	**35.100**
221	7	3	3050	6050	5800				6.100	12.100	11.600	0.000	0.000	0.000	**29.800**
222	7	9	3050	6200	6350				6.100	12.400	12.700	0.000	0.000	0.000	**31.200**
223	5	11	2300	4050					4.600	8.100	0.000	0.000	0.000	0.000	**12.700**
224	6	7	2900	5400					5.800	10.800	0.000	0.000	0.000	0.000	**16.600**
225	7	7	3050	6050	5850				6.100	12.100	11.700	0.000	0.000	0.000	**29.900**
226	6	8	3300	5400					6.600	10.800	0.000	0.000	0.000	0.000	**17.400**
227	7	9	3600	6900	7050				7.200	13.800	14.100	0.000	0.000	0.000	**35.100**
228	7	5	3100	6850	7050				6.200	13.700	14.100	0.000	0.000	0.000	**34.000**
229	5	7	2050	3400					4.100	6.800	0.000	0.000	0.000	0.000	**10.900**
230	6	11	3050	5850					6.100	11.700	0.000	0.000	0.000	0.000	**17.800**
231	7	8	2600	3200	3050				5.200	6.400	6.100	0.000	0.000	0.000	**17.700**

232	8	8	2500	4300	4450	4600			5.000	8.600	8.900	9.200	0.000	0.000	**31.700**
233	7	7	2700	4300	4250				5.400	8.600	8.500	0.000	0.000	0.000	**22.500**
234	8	11	3050	3200	5600	5450			6.100	6.400	11.200	10.900	0.000	0.000	**34.600**
235	6	4	2900	4550					5.800	9.100	0.000	0.000	0.000	0.000	**14.900**
236	7	8	2500	3050	3000				5.000	6.100	6.000	0.000	0.000	0.000	**17.100**
237	7	2	3350	6250	5600				6.700	12.500	11.200	0.000	0.000	0.000	**30.400**
238	9	4	2650	5400	5900	5750	5850		5.300	10.800	11.800	11.500	11.700	0.000	**51.100**
239	7	11	2700	5050	5200				5.400	10.100	10.400	0.000	0.000	0.000	**25.900**
240	7	11	2950	4050	4250				5.900	8.100	8.500	0.000	0.000	0.000	**22.500**
241	5	7	1950	3800					3.900	7.600	0.000	0.000	0.000	0.000	**11.500**
242	7	7	3300	6200	5900				6.600	12.400	11.800	0.000	0.000	0.000	**30.800**
243	8	3	2700	5300	5050	4800			5.400	10.600	10.100	9.600	0.000	0.000	**35.700**
244	7	6	3200	6400	6050				6.400	12.800	12.100	0.000	0.000	0.000	**31.300**
245	7	1	3100	5050	4900				6.200	10.100	9.800	0.000	0.000	0.000	**26.100**
246	6	8	3400	5300					6.800	10.600	0.000	0.000	0.000	0.000	**17.400**
247	8	4	2550	5400	4850	4850			5.100	10.800	9.700	9.700	0.000	0.000	**35.300**
248	7	5	3350	7200	6850				6.700	14.400	13.700	0.000	0.000	0.000	**34.800**
249	6	3	2950	5800					5.900	11.600	0.000	0.000	0.000	0.000	**17.500**
250	8	8	3050	3900	4250	4300			6.100	7.800	8.500	8.600	0.000	0.000	**31.000**
251	7	2	3300	6050	5400				6.600	12.100	10.800	0.000	0.000	0.000	**29.500**
252	6	4	3150	5650					6.300	11.300	0.000	0.000	0.000	0.000	**17.600**
253	5	6	2000	4500					4.000	9.000	0.000	0.000	0.000	0.000	**13.000**
254	6	7	3050	5800					6.100	11.600	0.000	0.000	0.000	0.000	**17.700**
255	7	11	3200	5400	5500				6.400	10.800	11.000	0.000	0.000	0.000	**28.200**
256	7	4	2700	5400	5650				5.400	10.800	11.300	0.000	0.000	0.000	**27.500**
257	7	8	3000	4800	4650				6.000	9.600	9.300	0.000	0.000	0.000	**24.900**
258	6	5	3400	6050					6.800	12.100	0.000	0.000	0.000	0.000	**18.900**
259	9	2	2900	7050	6550	6200	6350		5.800	14.100	13.100	12.400	12.700	0.000	**58.100**
260	7	1	3050	5300	5250				6.100	10.600	10.500	0.000	0.000	0.000	**27.200**
261	4	12	2050	2700					4.100	5.400	0.000	0.000	0.000	0.000	**9.500**

262	7	7	3050	6050	6000				6.100	12.100	12.000	0.000	0.000	0.000	**30.200**
263	8	11	2900	3050	5400	5300			5.800	6.100	10.800	10.600	0.000	0.000	**33.300**
264	7	8	2700	5600	5450				5.400	11.200	10.900	0.000	0.000	0.000	**27.500**
265	6	3	3100	6300					6.200	12.600	0.000	0.000	0.000	0.000	**18.800**
266	7	7	3300	6800	6550				6.600	13.600	13.100	0.000	0.000	0.000	**33.300**
267	7	7	3200	5050	5200				6.400	10.100	10.400	0.000	0.000	0.000	**26.900**
268	7	3	3050	5900	5700				6.100	11.800	11.400	0.000	0.000	0.000	**29.300**
269	7	8	2900	5700	5550				5.800	11.400	11.100	0.000	0.000	0.000	**28.300**
270	6	5	2800	5400					5.600	10.800	0.000	0.000	0.000	0.000	**16.400**
271	7	11	2850	4250	4150				5.700	8.500	8.300	0.000	0.000	0.000	**22.500**
272	6	11	3200	5900					6.400	11.800	0.000	0.000	0.000	0.000	**18.200**
273	7	7	2600	4050	4300				5.200	8.100	8.600	0.000	0.000	0.000	**21.900**
274	6	8	2800	5850					5.600	11.700	0.000	0.000	0.000	0.000	**17.300**
275	8	4	2650	4750	4800	4850			5.300	9.500	9.600	9.700	0.000	0.000	**34.100**
276	7	1	3050	4900	5050				6.100	9.800	10.100	0.000	0.000	0.000	**26.000**
277	7	4	2750	5250	5350				5.500	10.500	10.700	0.000	0.000	0.000	**26.700**
278	7	8	3050	3600	3500				6.100	7.200	7.000	0.000	0.000	0.000	**20.300**
279	7	2	2700	4400	4750				5.400	8.800	9.500	0.000	0.000	0.000	**23.700**
280	6	6	3100	5400					6.200	10.800	0.000	0.000	0.000	0.000	**17.000**
281	8	8	2650	4050	4300	4550			5.300	8.100	8.600	9.100	0.000	0.000	**31.100**
282	5	11	1700	4800					3.400	9.600	0.000	0.000	0.000	0.000	**13.000**
283	7	5	3500	7850	7600				7.000	15.700	15.200	0.000	0.000	0.000	**37.900**
284	6	4	3100	5750					6.200	11.500	0.000	0.000	0.000	0.000	**17.700**
285	8	11	2600	4600	5800	6050			5.200	9.200	11.600	12.100	0.000	0.000	**38.100**
286	6	8	3050	5600					6.100	11.200	0.000	0.000	0.000	0.000	**17.300**
287	7	7	2900	4200	4400				5.800	8.400	8.800	0.000	0.000	0.000	**23.000**
288	7	9	3200	6900	7050				6.400	13.800	14.100	0.000	0.000	0.000	**34.300**
289	6	5	3300	5650					6.600	11.300	0.000	0.000	0.000	0.000	**17.900**
290	7	6	3200	5050	5200				6.400	10.100	10.400	0.000	0.000	0.000	**26.900**
291	9	4	2450	5600	5800	6250	6150		4.900	11.200	11.600	12.500	12.300	0.000	**52.500**

292	6	3	3400	4500					6.800	9.000	0.000	0.000	0.000	0.000	**15.800**
293	6	7	3100	5800					6.200	11.600	0.000	0.000	0.000	0.000	**17.800**
294	4	13	2400	3100					4.800	6.200	0.000	0.000	0.000	0.000	**11.000**
295	5	6	2250	4050					4.500	8.100	0.000	0.000	0.000	0.000	**12.600**
296	7	1	2950	5100	5050				5.900	10.200	10.100	0.000	0.000	0.000	**26.200**
297	7	11	3100	5800	6050				6.200	11.600	12.100	0.000	0.000	0.000	**29.900**
298	7	2	2900	4800	5050				5.800	9.600	10.100	0.000	0.000	0.000	**25.500**
299	7	7	2700	4800	4650				5.400	9.600	9.300	0.000	0.000	0.000	**24.300**
300	7	11	3050	5600	5450				6.100	11.200	10.900	0.000	0.000	0.000	**28.200**
301	7	3	3250	6300	6150				6.500	12.600	12.300	0.000	0.000	0.000	**31.400**
302	7	7	2400	3800	3950				4.800	7.600	7.900	0.000	0.000	0.000	**20.300**
303	9	2	3050	6400	5750	5700	5600		6.100	12.800	11.500	11.400	11.200	0.000	**53.000**
304	6	5	3050	5600					6.100	11.200	0.000	0.000	0.000	0.000	**17.300**
305	7	6	3300	4700	4850				6.600	9.400	9.700	0.000	0.000	0.000	**25.700**
306	7	4	2850	5800	5400				5.700	11.600	10.800	0.000	0.000	0.000	**28.100**
307	7	7	2950	5050	4850				5.900	10.100	9.700	0.000	0.000	0.000	**25.700**
308	7	5	3300	7000	7400				6.600	14.000	14.800	0.000	0.000	0.000	**35.400**
309	7	11	2850	4300	4500				5.700	8.600	9.000	0.000	0.000	0.000	**23.300**
310	8	8	2750	3600	4050	4050			5.500	7.200	8.100	8.100	0.000	0.000	**28.900**
311	6	6	3200	6200					6.400	12.400	0.000	0.000	0.000	0.000	**18.800**
312	6	8	3000	6050					6.000	12.100	0.000	0.000	0.000	0.000	**18.100**
313	8	3	2950	4350	4850	5050			5.900	8.700	9.700	10.100	0.000	0.000	**34.400**
314	7	5	2950	7600	7750				5.900	15.200	15.500	0.000	0.000	0.000	**36.600**
315	5	11	1800	3500					3.600	7.000	0.000	0.000	0.000	0.000	**10.600**
316	6	4	3050	5950					6.100	11.900	0.000	0.000	0.000	0.000	**18.000**
317	8	4	2750	4950	4400	4600			5.500	9.900	8.800	9.200	0.000	0.000	**33.400**
318	7	9	3400	6800	6750				6.800	13.600	13.500	0.000	0.000	0.000	**33.900**
319	6	7	2900	5850					5.800	11.700	0.000	0.000	0.000	0.000	**17.500**
320	7	4	2700	3050	2900				5.400	6.100	5.800	0.000	0.000	0.000	**17.300**
321	7	7	3300	7000	7050				6.600	14.000	14.100	0.000	0.000	0.000	**34.700**

322	6	11	3100	5600					6.200	11.200	0.000	0.000	0.000	0.000	**17.400**
323	7	5	2900	5400	5350				5.800	10.800	10.700	0.000	0.000	0.000	**27.300**
324	7	1	2850	5300	4850				5.700	10.600	9.700	0.000	0.000	0.000	**26.000**
325	4	13	1800	2950					3.600	5.900	0.000	0.000	0.000	0.000	**9.500**
326	6	8	2650	4550					5.300	9.100	0.000	0.000	0.000	0.000	**14.400**
327	7	11	2700	5200	5050				5.400	10.400	10.100	0.000	0.000	0.000	**25.900**
328	6	3	3300	5450					6.600	10.900	0.000	0.000	0.000	0.000	**17.500**
329	7	3	3300	6400	6050				6.600	12.800	12.100	0.000	0.000	0.000	**31.500**
330	7	7	2700	4400	4350				5.400	8.800	8.700	0.000	0.000	0.000	**22.900**

4.2. Calculation of VDF for Different Vehicle types based upon Axle load Survey

The objective of analysis is to estimate the Vehicle Damage Factor (Mode wise) in either direction. The aim of estimating mode wise VDF is to judge the extent of damage by different categories of goods vehicle, having varying load patterns. In order to convert the axle load data into number of Equivalent Standard Axle loads (ESAL's) the axle loads were grouped in the interval of 0.9 Metric Tonnes. The Equivalency factor derived from the **"fourth power rule"** was multiplied with the frequency of that class interval to achieve the Equivalent Standard Axle Load for the weight class of the sample by using the following formulae as per (IRC : 37-2012)

Single axle with single wheel on either side $= \left(\frac{\text{axle load in kN}}{65}\right)^4$

Single axle with dual wheels on either side $= \left(\frac{\text{axle load in kN}}{80}\right)^4$

Tandem axle with dual wheels on either side $= \left(\frac{\text{axle load in kN}}{148}\right)^4$

Tridem axles with dual wheels on either side $= \left(\frac{\text{axle load in kN}}{224}\right)^4$

The summation of ESAL's of all categories giving the total number of ESAL's for total vehicles has been considered. The Vehicle damage Factor (VDF) was obtained by dividing the Total ESAL's by the number of vehicles surveyed. The VDF of TRUCK 2 Axle, TRUCK 3 AXLE and MULTI AXLE VEHICLE are calculated as shown in the tables from 5.3 to 5.5.

Table 4.3: VDF for Truck 2 Axle

MODE : TRUCK 2 Axle

AXLE LOAD CATEGORY (IN TONNES)	NO. OF AXELS IN EACH WEIGHT RANGE	LOAD EQUIVALENCY FACTORS	EQUIVALENT STANDARD AXLE LOADS (ESAL's)
< 0.9	0	0.0001	0.000

0.9-1.81	0	0.0024	0.000
1.81-2.72	0	0.0123	0.000
2.72-3.63	2	0.0392	0.078
3.63-4.54	5	0.0958	0.479
4.54-5.44	17	0.1975	3.358
5.44-6.35	48	0.3667	17.603
6.35-7.26	16	0.6266	10.025
7.26-8.16	4	1.0000	4.000
8.16-9.07	0	1.5264	0.000
9.07-9.98	14	2.2375	31.325
9.98-10.89	18	3.1721	57.098
10.89-11.79	27	4.3581	117.668
11.79-12.7	14	5.8675	82.145
12.7-13.61	2	7.7388	15.478
13.61-14.52	0	10.0255	0.000
14.52-15.42	0	12.7520	0.000
15.42-16.32	1	16.0000	16.000
16.32-17.23	0	19.8784	0.000
17.23-18.14	0	24.4224	0.000
18.14-19.051	0	29.7105	0.000
19.051-19.958	0	35.7855	0.000
19.958-20.865	0	42.7477	0.000
20.865-21.772	0	50.6795	0.000
21.772-22.680	0	59.6776	0.000
22.680-23.587	0	69.8120	0.000
23.587-24.494	0	81.1855	0.000
24.494-25.401	0	93.8950	0.000
Total	**168**		**355.257**
No. of Vehicles	**89**		
VDF	**3.99**		

Table 4.4: VDF for Truck 3 Axle

MODE : TRUCK 3 AXLE

AXLE LOAD CATEGORY (IN TONNES)	NO. OF AXELS IN EACH WEIGHT RANGE S. A.	NO. OF AXELS IN EACH WEIGHT RANGE T. A.	LOAD EQUIVALENCY FACTORS S. A.	LOAD EQUIVALENCY FACTORS T. A.	EQUIVALENT STANDARD AXLE LOADS (ESAL's)
< 0.9	0	0	0.0001	0.0000	0.000
0.9-1.81	0	0	0.0024	0.0002	0.000
1.81-2.72	0	0	0.0123	0.0011	0.000
2.72-3.63	1	1	0.0392	0.0035	0.043
3.63-4.54	2	0	0.0958	0.0085	0.192
4.54-5.44	33	0	0.1975	0.0174	6.519
5.44-6.35	98	4	0.3667	0.0324	36.068
6.35-7.26	57	5	0.6266	0.0553	35.993
7.26-8.16	18	13	1.0000	1.0000	31.000
8.16-9.07	0	0	1.5264	0.1348	0.000
9.07-9.98	49	50	2.2375	0.1976	119.519
9.98-10.89	33	30	3.1721	0.2802	113.086
10.89-11.79	14	23	4.3581	0.3849	69.867
11.79-12.7	17	10	5.8675	0.5183	104.931
12.7-13.61	10	8	7.7388	0.6836	82.856
13.61-14.52	9	8	10.0255	0.8855	97.314
14.52-15.42	2	3	12.7520	1.1264	28.883
15.42-16.32	1	1	16.0000	1.4133	17.413
16.32-17.23	0	0	19.8784	1.7558	0.000
17.23-18.14	0	0	24.4224	2.1572	0.000
18.14-19.051	0	0	29.7105	2.6243	0.000
19.051-19.958	0	0	35.7855	3.1609	0.000
19.958-20.865	0	0	42.7477	3.7759	0.000

20.865-21.772	0	0	50.6795	4.4765	0.000
21.772-22.680	0	0	59.6776	5.2713	0.000
22.680-23.587	0	0	69.8120	6.1665	0.000
23.587-24.494	0	0	81.1855	7.1711	0.000
24.494-25.401	0	0	93.8950	8.2937	0.000
25.401-26.308	0	0	108.0415	9.5432	0.000
26.308-27.216	0	0	123.7475	10.9306	0.000
27.216-28.123	0	0	141.0867	12.4621	0.000
28.123-29.03	0	0	160.1871	14.1492	0.000
29.03-29.937	0	0	181.1642	16.0021	0.000
29.937-30.844	0	0	204.1372	18.0313	0.000
30.844-31.752	0	0	229.2576	20.2502	0.000
31.752-32.66	0	0	256.6280	22.6678	0.000
32.66-33.566	0	0	286.3108	25.2897	0.000
33.566-34.473	0	0	319.5330	28.2242	0.000
Total	**344**	**156**			**743.682**
No. of Vehicles		**184**			
V D F		**4.04**			

Table 4.5: VDF for MULTI AXLE VEHICLE

MODE : MULTI AXLE VEHICLE

AXLE LOAD CATEGORY (IN TONNES)	NO. OF AXLES IN EACH WEIGHT RANGE S. A.	NO. OF AXLES IN EACH WEIGHT RANGE T. A.	LOAD EQUIVALENCY FACTORS S. A.	LOAD EQUIVALENCY FACTORS T. A.	EQUIVALENT STANDARD AXLE LOADS (ESAL's)
< 0.9	0	1	0.000	0.000	0.000

0.9-1.81	0	0	0.002	0.000	0.000
1.81-2.72	0	1	0.009	0.001	-0.001
2.72-3.63	0	1	0.031	0.003	0.003
3.63-4.54	2	0	0.080	0.006	0.160
4.54-5.44	18	0	0.176	0.013	3.168
5.44-6.35	15	0	0.350	0.024	5.250
6.35-7.26	5	0	0.610	0.043	3.050
7.26-8.16	5	0	1.000	0.070	5.000
8.16-9.07	2	0	1.550	0.110	3.100
9.07-9.98	7	0	2.300	0.166	16.100
9.98-10.89	3	0	3.270	0.242	9.810
10.89-11.79	0	0	4.480	0.342	0.000
11.79-12.7	2	0	5.980	0.470	11.960
12.7-13.61	0	0	7.800	0.633	0.000
13.61-14.52	1	0	10.000	0.834	10.000
14.52-15.42	0	1	12.500	1.080	1.080
15.42-16.32	2	3	15.500	1.380	35.140
16.32-17.23	0	4	19.000	1.730	6.920
17.23-18.14	0	6	23.000	2.140	12.840
18.14-19.051	0	2	27.700	2.510	5.020
19.051-19.958	0	5	33.000	3.160	15.800
19.958-20.865	0	1	39.300	3.790	3.790
20.865-21.772	0	3	46.500	4.490	13.470
21.772-22.680	0	4	55.000	5.280	21.120
22.680-23.587	0	3	69.812	6.170	18.510
23.587-24.494	0	2	81.185	7.150	14.300
24.494-25.401	0	3	93.895	8.200	24.600
25.401-26.308	0	2		9.400	18.800
26.308-27.216	0	1		10.700	10.700
27.216-28.123	0	1		12.100	12.100
28.123-29.03	0	0		13.700	0.000
29.03-29.937	0	0		15.400	0.000
29.937-30.844	0	0		17.200	0.000
30.844-31.752	0	0		19.200	0.000
31.752-32.66	0	1		21.300	21.300

32.66-33.566	0	0		23.600	0.000
33.566-34.473	0	0		26.100	0.000
34.473-35.38	0	0		28.800	0.000
35.38-36.288	0	0		31.700	0.000
Total	**105**	**45**			**303.09**
No. of Vehicles			**34**		
VDF			**8.91**		

4.3 Calculations for the Million Standard Axles for the year 2013 to 2028.

The Calculation of MSA is done with the traffic data and axle load survey as per IRC 37:2012. The design procedure given by IRC makes use of the CBR value, million standard axle concept, and vehicle damage factor. Traffic distribution along the lanes is taken into account. The design is meant for design traffic which is arrived at using a growth rate. Flexible pavements are considered to include the pavements which have bituminous surfacing and granular base and sub-base courses conforming to IRC/ MOST standards. These guidelines apply to new pavements.

Table 5.6: MSA Calculations

Year	Traffic					Traffic Growth Rate (%)	Lane Distribution	Vehicle Damage Factor					Traffic (CVD)	(365 * ((1+r)^n -1))/r	Million Standard Axle	
	Bus	LCV + Mini Bus	2 - Axle	3 - Axle	MAV			Bus	LCV + Mini Bus	2 - Axle	3 - Axle	MAV			MSA	Cumulative (MSA)
Present Year - 2010	123	336	791	1365	183								2798			
Base Year - 2013	153	417	983	1696	227	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	3476	1179.18	11.82	11.82
Year - 2014	164	448	1057	1823	244	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	3736	365.00	3.93	15.76
Year - 2015	176	482	1136	1960	262	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	4016	365.00	4.23	19.98
Year - 2016	189	518	1221	2107	282	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	4317	365.00	4.55	24.53
Year - 2017	203	557	1313	2265	303	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	4641	365.00	4.89	29.42
Year - 2018	218	599	1411	2435	326	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	4989	365.00	5.25	34.67
Year - 2019	234	644	1517	2618	350	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	5363	365.00	5.65	40.32
Year - 2020	252	692	1631	2814	376	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	5765	365.00	6.07	46.38
Year - 2021	271	744	1753	3025	404	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	6197	365.00	6.52	52.91
Year - 2022	291	800	1884	3252	434	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	6661	365.00	7.01	59.92
Year - 2023	313	860	2025	3496	467	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	7161	365.00	7.54	67.46
Year - 2024	336	925	2177	3758	502	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	7698	365.00	8.10	75.56
Year - 2025	361	994	2340	4040	540	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	8275	365.00	8.71	84.27
Year - 2026	388	1069	2516	4343	581	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	8897	365.00	9.37	93.64
Year - 2027	417	1149	2705	4669	625	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	9565	365.00	10.07	103.71
Year - 2028	448	1235	2908	5019	672	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	10282	365.00	10.83	114.54
Year - 2029	**482**	**1328**	**3126**	**5395**	**722**	**7.50%**	**0.75**	**1.00**	**1.00**	**3.99**	**4.04**	**8.91**	**11053**	**365.00**	**11.64**	**126.18**
Year - 2030	518	1428	3360	5800	776	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	11882	365.00	12.51	138.69
Year - 2031	557	1535	3612	6235	834	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	12773	365.00	13.45	152.14
Year - 2032	599	1650	3883	6703	897	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	13732	365.00	14.46	166.59

Year - 2033	644	1774	4174	7206	964	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	14762	365.00	15.54	182.14
Year - 2034	692	1907	4487	7746	1036	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	15868	365.00	16.71	198.84
Year - 2035	744	2050	4824	8327	1114	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	17059	365.00	17.96	216.80
Year - 2036	800	2204	5186	8952	1198	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	18340	365.00	19.31	236.11
Year - 2037	860	2369	5575	9623	1288	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	19715	365.00	20.76	256.87
Year - 2038	925	2547	5993	10345	1385	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	21195	365.00	22.32	279.19
Year - 2039	994	2738	6442	11121	1489	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	22784	365.00	23.99	303.18
Year - 2040	1069	2943	6925	11955	1601	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	24493	365.00	25.79	328.96
Year - 2041	1149	3164	7444	12852	1721	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	26330	365.00	27.72	356.69
Year - 2042	1235	3401	8002	13816	1850	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	28304	365.00	29.80	386.49
Year - 2043	1328	3656	8602	14852	1989	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	30427	365.00	32.04	418.53
Year - 2044	1428	3930	9247	15966	2138	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	32709	365.00	34.44	452.97
Year - 2045	1535	4225	9941	17163	2298	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	35162	365.00	37.02	489.99
Year - 2046	1650	4542	10687	18450	2470	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	37799	365.00	39.80	529.78
Year - 2047	1774	4883	11489	19834	2655	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	40635	365.00	42.78	572.57
Year - 2048	1907	5249	12351	21322	2854	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	43683	365.00	45.99	618.56
Year - 2049	2050	5643	13277	22921	3068	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	46959	365.00	49.44	668.00
Year - 2050	2204	6066	14273	24640	3298	7.50%	0.75	1.00	1.00	3.99	4.04	8.91	50481	365.00	53.15	721.15

4.4. Calculation of Pavement Thicknesses

Table 4.7: Pavement Composition for Soil Sample CBR 6% without Cement

PAVEMENT COMPOSITION AS PER IRC:37-2012					
Soil Sample CBR 6% Without Cement					
CASE - I		CBR	6%	MSA	126.180
		DESIGN PERIOD	15 YEARS	VDF	As Per VDF Calculations
Pavement Thickness & Composition					
MSA	PAVEMENT THICKNESS (mm)				
126.180	692.500	(Required Pavement Thickness)			
Proposed Pavement Thickness					
GSB	**WMM**	**DBM**	**BC**	**TOTAL PAVEMENT**	
260	**250**	**132.5**	**50**	**692.5**	

Table 4.8: Pavement Composition for Soil Sample with Cement addition 2%

PAVEMENT COMPOSITION AS PER IRC:37-2012					
With Cement addition 2%, CBR 7.98 % Say 7%					
CASE - II		CBR	7%	MSA	126.180
		DESIGN PERIOD	15 YEARS	VDF	As Per VDF Calculations
Pavement Thickness & Composition					
MSA	PAVEMENT THICKNESS (mm)				
126.180	625.000	(Required Pavement Thickness)			
Proposed Pavement Thickness					
GSB	**WMM**	**DBM**	**BC**	**TOTAL PAVEMENT**	
230	**250**	**131**	**50**	**661**	

Table 4.9: Pavement Composition for Soil Sample with Cement addition 4%

PAVEMENT COMPOSITION AS PER IRC:37-2012					
With Cement addition 4%, CBR 8.76 % Say 8%					
CASE - III		CBR	8%	MSA	126.180
		DESIGN PERIOD	15 YEARS	VDF	As Per VDF Calculations
Pavement Thickness & Composition					
MSA	PAVEMENT THICKNESS (mm)				
126.180	617.500	(Required Pavement Thickness)			
Proposed Pavement Thickness					
GSB	**WMM**	**DBM**	**BC**	**TOTAL PAVEMENT**	
200	**250**	**126**	**50**	**626**	

Table 4.10: Pavement Composition for Soil Sample with Cement addition 8%

PAVEMENT COMPOSITION AS PER IRC:37-2012					
With Cement addition 8% and 12%, CBR 9.54 % and 9.86 Say 9%					
CASE - III		CBR	9%	MSA	126.180
		DESIGN PERIOD	15 YEARS	VDF	As Per VDF Calculations
Pavement Thickness & Composition					
MSA	PAVEMENT THICKNESS (mm)				
126.180	617.500	(Required Pavement Thickness)			
Proposed Pavement Thickness					
GSB	**WMM**	**DBM**	**BC**	**TOTAL PAVEMENT**	
200	**250**	**117.5**	**50**	**617.5**	

Table 4.11: Pavement Composition for Soil Sample with Cement addition 15%

PAVEMENT COMPOSITION AS PER IRC:37-2012					
With Cement addition 15%, CBR 4.31 % Say 4%					
CASE - III		CBR	4%	MSA	126.180
		DESIGN PERIOD	15 YEARS	VDF	As Per VDF Calculations
Pavement Thickness & Composition					
MSA	PAVEMENT THICKNESS (mm)				
126.180	617.500	(Required Pavement Thickness)			
Proposed Pavement Thickness					
GSB	**WMM**	**DBM**	**BC**	**TOTAL PAVEMENT**	
330	**250**	**147**	**50**	**677**	

Table 4.12: Pavement Composition for Soil Sample CBR 3% without Lime

PAVEMENT COMPOSITION AS PER IRC:37-2012					
Soil Sample CBR 3% Without Lime					
CASE - I		CBR	3%	MSA	126.180
		DESIGN PERIOD	15 YEARS	VDF	As Per VDF Calculations
Pavement Thickness & Composition					
MSA	PAVEMENT THICKNESS (mm)				
126.180	842.500	(Required Pavement Thickness)			
Proposed Pavement Thickness					
GSB	**WMM**	**DBM**	**BC**	**TOTAL PAVEMENT**	
380	**250**	**162.5**	**50**	**842.5**	

Table 4.13: Pavement Composition for Soil Sample with Lime addition 2%

PAVEMENT COMPOSITION AS PER IRC:37-2012					
With Lime addition 2%, CBR 6.52 % Say 6%					
CASE - II		CBR	6%	MSA	126.180
		DESIGN PERIOD	15 YEARS	VDF	As Per VDF Calculations
Pavement Thickness & Composition					
MSA	PAVEMENT THICKNESS (mm)				
126.180	692.500	(Required Pavement Thickness)			
Proposed Pavement Thickness					
GSB	**WMM**	**DBM**	**BC**	**TOTAL PAVEMENT**	
260	**250**	**132.5**	**50**	**692.5**	

Table 4.14: Pavement Composition for Soil Sample with Lime addition 4%

PAVEMENT COMPOSITION AS PER IRC:37-2012					
With Lime addition 4%, CBR 8.96% Say 9%					
CASE - III		CBR	8%	MSA	126.180
		DESIGN PERIOD	15 YEARS	VDF	As Per VDF Calculations
Pavement Thickness & Composition					
MSA	PAVEMENT THICKNESS (mm)				
126.180	626	(Required Pavement Thickness)			
Proposed Pavement Thickness					
GSB	**WMM**	**DBM**	**BC**	**TOTAL PAVEMENT**	
200	**250**	**126**	**50**	**626**	

Table 4.15: Pavement Composition for Soil Sample with Lime addition 10%

PAVEMENT COMPOSITION AS PER IRC:37-2012					
With Lime addition 10%, CBR 9.12% Say 9%					
CASE - III		CBR	9%	MSA	126.180
		DESIGN PERIOD	15 YEARS	VDF	As Per VDF Calculations
Pavement Thickness & Composition					
MSA	PAVEMENT THICKNESS (mm)				
126.180	617.500	(Required Pavement Thickness)			
Proposed Pavement Thickness					
GSB	**WMM**	**DBM**	**BC**	**TOTAL PAVEMENT**	
200	**250**	**117.5**	**50**	**617.5**	

Chapter 5
Conclusions

5.1. General

The major conclusions drawn at the end of this work are as follows:

1. With addition of stabilizers i.e. cement and lime, the C.B.R. increases upto a certain limit but after that the C.B.R. value decreases even on further addition of stabilizers.
2. As in the case of cement stabilization, the C.B.R. increases up till addition of 8% cement content but on further increase in cement content i.e. 12% there is hardly any increase in value of C.B.R. and on further addition of cement content i.e. 15%, the value of C.B.R. reduces drastically.
3. Similarly, in the case of lime stabilization, the C.B.R. value first increases upto a certain limit and after that the value decreases with further addition of lime.

5.2. Pavement Thickness

The thickness of crust varies with the change in the value of C.B.R. With higher value of C.B.R. the crust thickness is less and vice versa. Below shown are the crust thicknesses with different percentages of cement and lime content.

5.2.1. In Case Of Cement Stabilization

Table 5.1 Crust thickness with different percentages of Cement addition

S.No	Description	Layers	Layer Thickness (mm)
1	Soil Sample CBR 6%	GSB	260
		G. Base	250
		DBM	132.5
		BC	50
		TOTAL	**692.5**
2	With Cement addition 2%, CBR 7.98 % Say 7%	GSB	230
		G. Base	250
		DBM	131

		BC	50
		TOTAL	**661**
3	With Cement addition 4%, CBR 8.76 % Say 8%	GSB	200
		G. Base	250
		DBM	126
		BC	50
		TOTAL	**626**
4	With Cement addition 8% and 12%, CBR 9.54 % and 9.86% Say 9%	GSB	**200**
		G. Base	**250**
		DBM	**117.5**
		BC	**50**
		TOTAL	**617.5**
5	With Cement addition 15%, CBR 4.31 % Say 4%	GSB	**330**
		G. Base	**250**
		DBM	**147**
		BC	**50**
		TOTAL	**677**

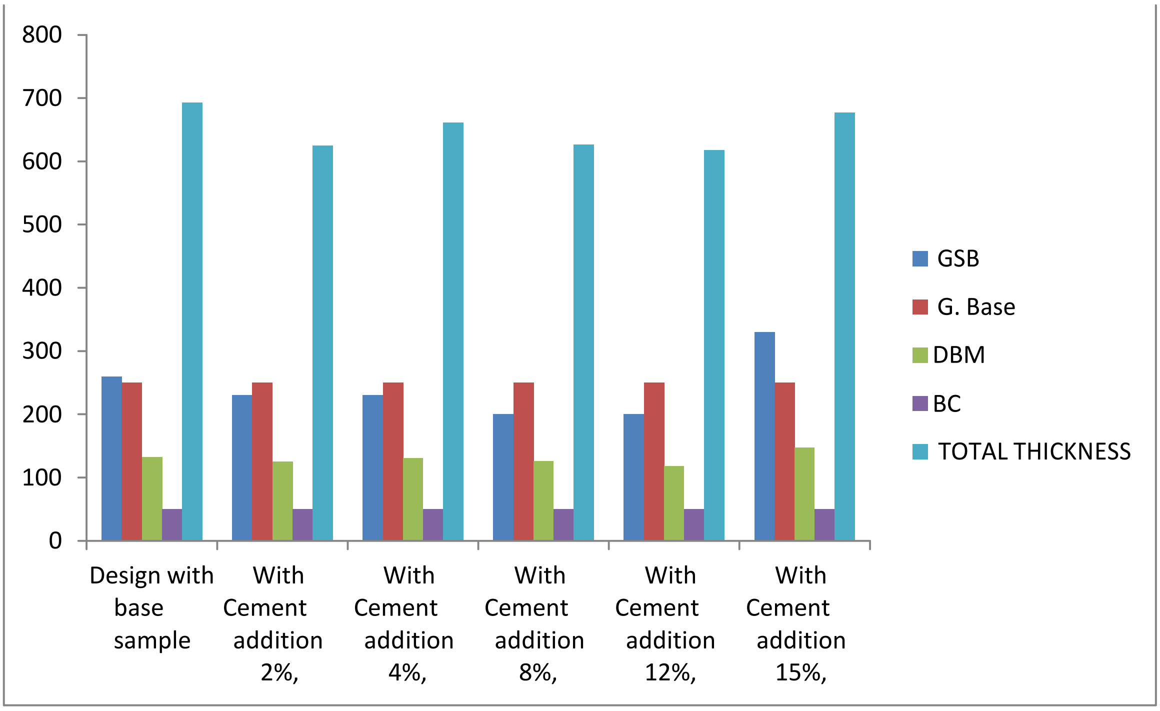

Figure 5.1: Crust thickness with different percentages of Cement addition

5.2.2. In Case Of Lime Stabilization

Table 5.2 Crust thickness with different percentages of Lime addition

S.No	Description	Layers	Layer Thickness (mm)
1	Soil Sample CBR 3%	GSB	380
		G. Base	250
		DBM	162.5
		BC	50
		TOTAL	**842.50**
2	With Lime addition 2%, CBR 6.52 % Say 6%	GSB	260
		G. Base	250
		DBM	132.5
		BC	50
		TOTAL	**692.50**
3	With Lime addition 4%, CBR 8.96%, Say 8%	GSB	200
		G. Base	250
		DBM	126
		BC	50
		TOTAL	**626**
3	With Lime addition 10%, CBR 9.12% Say 9%	GSB	200
		G. Base	250
		DBM	117.5
		BC	50
		TOTAL	**617.50**

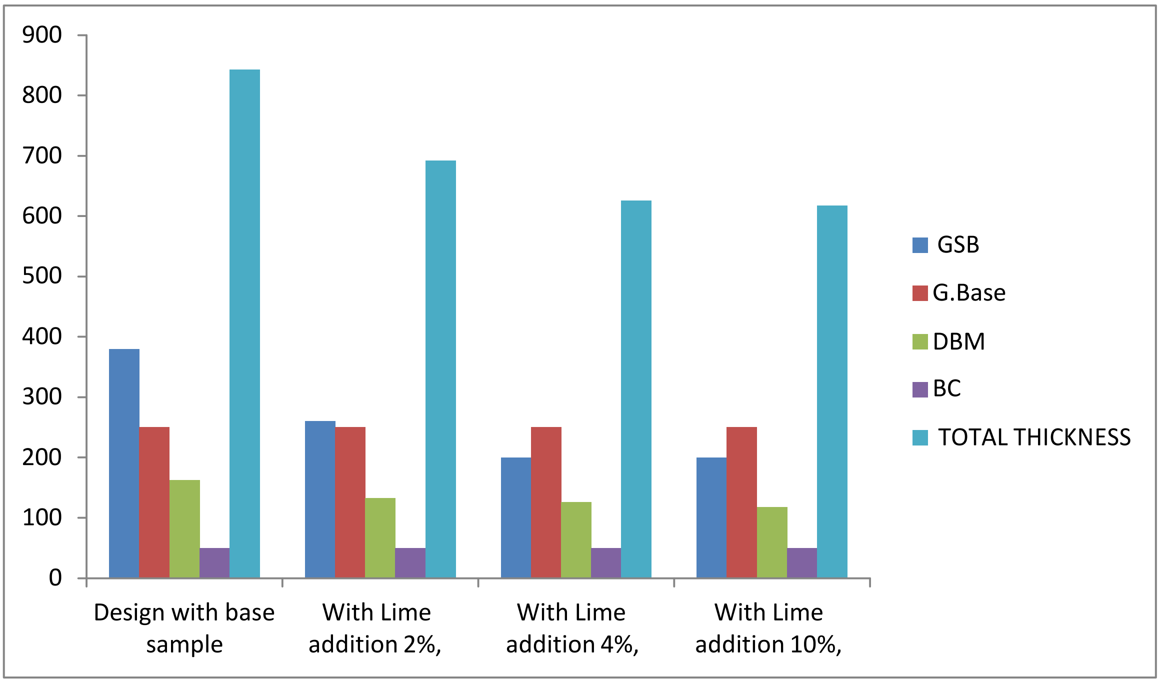

Figure 5.2: Crust thickness with different percentages of Lime addition

5.3. Saving In Pavement Thickness

The thickness of crust varies with the change in the value of C.B.R. With higher value of C.B.R. the crust thickness is less and vice versa. Below shown are the savings in crust thicknesses with different percentages of cement and lime content.

5.3.1. In Case Of Cement Stabilization

Table 5.3 Saving in the crust thickness with addition of Cement

S.No	Description	Saving in Thickness (mm)
1	Soil Sample CBR 6% without cement	Design with Base Sample
2	With Cement addition 2%, CBR 7.98 % Say 7%	30
		0
		1.50
		0

		31.50
3	With Cement addition 4%, CBR 8.76 % Say 8%	60
		0
		6.5
		0
		66.50
4	With Cement addition 8% and 12%, CBR 9.54 % and 9.86% Say 9%	60
		0
		15
		0
		75
5	With Cement addition 15%, CBR 4.31 % Say 4%	-70
		0
		-14.50
		0
		-84.50

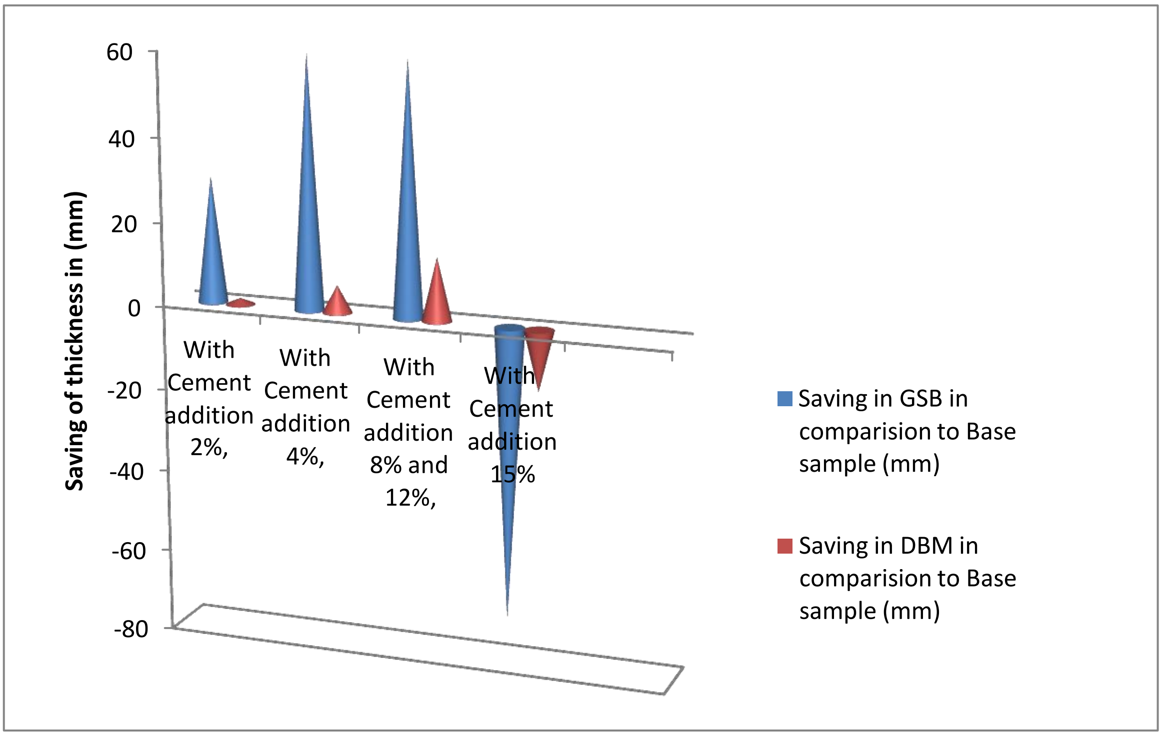

Figure 5.3: Saving in Layer Composition in comparison to Base sample (Cement Stabilization)

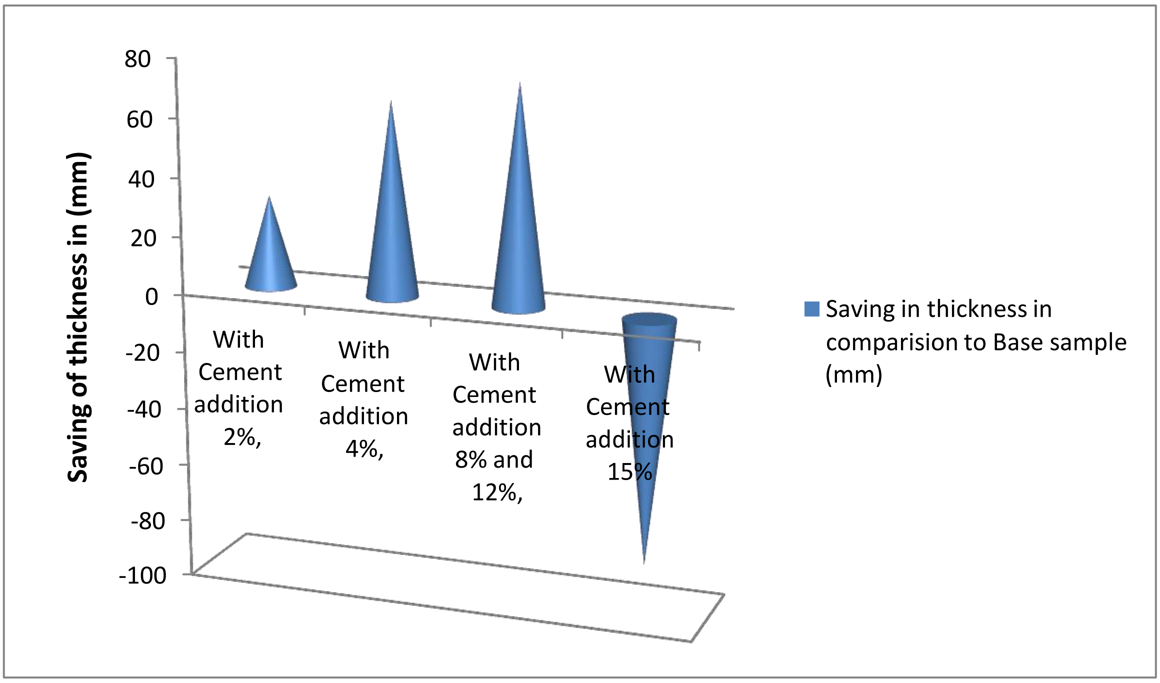

Figure 5.4: Saving in Total Crust in comparison to Base sample (Cement Stabilization)

5.3.2. In Case Of Lime Stabilization

Table 5.4 Saving in the crust thickness with addition of Lime

S.No	Description	Saving in Thickness (mm)
1	Soil Sample CBR 3%	Design with Base Sample
2	With Lime addition 2%, CBR 6.52 % Say 6%	120
		0
		30
		0
		150
3	With Lime addition 4%, CBR 8.96%, Say 8%	180
		0
		36.5

		0
		216.50
4	With Lime addition 10%, CBR 9.12% Say 9%	180
		0
		45
		0
		225

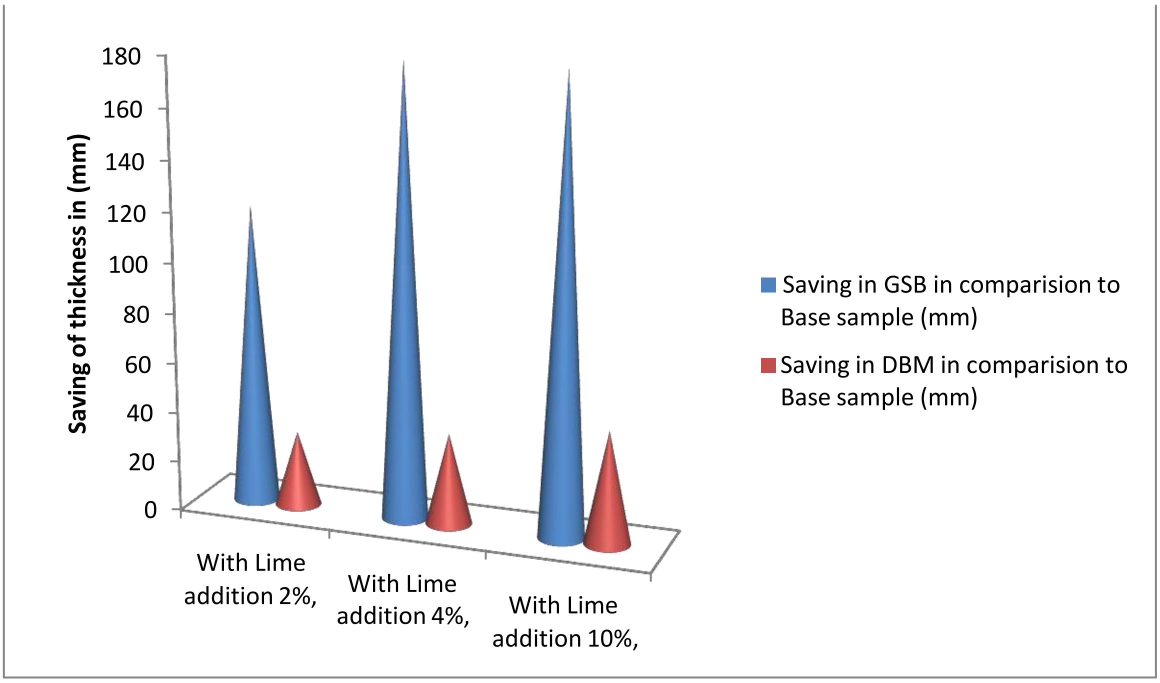

Figure 5.5: Saving in Layer Composition in comparison to Base sample (Lime Stabilization)

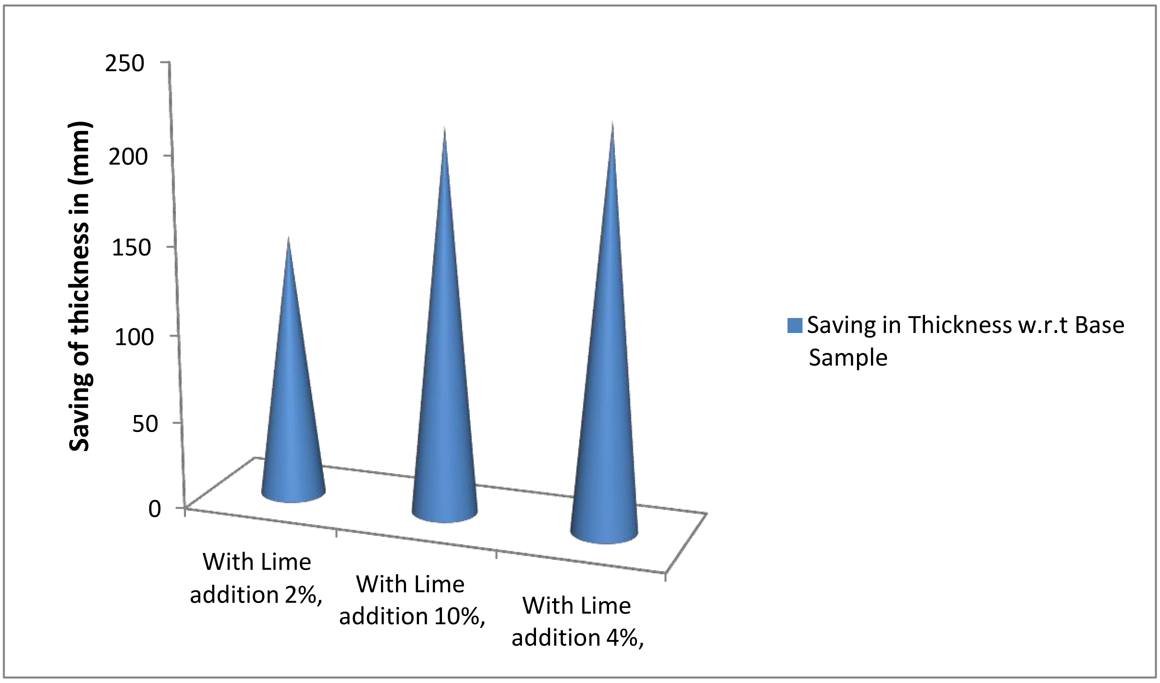

Figure 5.6: Saving in Total Crust in comparison to Base sample (Lime Stabilization)

5.4. Scope for Further Work

In the field of stabilization of sub grades, there is a lot of scope for further work. Similar stabilizations can be done using various other different materials available, most important being RBI Grade 81. It is a very new patented material and has a large scope in research work. Stabilizations can be performed on different type of soils. The stabilization can also be done with different combinations of stabilizers like cement and lime mixed together.

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