COMPARATIVE STUDY OF FLY ASH STABILIZED BY LIME AND GYPSUM/PHOSPHOGYPSUM BINDER ADMIXTURES SUB BASES FOR RURAL ROADS

A PROJECT

Submitted in partial fulfillment of the requirements for the award of the degree of

BACHELOR OF TECHNOLOGY

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CERTIFICATE

We hereby certify that the work which is being presented in the project titled **"COMPARATIVE STUDY OF FLY ASH STABILIZED BY LIME AND GYPSUM/PHOSPHOGYPSUM BINDER ADMIXTURES SUB BASES FOR RURAL ROADS"** in partial fulfillment of the requirements for the award of the Degree of Bachelor of Technology and submitted in Civil Engineering Department, Jaypee University of Information Technology, Waknaghat, is an authentic record of work carried by **Mr. Shaurya Sood (111659)** and **Mr. Rishav Chauhan (111645)** out during a period from **August 2014** to **May 2015** under the supervision of **Mr. Saurabh Rawat** , Assistant Professor, Civil Engineering Department, Jaypee University of Information Technology, Waknaghat.

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ABSTRACT

This major project report presents the comparative study of durability characteristics of a class F fly ash stabilized with lime and modified with gypsum/phosphogypsum. The content of lime, gypsum and phosphogypsum was varied from 2 to 14 %, 0.5 to 2 % and 0.5 to 4 % respectively.

The curing period was varied from 7 to 28 days. The results of this study reveals that the durability characteristics of the fly ash-lime mix modified with gypsum/phosphogypsum improved with dry and wet cycles. Further, the durability characteristics improve with the increase in curing period.

The results further reveal that the improvement in durability of fly ash-lime-gypsum was more in comparison to the fly ash-lime-phosphogypsum mixes.

The improved durability characteristics make the materials suitable for use in base/sub base courses in road pavements.

Three pavement models: (i) without fly ash – lime - gypsum/phosphogypsum admixture, (ii) with fly ash – lime - gypsum admixture, (iii) with fly ash – lime – phosphogypsum admixture have been developed to gain an insight into the effectiveness of load bearing capacity of each model.

Keywords: Fly ash, Lime, gypsum, phosphogypsum, durability, pavement model

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LIST OF ABBREVIATIONS AND SYMBOLS

UCS	Unconfined Compressive Strength
RUCS	Residual Unconfined Compressive Strength
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
LMA	Lime Fly Ash Aggregate
WBM	Water Bound Macadam
G	Gypsum
PG	Phosphogypsum
PM	Pavement model
L	Lime
SEM	Scanning Electron Microscope
ρ_d	Dry Density
ε	Strain

CHAPTER 1

INTRODUCTION

1.1 General

Many procedures have been developed to improve the physical behavior of soil by incorporating a wide range of stabilizing agents, additives and conditioners. The effectiveness of such agents relies on the formation of cementing bonds between the particles in the soil system. The two most common stabilizing agents are cement and lime. Soil stabilization has been widely recommended for developing countries for the construction of various elements of the pavements. The reasons usually put forward are that the use of locally available materials will lead to lower costs. Lime as an additive, brings several beneficial changes in the engineering properties of soil such as decrease in soil plasticity and shrink swell potential apart from improving strength characteristics. Stabilization of soil by lime is achieved through cation exchange, flocculation and agglomeration, lime carbonation and pozzolanic reaction. Cation exchange and flocculation agglomeration reaction takes place rapidly and brings immediate change in soil properties, whereas, pozzolanic reactions are time dependent. These reactions involve interaction between soil silica and (or) alumina and lime to form various types of cementing agents thus enhancing the strength. Certain natural substance, such as volcanic ash reacts to the lime addition much better than do the ordinary soil types. If such materials are added to soil, the efficiency of lime stabilization may be greatly increased. The characteristics of compacted soil, if improved, resulting from residue utilization like fly ash, blast furnace slag, rice husk ash etc. can be a practical way of encouraging sustainable development apart from environmental and economic benefits. However use of such industrial wastes has not found wide application in civil engineering construction activities. The existing literature also suggests that the maximum amount of fly ash to be used in a mix should be around 25% (Consoli et al. 2001). In order to ensure bulk utilization of fly ash there is an urgent need to find out ways and means to use higher volume of fly ash in the mix.

Most developed and developing countries all over the world have huge resources of waste materials such as fly ash, phosphogypsum. The quantities of wastes that are accumulating in developed and developing countries are causing disposal problems that are both financially and environmentally expensive. One method to reduce some portion of the waste disposal problem is by utilizing these waste materials for engineering purposes. In India, extensive road network is under construction. On the other hand, there is scarcity of materials especially for the road bases/sub bases. Civil engineers around the world are in search of new alternative materials which are required both for cost effective solution for roads and for conservation of scarce natural resources. In this context, fly ash–lime–gypsum/phosphogypsum hold promise as alternate materials for use in roads.

Development of an adequate network of roads, especially in remote rural areas is of vital importance towards the social and economic upliftment of the villages in the country. The traditional specifications for construction of low volume roads in the country comprise stone/brick soling or WBM layer in the sub base course. This is followed by a layer of WBM as base and is, generally, topped with bituminous surfacing. Due to haulage of stone material over long distances, the cost of road construction has become prohibitive. It is, therefore, felt that endeavour should be coarse, loose, well-rounded water-worn detritus or alluvial material of various sizes generally measuring 20-200 mm in diameter formed due to river action. Initial cost of construction is much less in the process of stage construction, the overall costs per ton basis is high for low trafficked roads (village roads) than for highly trafficked roads. This is because of iron tired traffic on village roads necessitates the provision of a minimum thickness of hard wearing surface on the base. Aggregates containing a significant proportion of plastic fines, are treated with lime and other admixtures to reduce the plasticity and improve strength, volumetric stability and durability. Some studies on high grade materials have shown that the response of granular material under repeated loading is markedly non-linear which means that no unique value of modulus exists, i.e., that the latter varies with applied stress level. This evaluation of stress dependent, non-linear stiffness is particularly significant for design of granular mixes for Indian conditions where a major part of the pavement structure is comprised of unbound granular material. With the above requirements a research investigation has been formulated to examine some of the stress-strain characteristics of naturally occurring aggregate gravel.

1.2 Chapter Outline

The project report is presented in five Chapters. Brief details about each chapter are as follows:

Chapter 1: Introduction

This chapter gives an introduction of fly ash stabilization necessities for ground improvement purposes.

Chapter 2: Literature review

This chapter presents a brief review of relevant literature of the work carried out by various investigators. The large amount of literature available on the effect of lime for fly ash stabilization is summarized. The need for a detailed investigation to understand the UCS and durability behaviors of fly ash stabilized by lime and gypsum/phosphogypsum at various percentage content is identified. The research work done on development of pavement models with gravel and fly ash-lime-gypsum/phosphogypsum admixtures is discussed in this chapter.

Chapter 3: Methodology

This chapter describes the various experiments done on the fly ash, lime and gypsum/phosphogysum combinations to prepare reference mixes based on OMC and MDD values. To study the compression behavior the unconfined compression strength test were conducted on these reference mixtures. The UCS is conducted on the specimen of size 38 mm × 76 mm at 7, 14 and 28 days curing periods. The drying-wetting cycles have also been incorporated to study the durability behavior of the sample followed by UCS measurement once again (termed as Residual UCS) and a comparison between the two *i.e.* UCS v/s RUCS has been made. The construction of pavement models by development of each layer is discussed.

Chapter 4: Results and Discussions

The results of the fly ash, lime and gypsum/phosphogypsum reference mixes cured at 7, 14 and 28 days on the compression behavior so as to study the axial stress and axial strain have been presented in this chapter. The effect of repetitive wetting-drying cycles have also been presented in results. In addition to this, the comparison of UCS & RUCS measurement for gypsum (G) and phosphogypsum (PG) as modifier is also presented in this chapter. The load bearing capacity and settlement of layers of each model is discussed.

Chapter 5: Conclusions

This chapter presents an overall summary of the work carried out and brings out the salient conclusions. The potential application of fly ash stabilized by lime and gypsum/phosphogypsum admixtures is highlighted. In addition to this, future scope of the work has also been highlighted.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Fly ash produced by thermal power plants takes huge disposal area and creates environmental problems like leaching and dusting. It is collected by mechanical or electrostatic precipitators from the flue gases of power plants. Presently, in India, extensive road network is under construction. In some of the road projects, attempt has been made to use pond ash as a construction material, solution to the scarcity of conventional construction material, and disposal of fly ash. The major problems the world is facing today are the scarcity of conventional construction material on one hand while on the other hand, large amount of unutilized industrial wastes causing serious environmental problems and ecological imbalance. Utilization of fly ash in construction, such as embankments and structural fills and dykes, is the most promising solution to the problem of the disposal of fly ash and also to reduce the construction cost of the projects. Previous researchers studied different uses of fly ash such as bulk fill material (Raymond 1958; DiGioia and Nuzzo 1972), soil stabilization (Vasquez and Alonso 1981), and land reclamation (Kim and Chun 1994). Potential application of fly ash alone or soil stabilized with fly ash or fly ash and admixtures for road construction has been reported by a number of researchers (Ghosh et al. Reddy and Rama Moorthy 2004; Ghosh and Subbarao 2006). Jute-geotextile reinforcing fly ash was found to be a promising technique to improve the bearing capacity of the foundation medium (Ghosh et al. 2005). Fly ash has found potential application in the construction field because of its self-hardening characteristics which depends on the availability of lime. Gypsum has also been used to stabilize fly ash (Pandian 2004).

2.2 Studies on Durability

Durability which can be defined as the ability of a material to retain stability and integrity over years of exposure to the destructive forces of weathering is one of the most important properties (Dempesy and Thompson 1968).The durability tests on soil-fly ash-lime mixture were conducted as per IS: 4332-1968 and reaffirmed in 1995. For these tests, specimens were prepared at the maximum dry density and optimum moisture content and then moist cured for a specific number of days. Subsequently, specimens were immersed in water for 5 hours followed by air drying for

42 hours at room temperature, which completes single cycle of wetting and drying. After each cycle, the specimens were brushed with a steel wire brush and the loss in the material is recorded as mass loss (brush loss) in percentage. Further triplicate sets of samples were prepared following the same standard test procedure, and were subjected to 12 cycles of wetting and drying but brushing was omitted. Brushing of specimens has been known to cause uncertainty in the results because it is manual and hence could very well be affected by the consistency of technician's procedure. Replacing brushing by measuring the compressive strength of specimens after they are subjected to the 12 cycles of wetting-drying could provide a more consistent and convenient measures of the deterioration of the mix. Shihata and Baghdadi (2001) also suggested using the residual compressive strength of durability specimens without brushing as an indicator of resistance potential since it gives more consistent results. Thus the samples prepared without brushing were tested for unconfined compressive strength. This compressive strength was levelled as unbrushed residual strength (URS). The aim of conducting compressive strength test without brushing is to explore the possibility of using residual (compressive) strength of soil-fly ash-lime mix as a viable indicator of durability resistance. The durability indices [unbrushed residual strength ratio (URSR) in the present study] of the specimens were obtained as a ratio of the compressive strength after 12 wetting and drying cycles without brushing divided by the compressive strength of a sample prepared simultaneously, but stored under wet conditions during the entire test period.

2.3 For fly ash-lime-gypsum/phosphogypsum binder admixture development:

Ghosh and Subbarao (2001) reported that an addition of 1% gypsum increased the strength within a short curing and period. The test results presented indicate that the strength has increased by three and 22 times in comparison with that of strength compared to a mix without gypsum after 7 days of curing. The strength of fly ash, stabilized with 10% lime and 1% gypsum, has reached a value of 6,307 kPa at 3 months' curing. The addition of 10% lime along with 1% gypsum to fly ash increased the slake durability indexes up to 98% for three months curing (Ghosh 1996). Sivapullaiah and Ali Baig, 2011 reported that the strength of low lime-fly ashes which increases with lime content is significant up to an optimum lime content of about 5% and proceeds gradually thereafter. Addition of gypsum increases the strength of fly ashes at any lime content. At lower curing the increase in strength with gypsum is quite significant. Fly ash which responds readily to lime stabilization shows accelerated gain in strength due to the addition of gypsum at early curing periods. Behera, Kumar and Mishra, 2012 reported that compressive strengths of mine overburden stabilized with 15, 20, 25, 30, 35, 40, 45, and 50% fly ash were 0.71–3.14 MPa after 7, 28 and 56 days of curing. Tensile strengths of mine overburden and fly ash (15, 20, 25, 30, 35, 40, 45 and 50%) mixes stabilized with 2, 3 6 and 9% of lime. Brazilian tensile strength test results were 55.7–291 kPa and 73–357 kPa at 28 and 56 days of curing respectively. In unsoaked condition, the bearing ratio of overburden stabilized with 35% fly ash was more as compared to that of other mixes by 50%. But, in case of soaked condition, the CBR value of 15% fly ash was higher than that for other mixes by 78%.

One of the most important properties that the stabilized mixes should have is the ability to retain its strength over the year when exposed to the destructive forces of weather. One of the most commonly used durability tests on stabilized mixes in a non-frost area is wetting and drying test. Dempesy and Thompson (1973) highlighted that durability of stabilized specimens is one of the most important property. Ghosh and Subbarao (2001) reported that durability is an important property to be studied for any construction material. Many investigators studied the durability of stabilised materials through slake durability tests (Franklin and Chandra 1972; ISRM 1981). Ghosh (1996) reported an increase in slake durability indexes up to 98 % for three months' curing with the addition of 10 % lime along with 1% gypsum to fly ash. Shihata and Baghdadi (2001) reported that the residual compressive strength of durability specimens without brushing gives more consistent results. Jha et al. (2009) reported that addition of fly ash to soil-lime mixture increases the durability. They further reported that the durability requirement was satisfied after 28 days curing with a lime content of 10 %. Kumar (2002) reported that addition of phosphogypsum in fly ash-lime mixes improves the durability characteristics. Ghosh and Subbarao (2006) conducted study on the slake durability characteristics of class F fly ash stabilized with lime alone or in combination with gypsum. The effects of lime content (4, 6, and 10 %), gypsum content (0.5 and 1.0%), and curing period (up to 90 days) on the durability characteristics of the stabilized fly ash were studied. Study reported that unstabilized fly ash samples and samples stabilized with only 4 % lime does not even last for first cycle at all the curing periods. Whereas, additions of small percentages of gypsum (0.5 and 1.0%) in fly ash-lime mixes improved the durability characteristics at 28 days of curing. Mishra and Karanam (2006) conducted slake durability test on the fly ash, lime and gypsum mixes in order to assess the resistance offered by a fly ash mix to weakening and disintegration when subjected to two standard cycles of drying and cooling on specimen cured for 28 and 56 days. The studies reported that slake durability index for fly ash mix without gypsum varied from 96 % to 97 % during the 1st cycle for 28 days cured specimen and increased marginally for 56 days cured specimen. They further reported that considerable increase in durability at both the curing period with the addition of 5 % gypsum to the fly ash-lime mix. Singh and Garg (2007) reported the effect of temperature on the durability study on the fly ash mixed with fluorogypsum, hydrated lime sludge mixed with/ without Portland cement. The study concluded a decrease in the durability with the increase in temperature. Sivapullaiah and Moghal (2011) conducted wetting and drying test on fly ash-lime-gypsum mixes and reported no significant reduction in the strength of the fly ash-lime-gypsum mixes even after 20 cycles of wetting and drying.

The past studies on durability of fly ash have been conducted with lime as a stabilizer and gypsum/phosphogypsum as modifiers but none of them have attempted a comparative studies on durability with these modifiers. **Therefore, the present study tries to fill this gap with the comparative studies of durability of fly ash stabilized with lime and modified with gypsum/phosphogypsum.**

2.4 For pavement model development:

Sharma et al. calculated overall costs of road for various alternative stage construction for each type of traffic pattern. Stage construction means providing layers of increased strength, in design of surfacing only. Review of the results indicated that the initial cost of bitumen stabilization is excessively high due to high cost of bitumen. Initial costs of other type of stabilization are almost same. The initial cost of construction is quite high for cement concrete for village roads.

According to Lo study, an all weather low cost road is cheap not only in initial construction but also in subsequent maintenance. The cost per ton mile should be least. The objective is to construct large mileage of roads with limited amount of funds.

According to Lees and Bindra unbound granular mixes for road construction cover a variety of locally available naturally occurring and artificially prepared aggregates. The material should be clean and free from organic or other deleterious contents, so as to use in granular pavements.

Bhasin et al. conducted some studies on use of low grade materials for road construction in rural areas and found that locally available low grade materials like soil-gravel mixes could be effectively utilized for rural roads construction.

Kumar had done extensive work on gravel and found that it can be used successfully used in base course construction instead of crushed stones. According to authors, gravels can be stabilized in a better way by mixing some proportion of stone screenings and sand. For the size of gravel which is used in the present study, the proportion of gravel: stone screenings: sand is 4.5: 2.5:1 by weight, as for this proportion density achieved is maximum.

Where stone screenings are not available, fine shingles (passing 4.75 mm - retained 0.075 mm) can be used with proportion of gravel: fine shingle: sand as 3:1.5:1 by weight. But in this case surface deflection is 5 per cent more that for previous one. Shingle: sand in proportion 2.5:1 can also be used in WBM base course construction but surface deflection is 8 per cent higher than that for previous one. Greater deflection values have been observed for pavement models constructed using shingles than that for pavement model constructed using crushed stones. If shingle is used in base course construction the pavement thickness is required is less, it is economical to use shingle in base course construction. Considerable saving in cost is achieved when shingle: screening: sand in proportion of 4.5:2.5:1 are used. The cost of pavement will decrease further if the shingle is locally available.

Therefore, the present study also tries to use locally available gravel and the two developed binders (fly ash-lime-gypsum/phosphogypsum) to construct pavement models and thereby calculate the load bearing capacity and settlement in layers of models.

From the literature review, the following objectives of the study are derived.

2.5 Objectives of the Study

The objective of present study is therefore to evaluate the effectiveness of using large fraction class F fly ash as a pozzolan to enhance the lime treatment of soil. Mixture possessing large fraction of ash were used in the testing program. Test specimens were subjected to compaction tests and unconfined compression tests. In any stabilization application since the stabilized material should have the ability to retain its integrity and strength under in service condition, wet-dry tests were also conducted to evaluate the durability aspects of the specimen. Since pozzolanic reactions between lime and clay/fly ash particles is a time dependent chemical reaction, effect of curing period on these soil-fly ash-lime mixes were also studied in the present investigation. Specimens were cured for 7, 14 and 28 days before testing.

With the above in view, the present studies was planned to make a comparative study of the fly ash-lime-gypsum/phosphogypsum mix with variation in curing periods. The durability behaviour of these wastes was examined thoroughly through compaction, unconfined compressive strength and residual unconfined compressive strength tests.

More specifically, the proposed research includes

(a) A study of compaction behaviour of fly ash–lime–gypsum/phosphogypsum mix.

(b) A study of unconfined compressive strength of fly ash–lime–gypsum/phosphogypsum mix at different curing period.

(c)A study of residual unconfined compressive strength of fly ash–lime–gypsum/phosphogypsum mixes at different curing period and subjected to dry-wet cycles.

An extensive laboratory testing programme was devised and the comparative study was conducted to critically assess the possible application of these waste materials in roads.

The aim of the present study was also to use gravel in base course of pavement. This material is available in abundance in river bed. Therefore, wherever gravel is easily available it may be economical to utilize this material for road construction.

In the present study the materials used in the base course construction were coarse aggregate of grading No. 2 of WBM Construction. Fine aggregate used was screenings which falls in type-A grading of I.R.C specifications and binding material used was local sandy soil having its plasticity index less than 6 (for reference; from data of Kullu soil).

The following preliminary tests were conducted on the coarse aggregates according to IS: 2386 Part 1 - 1963 (3), to judge their suitability prior to their use as pavement construction material.

1. Flakiness test
2. Elongation test
3. Crushing test
4. Abrasion test
5. Specific gravity test
6. Water absorption test

The soil which was used as subgrade was Kullu soil and is categorized SP according to IS classification.

2.6 Scope of the work

A major source of generation of power in India is from Thermal Power Plants (TPPs) which typically use pulverized coal as fuel. The by-product from these plants typically contain a coarse material type known as bottom ash and fine material type known as fly ash, with both exhibiting variable physicochemical properties (Sridharan et al.1996). It is reported that Indian coal based TPPs produce around 90 million tons of ash per year which covers an area of 265 km^2 as ash pond (Das and Yudbhir 2005). Acquiring open lands for disposal of fly ash in developing countries like India is difficult, where the land to population ratio is small. Since the land requirement and the cost of land are increasing day by day, therefore it is essential to find out different ways for gainful utilization of this waste in civil engineering activities. Kamon and Nontananandh (1991) reported that successful waste utilization (combining industrial waste with lime for soil stabilization) could result in considerable saving in construction cost. Bulk utilization of fly ash is possible only by way of its use in geotechnical applications such as embankment construction, backfill materials and sub base materials etc. (Pandian 2004).

The quantity of fly ash produced worldwide is huge and keeps increasing from year to year. Kaniraj and Gayathri (2004) reported that countries like China, India, Poland, and the United States alone produce more than 270 million tons of fly ash every year.

The total installed capacity of power generation is 2, 11,766.22 MW in India up to the year 2013**.** Of this, coal based thermal power plants account for 57.42% of electricity generation (Ministry of Power). Present generation of fly ash from coal based thermal power plants is 131 MT/year and it is expected to increase to 300-400 MT/year by 2016-17 with the setting up of new thermal power plants in India.

The utilization of ash in India was 55.79 % during the year 2010-12. Fly ash is a fine-grained material of mostly silt size particles. It is non plastic and can therefore be handled easily. Further, because of its pozzolanic properties, it can be stabilized with lime/gypsum/phosphogypsum or in combination to achieve the required properties. Thus, in this scenario, it is planned to conduct a detailed study on the durability of fly ash-lime-gypsum/ phosphogypsum mix for possible use in roads.

Gravels in India are available in large quantities in many places such as stream bottoms or terraces adjacent to streams, outwash plains and adjacent to mountains. Aggregates form the major portion of the pavement structure and it is the prime material used in pavement construction. Aggregates primarily bear the stresses and have to resist wear due to abrasive action of traffic.

In the present study, gravel has been used in base course for road construction. Various tests such as combined flaky and elongation indices, density test and compressive test (using large shear box) have been conducted for judging suitability of the aggregates.

Compressive test on pavement model using gravel and sand aggregate mixture have also been conducted. It has been found that gravel can be used successfully in road construction. The use of gravel will be much more economical if it is found locally. It has been recommended that vast potentials offered by gravel should be fully exploited for the construction of base course in the flexible pavements.

CHAPTER 3

MATERIALS, SETUP AND METHODOLOGY

3.1 Materials

3.1.1 Fly Ash

Chemical composition of Ropar Thermal Power Plant Fly ash for present testing is given in Table 3.1. The fly ash was collected from Guru Gobind Singh Thermal Power Plant, Ropar (Punjab). Table 3.2 gives the physical and engineering properties of fly ash. The fly ash is classified as class F fly ash as per ASTMC 618 (ASTM 1993).

Chemical component	Percentage (%)
Silicon dioxide SiO_2	56.80
Aluminium oxide Al_2O_3	26.10
Ferric oxide Fe_2O_3	5.0
Calcium oxide CaO	3.8
Magnesium oxide MgO	2.3
Titanium oxide TiO_2	1.4
Potassium oxide K_2O	0.6
Sodium oxide Na_2O	0.4
Sulphur trioxide SO_3	1.6
LOI (1000 0C)	1.9
Moisture	0.3

Table 3.1 - Chemical composition of Ropar Thermal Power Plant Fly ash

Property	Values
Grain Size Distribution:	
Sand (0.075 to 1.000 mm) (%)	97
Silt (%) and Clay (%) (≤ 0.075 mm)	3
Specific gravity	1.96
Standard proctor compaction:	
Maximum dry density (kN/m^3)	11.41
Optimum moisture Content (%)	32
Unsoaked CBR (%)	29
Angle of internal friction (Φ)	34.5

Table 3.2 - Physical and engineering properties of Fly ash

Engineering property *i.e.* compaction test result of fly ash used for present testing are given in Table 3.3.

Property	Values
Standard proctor compaction:	
Maximum dry density (MDD) (kN/m^3)	11.41
Optimum moisture Content (OMC) (%)	32

Table 3.3 – Compaction test results of fly ash

The maximum dry unit weight and optimum water content obtained from standard proctor test for the fly ash is 11.41 kN/m^3 and 32% respectively (see figure 3.1).

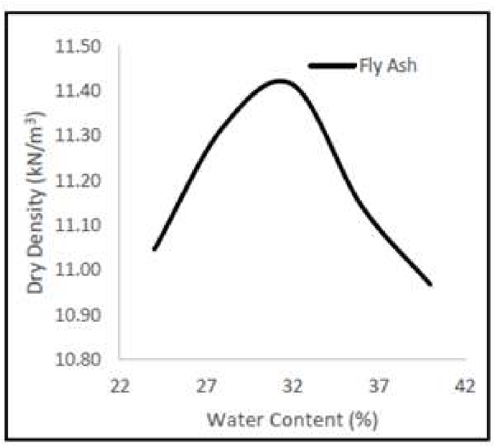


Figure 3.1 – Standard Proctor compaction test results for fly ash

3.1.2 Lime

Stabilization using lime is an established practice to improve the characteristics of fine grained soils. The first field applications in the construction of highways and airfields pavements were reported in 1950-60. With the proven success of these attempts, the technique was extended as for large scale soil treatment using lime for stabilization of subgrades as well as improvement of bearing capacity of foundations in the form of lime columns. Lime also imparts some binding action. Hydrated lime is created when quicklime chemically reacts with water. It is hydrated lime that reacts with clay particles and permanently transforms them into a strong cementitious matrix. If quicklime is used, it immediately hydrates (i.e., chemically combines with water) and releases heat. Soils are dried, because water present in the soil participates in this reaction, and because the heat generated can evaporate additional moisture. The hydrated lime produced by these initial reactions will subsequently increases the electrolytic concentration and pH of the pore water and dissolves the silicates (SiO_2) and aluminates (Al_2O_3) from the clay particles. Na^+ and other cations adsorbed to the clay mineral surfaces are exchanged with Ca^{++} ions. These reactions will slowly produce additional drying because they reduce the soil's moisture holding capacity. When adequate quantities of lime and water are added, the pH of the mixture quickly increases to above 10.5, which enables the clay particles to break down. Silica and alumina are released and react with calcium from the lime to form calciumsilicate- hydrates (CSH) and calcium-aluminate-hydrates (CAH). CSH and CAH are cementitious products similar to those formed in Portland cement. They form the matrix that contributes to the strength of lime-stabilized soil layers. The matrix formed is permanent, durable, and significantly impermeable, producing a structural layer that is both strong and flexible.

Lime varies widely in its quality when collected from different sources or collected in batches from the same source. In order to keep uniformity in quality of lime, high calcium calcite lime was used throughout the investigation. Its properties and chemical composition, as supplied by the manufacturer, are reported in table 3.4.

Composition or property	Value
Specific gravity	2.05
Normal consistency	43.50
Initial setting time (min)	165
Final setting time (h)	46.25
Fineness(%age by weight on 300 μm sieve)	2.65
Soundness [(Lechatlier's expansion (mm)]	1.8
Compressive strength(14days)(N/mm^2)	1.45
Compressive strength(28days)(N/mm^2)	2.18
Calcium hydroxide (%)	Maximum 95
Chloride (%)	Maximum 0.01
Sulphate (%)	Maximum 0.2
Aluminum, iron, insoluble matter etc (%)	Maximum 1.0
Arsenic (%)	Maximum 0.0004
Lead (%)	Maximum 0.0001

Table 3.4 - Chemical composition and physical properties of hydrated lime

3.1.3 Gypsum

Gypsum which is commonly called Plaster of Paris (POP) is a by-product of phosphoric acid production. In India, the annual production of gypsum is approximately 4.5 million t, whereas its annual world production corresponds to 280 million t. Gypsum is composed of a mixture of calcium sulphate dihydrate ($CaSO_4 \cdot 2H_2O$) and calcium sulfate pentahydrate ($CaSO_4 \cdot 5H_2O$) with impurities of silica (SiO_2), aluminium oxide (Al_2O_3), iron oxide (Fe_2O_3), and phosphorous oxide (P_2O_5). According to estimates, only 15% of the world's gypsum production is reused in the manufacture of building materials, fertilizers, soil stabilization amendments, and the manufacture of Portland cement, and the remaining amount is stored near the industrial.

3.1.4 Phosphogypsum

Phosphogypsum refers to the gypsum formed as a by-product of processing phosphate ore into fertilizer with sulfuric acid. Phosphogypsum is produced from the fabrication of phosphoric acid by reacting phosphate ore (apatite) with sulfuric acid according to the following reaction:

$Ca_5(PO_4)_3X + 5H_2SO_4 + 10H_2O \rightarrow 3H_3PO_4 + 5CaSO_4 \cdot 2H_2O + HX$………… (Eqn… 1)

where X may include OH, F, Cl, or Br

Phosphogypsum is radioactive due to the presence of naturally occurring uranium and radium in the phosphate ore. Marine-deposited phosphate typically has a higher level of radioactivity than igneous phosphate deposits, because uranium is present in seawater.

3.2 Experimental Methods for Fly ash-lime-gypsum/phosphogypsum admixture preparation

3.2.1 Proctor Compaction Test

This experiment gives a clear relationship between the dry density of the soil and the moisture content of the soil. The experimental setup consists of (i) cylindrical metal mould internal diameter- 10.15 cm and internal height-11.7 cm), (ii) detachable base plate, (iii) collar (5 cm effective height), (iv) rammer (2.5 kg). Compaction process helps in increasing the bulk density by driving out the air from the voids. The theory used in the experiment is that for any compactive effort, the dry density depends upon the moisture content in the soil. The maximum dry density (MDD) is achieved when the soil is compacted at relatively high moisture content and almost all the air is driven out, this moisture content is called optimum moisture content (OMC). After plotting the data from the experiment with water content as the abscissa and dry density as the ordinate, we can obtain the OMC and MDD. The equations used in this experiment are as follows:

$$Wet\ Density = \frac{Weight\ of\ wet\ soil\ in\ mould(g)}{Volume\ of\ mould(cc)}$$

$$Moisture\ Content\ \% = \frac{Weight\ of\ water(g)}{Weight\ of\ dry\ soil(g)} * 100$$

$$Dry\ Density\ \rho_d(g/cc) = \frac{Wet\ Density}{1 + \frac{moisture\ content}{100}}$$

Factors Affecting Compaction

The factors that influence the achieved degree of compaction in the laboratory are:

- Water content
- Compactive effort

Effect of Increasing Water Content

As water is added to a soil at low moisture contents, it becomes easier for the particles to move past one another during the application of compacting force. The particles come closer, the voids are reduced and this causes the dry density to increase. As the water content increases, the soil particles develop larger water films around them. This increase in dry density continues till a stage is reached where water starts occupying the space that could have been occupied by the soil grains. Thus the water at this stage hinders the closer packing of grains and reduces the dry unit weight. The **maximum dry density (MDD)** occurs at an **optimum water content (OMC),** and their values can be obtained from the plot.

Effect of Increasing Compactive Effort

The effect of increasing compactive effort is shown. Different curves are obtained for different compactive efforts. A greater compactive effort reduces the optimum moisture content and increases the maximum dry density.

An increase in compactive effort produces a very large increase in dry density for soil when it is compacted at water contents drier than the optimum moisture content. It should be noted that for moisture contents greater than the optimum, the use of heavier compaction effort will have only a small effect on increasing dry unit weights. It can be seen that the compaction curve is not a unique soil characteristic. It depends on the compaction effort. For this reason, it is important to specify the compaction procedure (light or heavy) when giving values of MDD and OMC.

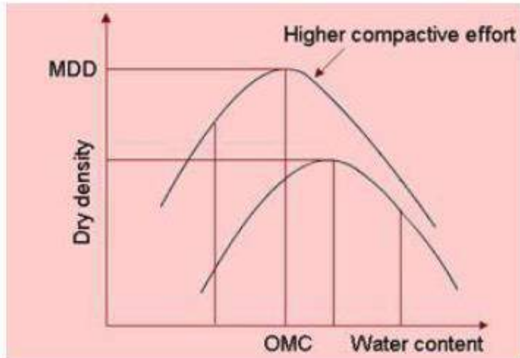


Figure 3.2 - Dry Density v/s Water Content with increasing Compactive effort

3.2.2 Unconfined Compression Test

This test is a specific case of tri-axial test where the horizontal forces acting are zero. There is no confining pressure in this test and the soil sample tested is subjected to vertical loading only. The specimen used is cylindrical and is loaded till it fails due to shear. This experiment is used to determine the unconfined compressive strength of the soil sample which in turn is used to calculate the unconsolidated, undrained shear strength of unconfined soil. The unconfined compressive strength (q_u) is the compressive stress at which the unconfined cylindrical soil sample fails under simple compressive test. The experimental setup constitutes of the compression device and dial gauges for load and deformation.

$$q_u = \text{Load/corrected Area (A)}$$

$$q_u \text{ - Compressive Stress}$$

$$A = \text{cross-sectional area } (A_o) / (1-\varepsilon)$$

One of the popular methods of evaluating the effectiveness of stabilization is the unconfined compressive strength. As a general rule, for a given type of stabilization, the higher the compressive strength, the better is the quality of the stabilized material. Unconfined compression tests were carried out on cylindrical specimens 38 mm diameter and 76 mm long. The fly ash-lime-gypsum mixtures were compacted at optimum moisture content and maximum dry density in standard molds. The mixture was compacted in three layers and each layer was compacted using 2.6 kg rammer under a free fall of 310 mm. Specimens of 38 mm diameter and 76 mm long were extracted from the mold and were cured by burlap method before being tested in compression. Curing times adopted were 7, 14 and 28 days and at least four specimens were tested for each case. The unconfined compressive strength was determined at a loading rate of 1.25 mm/min.

Figure 3.3 – Setup for Unconfined Compressive Strength (UCS) measurement

3.2.3 Durability test

Durability which can be defined as the ability of a material to retain stability and integrity over years of exposure to the destructive forces of weathering is one of the most important properties (Dempesy and Thompson 1968).The durability tests on fly ash-lime-gypsum mixture were conducted as per IS: 4332-1968 and reaffirmed in 1995. For these tests, specimens were prepared at the maximum dry density and optimum moisture content and then moist cured for a specific number of days. Subsequently, specimens were immersed in water for 5 hours followed by air drying for 42 hours at room temperature, which completes single cycle of wetting and drying. After each cycle, the specimens were brushed with a steel wire brush and the loss in the material is recorded as mass loss (brush loss) in percentage. Replacing brushing by measuring the compressive strength of specimens after they are subjected to the 12 cycles of wetting-drying could provide a more consistent and convenient measures of the deterioration of the mix. Shihata and Baghdadi (2001) also suggested using the residual compressive strength of durability specimens without brushing as an indicator of resistance potential since it gives more consistent results. Thus the samples prepared without brushing were tested for unconfined compressive strength. This compressive strength was levelled as unbrushed residual strength (URS). Aim of conducting compressive strength test without brushing is to explore the possibility of using residual (compressive) strength of soil-flyash-lime mix as a viable indicator of durability resistance.

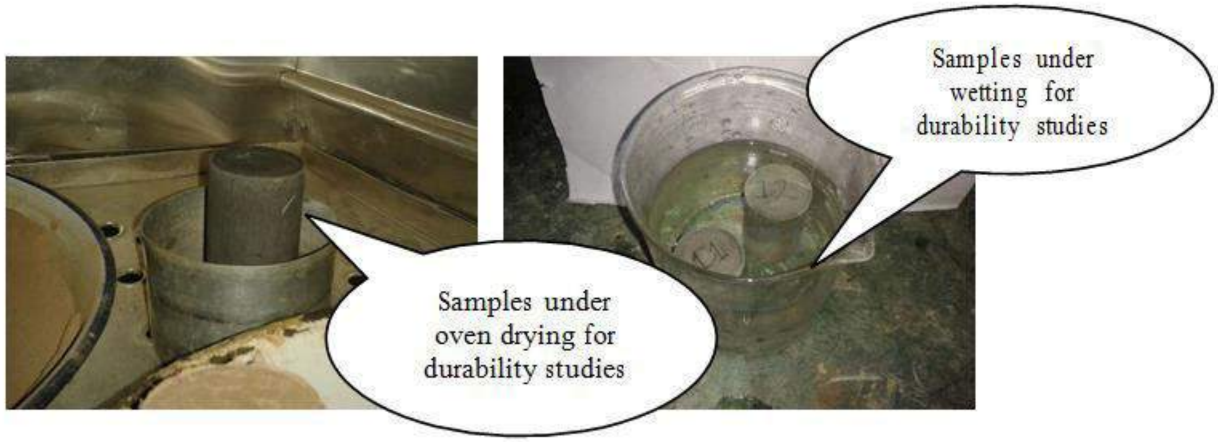


Figure 3.4 – Setup for drying-wetting cycles

3.3 Experimental work of fly ash – lime – gypsum / phosphogypsum admixture preparation

3.3.1 Preparation of reference mixes

Fly ash-lime-gypsum reference mix is prepared using MDD & OMC results (see section 4.2) obtained by "Standard Proctor compaction test" using varying gypsum content (0.5 to 2.0%) on Fly ash + 8% lime mixture composition. The reference mixture reported as MDD and OMC values of 1.208 g/cc and 28% respectively. The idea behind this lies in the fact that MDD & OMC results are best representatives of actual field conditions where we attempt to obtain a Relative Compaction (RC) of 90-95%.

Fly ash-Lime-Phosphogypsum reference mix is prepared using compaction test results (see section 4.2) obtained by "Standard Proctor compaction test" using varying Phosphogypsum content (0.5 to 4.0%) on Fly ash + 8% lime mixture composition. The reference mixture reported as MDD and OMC values of 1.23 g/cc and 24% respectively. The idea behind this lies in the fact that MDD & OMC results are best representatives of actual field conditions where we attempt to obtain a Relative Compaction (RC) of 90-95%.

3.3.2 UCS Sample preparation (12 samples each for both binders)

For UCS measurement experimentation 12 Nos. samples prepared (Numbered as S1 – S12) by using compaction mold. These 12 samples were divided into three groups of 4 samples each; for testing after 7, 14 and 28 days curing with 2 samples for UCS after Curing and 2 samples for UCS after Durability Study. Sample Dimension were reported to be 38 mm Diameter & 76 mm Length with 104.0 g dry mass and 133.3 g wet mass (fly ash-lime-gypsum) and 106.0 g dry mass and 132.0 g wet mass (fly ash-lime-phosphogypsum). Samples are kept for 24 hours oven drying after extraction from compaction mould (figure 3.5).

Figure 3.5 – Sample preparation using compaction mould and sample extractor

3.3.3 Curing

Burlap curing method was used for 7, 14 and 28 days curing of prepared samples. Burlap method involves covering the samples underneath burlap wetted periodically having kept at 27°C temperature.

Figure 3.6 – Burlap curing method for 7, 14 and 28 days sample curing

3.3.4 Durability studies

Two number of samples after the completion of their respective curing periods are studies for durability as per IS 4332 part 4, by using 12 cycles of repetitive 42 hours drying followed by 5 hours of over drying (See figure 3.7).

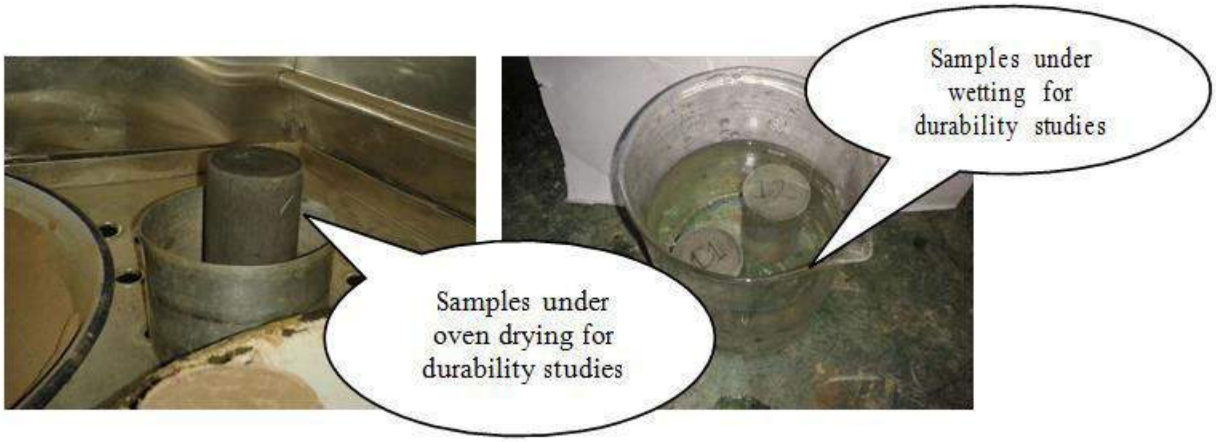


Figure 3.7 – Setup for drying-wetting cycles

3.3.5 UCS measurement

UCS measurement tests were carried out as per guidelines of IS 4332 Part 5 after completion of 7, 14 and 28 days curing for 6 numbers of samples (2 samples at a particular curing period) and for other 6 number of samples (2 samples at a particular curing period) after curing as well as durability studies. Specifications written in table 3.5 were employed for testing.

Deflection Gauge Specifications	
Strain rate	1.25 mm/min
Dial gauge LC	0.00001 m
Proving Ring Specifications	
2 kN LC	3.0053 N
10 kN LC	0.0127 kN
25 kN LC	0.0365 kN

Table 3.5 – UCS measurement specifications

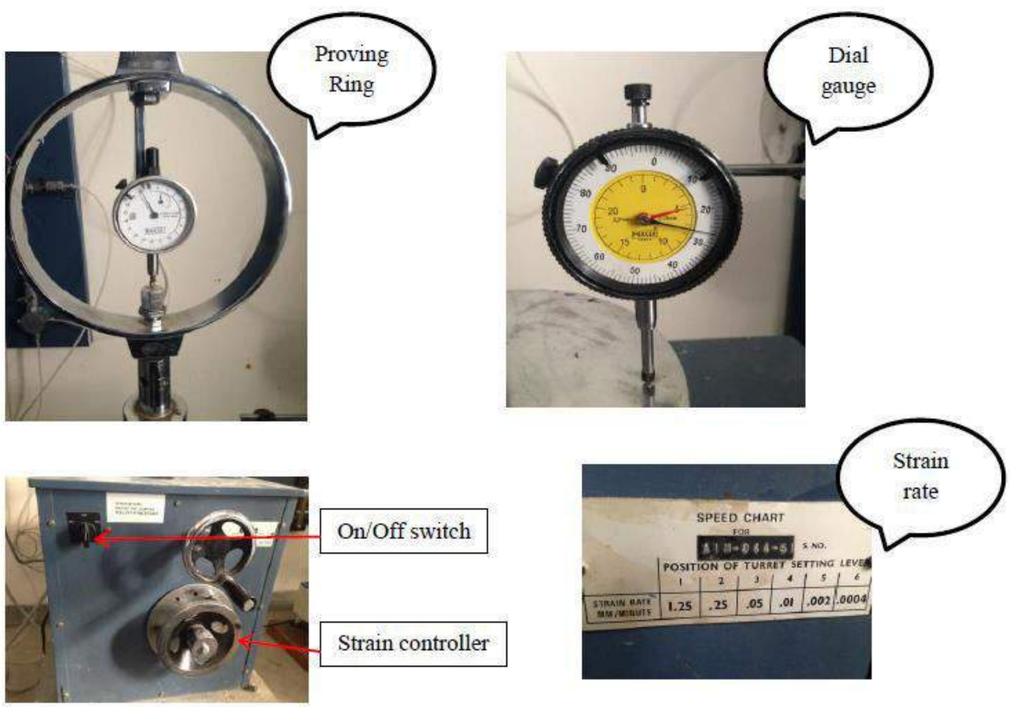


Figure 3.8 – Components of UCS machine

3.4 Experimental tests on coarse aggregates (used for pavement models)

3.4.1 Flakiness Index (IS 2386 Part 1 1963)

The flakiness index of aggregates is determined by the percentage of flaky particles contained in it. For base course, and construction of asphalt and cement concrete types the presence of flaky particles is considered undesirable as it may cause inherent weakness with possibilities of braking down under heavy loads. Therefore, in pavement construction the flaky particles are to avoid particularly is surface course. The test apparatus consists of a standard thickness gauge (6.30 – 63.0 mm) is used for this test. The flakiness index test (FI) is not applicable passing 6.30 mm or retaining 63.0 mm test sieve. The FI is the total weight of the material passing the various thickness gauges expressed as a percentage of the total weight of the sample gauged. The flakiness index value of the aggregate should be below 35 % that recommended for the road construction.

FI = (Weight of passing flaky particles/Total weight of sample gauged)*100

3.4.2 Elongation Index (IS 2386 Part 1 1963)

In order to calculate the elongation index of the entire sample of aggregates, first the weight of each fraction of aggregate passing and retained on the specified set of sieves is noted (Y1, Y2, Y3, Y4…..etc). Each piece of these are tried to be passed through specified length of the gauge length with its longest side and those elongated pieces which do not pass the gauge are separated and weighed (y1, y2, y3, y4…etc). Then the elongated index is the total weight of the material retained on the various length gauges, expressed as a percentage of the total weight of the sample gauged.

EI = (y1+y2+y3+…)/(Y1+Y2+Y3+…)

The results of flakiness index and elongation index and their recommended values are given in table 3.6.

Property	Test Value for Gravel	IRC requirement (max)	
		Subbase course	Base course
Flakiness Index (%)	25	30	30
Elongation Index (%)	34.06	40	40
Combined Flakiness and Elongation indices (%)	29.53	30	30

Table 3.6 – Values of Flakiness index and Elongation index

The values of flakiness index and elongation are well within limits as specified by IRC. The value of combined flakiness and elongation indices is nearly 30 %.

3.4.3 Aggregate Crushing Value (IS 2386 Part 4 1963)

The aggregates passing through 12.5 mm and retained on 10mm IS Sieve are oven-dried at a temperature of 100 to 110^{o}C for 3 to 4 hrs. The cylinder of the apparatus is filled in 3 layers, each layer tamped with 25 strokes of a tamping rod. The weight of aggregates is measured (Weight 'A'). The surface of the aggregates is then leveled and the plunger inserted. The apparatus is then placed in the compression testing machine and loaded at a uniform rate so as to achieve 40t load in 10 minutes. After this, the load is released. The sample is then sieved through a 2.36mm IS Sieve and the fraction passing through the sieve is weighed (Weight 'B').

Aggregate Crushing Value = (B/A)*100

3.4.4 Aggregate Abrasion Value (IS 2386 Part 4 1963)

The test sample (oven dried at 105 – 110^{o} C and the abrasive charge (12 no. iron balls – 48 mm dia., 390 g each) should be placed in the Los Angles abrasion testing machine and the machine rotated at a speed of 20 to 33 revolutions/minute for 1000 revolutions. At the completion of the test, the material should be discharged and sieved through 1.70mm IS Sieve. The material coarser than 1.70mm IS Sieve should be washed, dried in an oven at a temperature of 100 to 110^{o}C to a constant weight and weighed (Weight 'B').

Aggregate Abrasion Value = ((A-B)/B)*100

The results of aggregate crushing value and impact value are given in table 3.7.

Property	**Test Value for Gravel**	**IRC requirement (max)**	
		Subbase course	**Base course**
Aggregate Crushing value (%)	20.34	40	40
Aggregate Impact value (%)	18.84	50	40

Table 3.7 – Values of Aggregate Crushing value and Impact value

The values of crushing value and impact value are within specified limits of IRC.

3.4.5 Specific Gravity and Water Absorption (IS 2386 Part 4 1963)

The results of specific gravity and water absorption of coarse aggregates are given in table 3.8.

Property	**Test Value for Gravel**	**IRC requirement (max)**	
		Subbase course	**Base course**
Specific Gravity	2.7	2.67 - 2.9	2.5 - 3.0
Water absorption (%)	0.8	1	1

Table 3.8 – Values of Specific Gravity and Water absorption

The values of specific gravity and water absorption are within limits of IRC.

3.5 Preparation of Shear Box from Perspex Sheet

3.5.1 Shear Box 1 (without fly ash-lime-gypsum/phosphogypsum binder admixture)

A rectangular box of 0.4 m x 0.3 m x 0.35 m was constructed from perspex sheet. A wooden frame and base was provided for the manufacture of shear box. The box is filled with the material, proper compaction was to be done to achieve maximum density and bond developments between different layers. The perspex sheet was purchased from Bhagra Steel Sales Pvt. Ltd. in Tara Devi Shimla.

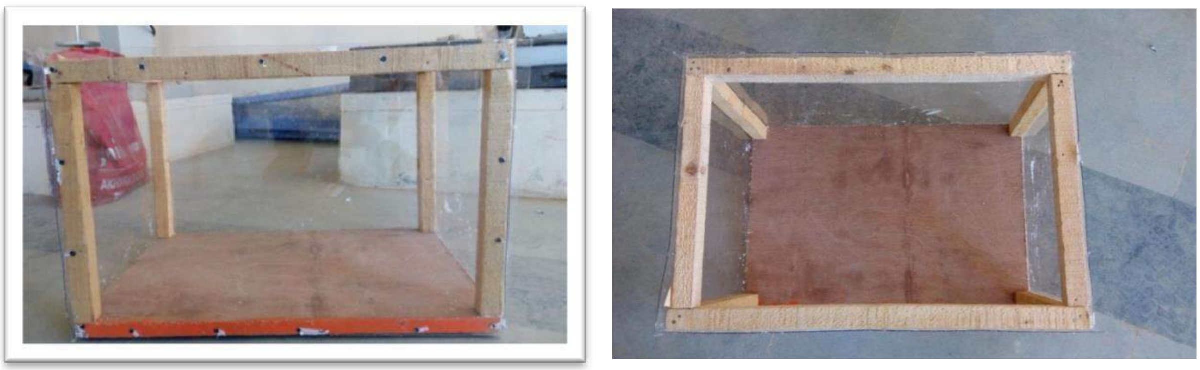


Figure 3.9 – Shear Box 1

3.5.2 Shear Box 2 (with fly ash-lime-gypsum binder admixture)

A rectangular box of 0.29 m x 0.3 m x 0.35 m was constructed from perspex sheet. A steel frame and wooden base was provided for the manufacture of shear box. The box is filled with the material, proper compaction was to be done to achieve maximum density and bond developments between different layers. The perspex sheet was obtained from Department of Civil Engineering, JUIT Waknaghat.

Figure 3.10 – Shear Box 2

3.5.3 Shear Box 3 (with fly ash-lime-phosphogypsum binder admixture)

A rectangular box of 0.29 m x 0.3 m x 0.35 m was constructed from perspex sheet. A steel frame and wooden base was provided for the manufacture of shear box. The box is filled with the material, proper compaction was to be done to achieve maximum density and bond developments between different layers. The perspex sheet was obtained from Department of Civil Engineering, JUIT Waknaghat.

Figure 3.11 – Shear Box 3

3.6 Introduction to Flexible Pavement

A highway pavement is a structure consisting of superimposed layers of processed materials above the natural soil sub-grade, whose primary function is to distribute the applied vehicle loads to the sub-grade.

An ideal pavement should meet the following characteristics:

- Sufficient thickness to distribute the wheel load stresses to a safe value on the sub-grade soil
- Structurally strong to withstand all types of stresses imposed upon it
- Adequate coefficient of friction to prevent skidding of vehicles
- Smooth surface to provide comfort to road users even at high speed
- Produce least noise from moving vehicles
- Dust proof surface so that traffic safety is not impaired by reducing visibility
- Impervious surface, so that sub-grade soil is well protected
- Long design life with low maintenance cost

We are going to design a flexible pavement according to the recommendations of IRC 37 – 2001. In flexible pavements, wheel loads are transferred by grain-to-grain contact of the aggregate through the granular structure. The flexible pavement, having less flexural strength, acts like a flexible sheet (e.g. bituminous road).

Flexible pavements will transmit wheel load stresses to the lower layers by grain-to-grain transfer through the points of contact in the granular structure. The wheel load acting on the pavement will be distributed to a wider area, and the stress decreases with the depth. Taking advantage of this stress distribution characteristic, flexible pavement normally has many layers.

Hence, the design of flexible pavement uses the concept of layered system. Based on this, flexible pavement may be constructed in a number of layers and the top layer has to be of best quality to sustain maximum compressive stress, in addition to wear and tear. The lower layers will experience lesser magnitude of stress and low quality material can be used.

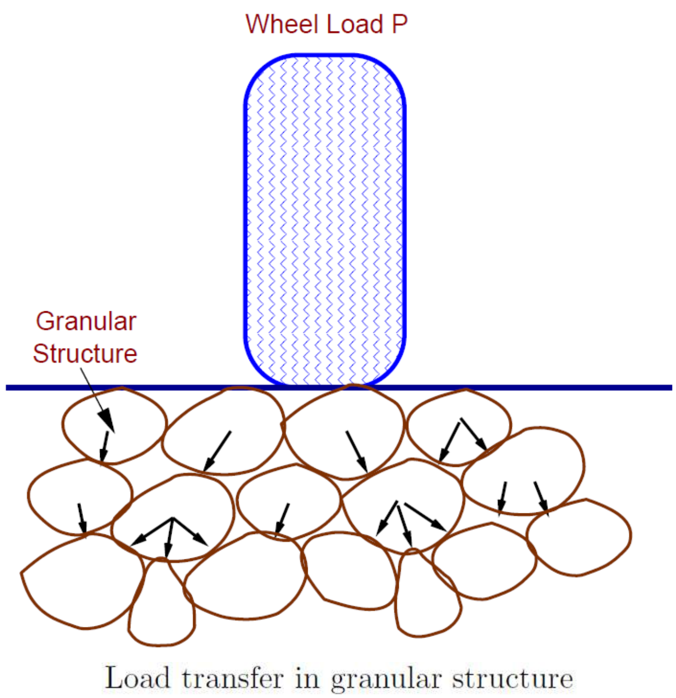


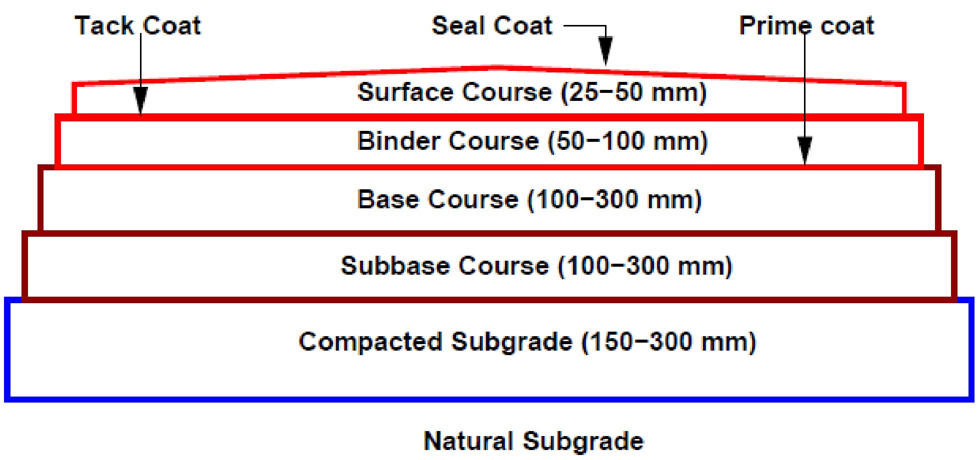
Figure 3.12 – Grain to grain load transfer/Layer to layer load transfer

Flexible pavements are constructed using bituminous materials. These can be either in the form of surface treatments (such as bituminous surface treatments generally found on low volume roads) or, asphalt concrete surface courses (generally used on high volume roads such as national highways).

Flexible pavement layers reflect the deformation of the lower layers on to the surface layer (e.g., if there is any undulation in sub-grade then it will be transferred to the surface layer).

In the case of flexible pavement, the design is based on overall performance of flexible pavement, and the stresses produced should be kept well below the allowable stresses of each pavement layer.

Typical Layers of Flexible Pavement



Typical cross section of a flexible pavement

Figure 3.13 – Layers of Flexible Pavement

3.7 Construction of Pavement Models (PMs)

3.7.1 Pavement Model (PM) 1 – Without fly ash – lime - gypsum / phosphogypsum binder admixture

Layer 1 – Compacted Subgrade

The top soil or sub-grade is a layer of natural soil prepared to receive the stresses from the layers above. It is essential that at no time soil sub-grade is overstressed. It should be compacted to the desirable density, near the optimum moisture content.

Soil taken – Kullu soil

The soil which was used as subgrade was Kullu Soil and is classified as SP (poorly graded sand) according to Unified Soil Classification System (USCS).

SP soil:

- Coarse grained soils more than 50% retained on or above No.200 (0.075 mm) sieve.

- Sand ≥ 50% of coarse fraction passes No.4 sieve.

- Clean sand with less than 12% fines.

The following table shows the properties of subgrade soil.

Property	**Result**
Soil Type	SP soil
OMC (%)	20
Dry Density (kg/m^3)	1950

Table 3.9 – Properties of Kullu soil

Figure 3.14 – Layer 1 – Compacted Subgrade course

Layer 2 – Sub-Base Course (optional)

- The sub-base course is the layer of material beneath the base course and the primary functions are to provide structural support, improve drainage, and reduce the intrusion of fines from the sub-grade in the pavement structure.
- If the base course is open graded, then the sub-base course with more fines can serve as a filler between sub-grade and the base course.
- A sub-base course is not always needed or used. For example, a pavement constructed over a high quality, stiff sub-grade may not need the additional features offered by a sub-base course. In such situations, sub-base course may not be provided.

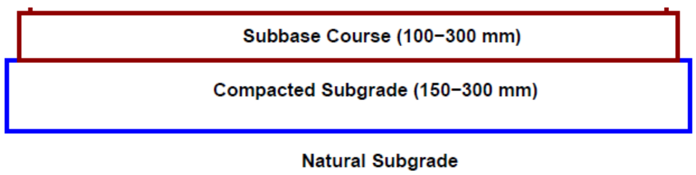


Figure 3.15 – Layer 2 – Sub-base course

Layer 3 – Base Course

- The base course is the layer of material immediately beneath the surface of binder course and it provides additional load distribution and contributes to the sub-surface drainage.
- It may be composed of crushed stone, crushed slag, and other untreated or stabilized materials.

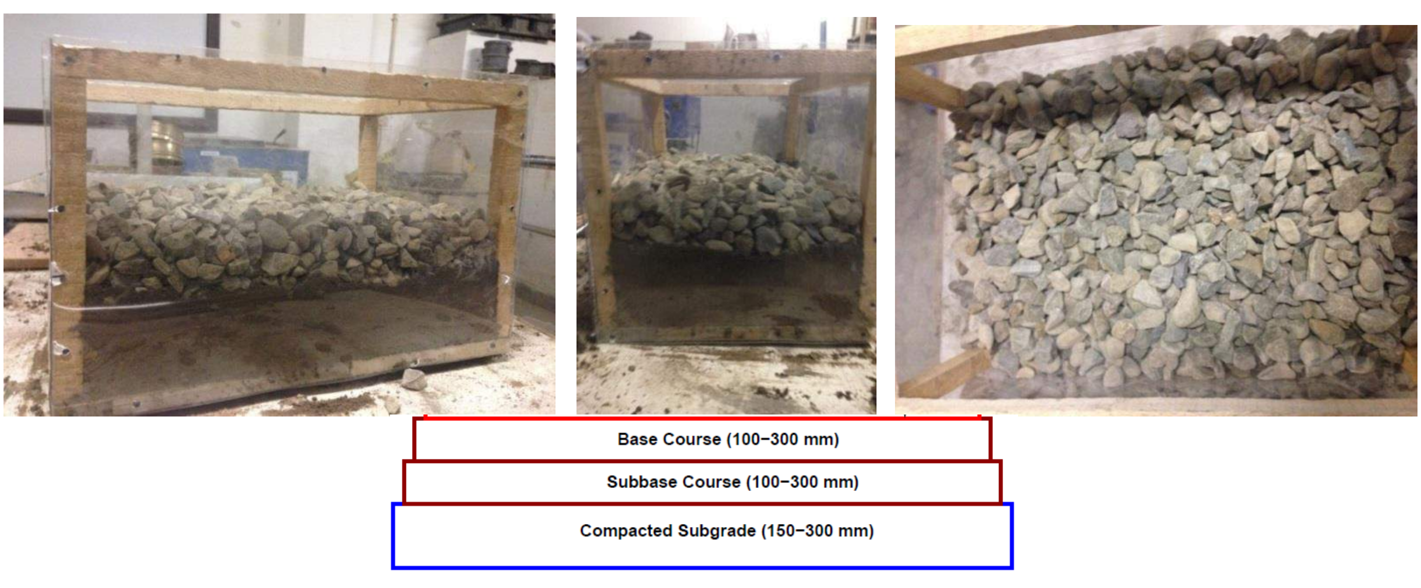


Figure 3.16 – Layer 3 – Base course

Layer 4 – Binder course

- This layer provides the bulk of the asphalt concrete structure. It's chief purpose is to distribute load to the base course.
- The binder course generally consists of fine aggregates having less asphalt and doesn't require quality as high as the surface course, so replacing a part of the surface course by the binder course results in more economical design.
- This layer is not to be confused with admixture binder layer that we are going to provide in the next two pavement models.

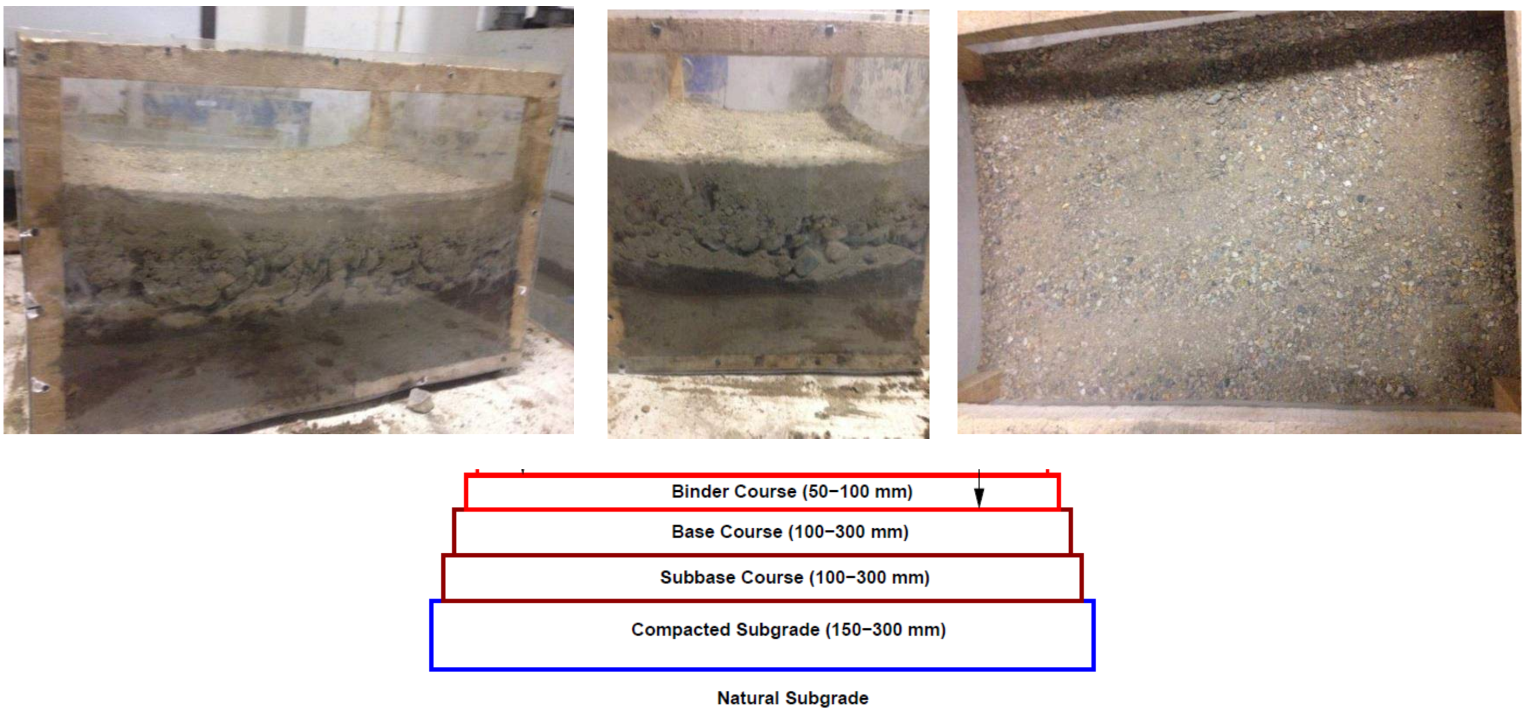


Figure 3.17 - Layer 4 – Binder course

Tack Coat on Layer 4 - Binder Course

- Tack coat is a very light application of asphalt, usually asphalt emulsion or bitumen diluted with water.
- It provides proper bonding between binder course and surface course and must be thin, uniformly cover the entire surface, and set very fast.

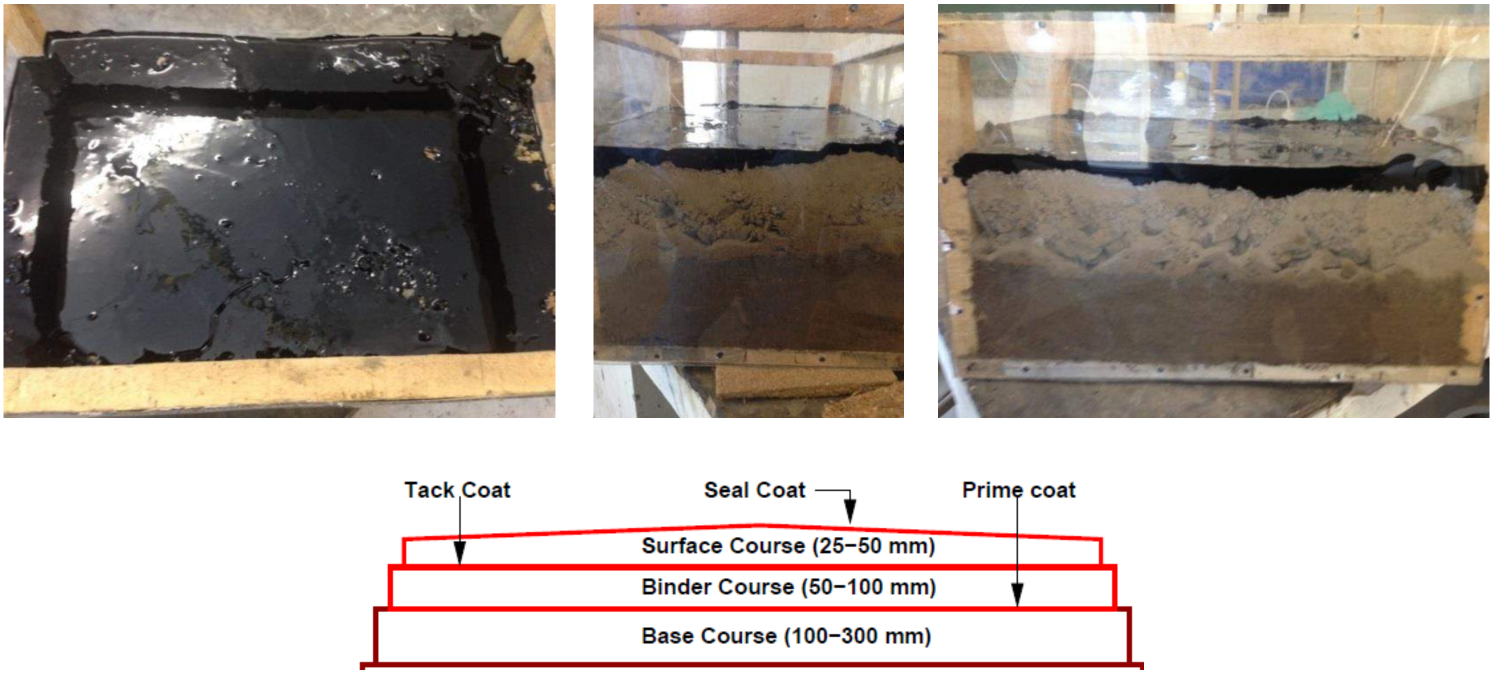


Figure 3.18 – Tack coat on Layer 4 – Base course

Layer 5 – Surface course

- Surface course is the layer directly in contact with traffic loads and generally contains superior quality materials.
- They are usually constructed with coarse aggregates mixed with hot bitumen. The functions and requirements of this layer are:
  - It provides characteristics such as friction, smoothness, drainage, etc. Also it will prevent the entrance of excessive quantities of surface water into the underlying base, sub-base and sub-grade.
  - It must be tough to resist the distortion under traffic and provide a smooth and skid- resistant riding surface.
  - It must be water proof to protect the entire base and sub-grade from the weakening effect of water.

Figure 3.19 – Mixing of aggregates with hot bitumen

Figure 3.20 – Laying of first layer of surface course consisting of bigger aggregates mixed with hot bitumen

Figure 3.21 – Laying of second layer of surface course consisting of smaller aggregates mixed with hot bitumen

Figure 3.22 – Compaction of surface course

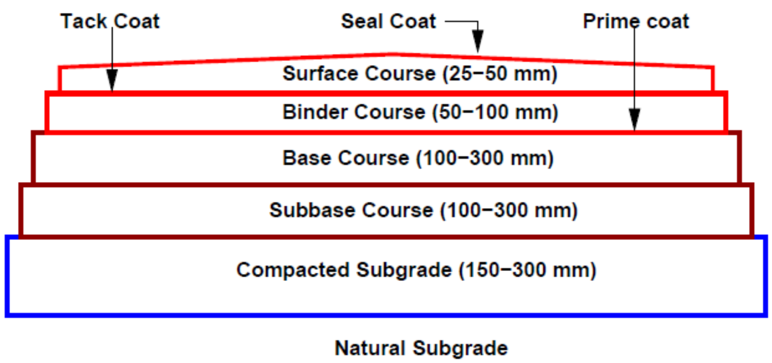


Figure 3.23 – Layer 5 – Surface course

Seal Coat on Layer 5 – Surface course

Seal coat is a thin surface treatment used to water-proof the surface and to provide skid resistance.

Figure 3.24 – Seal coat on Layer 5 - Surface course

3.7.2 Pavement Model (PM) 2 – With fly ash – lime - gypsum binder admixture

The procedure of development of model is same as discussed in section 3.7.1.

In this case, the following points are to be kept in mind.

- The binder admixture of fly ash-lime-gypsum is mixed in compacted subgrade layer.
- A layer of binder admixture is placed between compacted subgrade layer and base course.
- Use of tracers (red color) between subgrade layer and binder admixture layer; and between binder admixture layer and base course has been done to show settlement or deflection of these layers under the application of load.

  The steps are shown with photographs as follows:

Step 1: Layer 1 – Compacted subgrade layer – Kullu soil

Figure 3.25 – Soil layer

Step 2: Tracer between soil layer and binder admixture layer

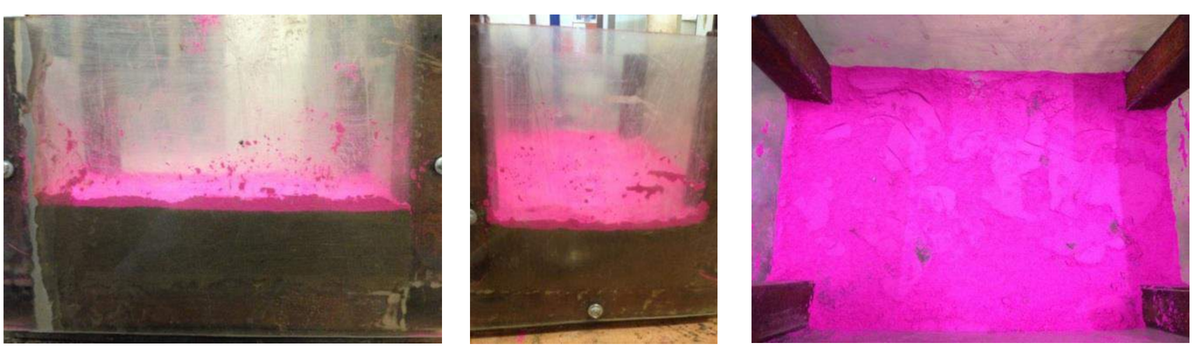


Figure 3.26 – Tracer layer

Step 3: Fly ash – lime - gypsum binder admixture layer

Figure 3.27 – Fly ash-lime-gypsum layer

Step 4: Tracer layer between binder admixture layer and base course

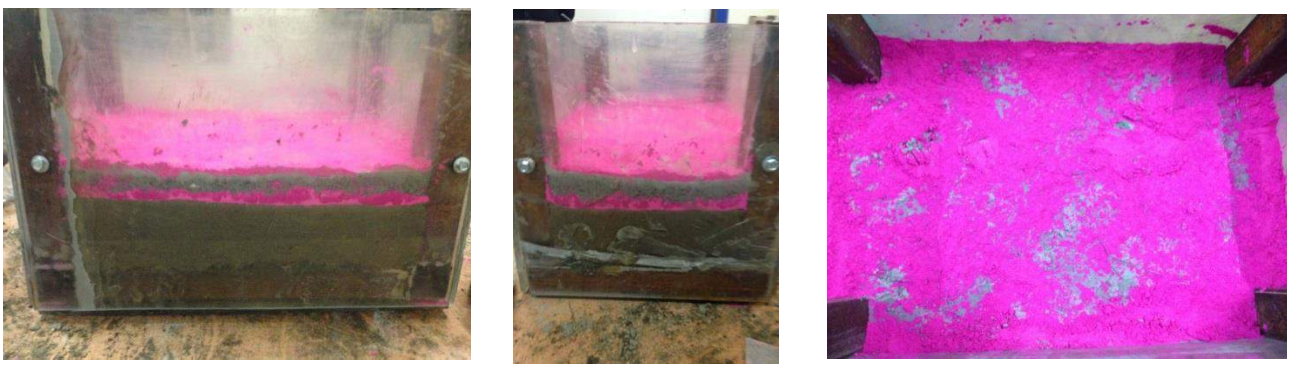


Figure 3.28 – Tracer layer 2

Step 5: Layer 2 – Base course

Figure 3.29 – Base course

Step 6: Layer 3 – Binder course

Figure 3.30 – Binder layer

Step 7: Tack coat on binder layer

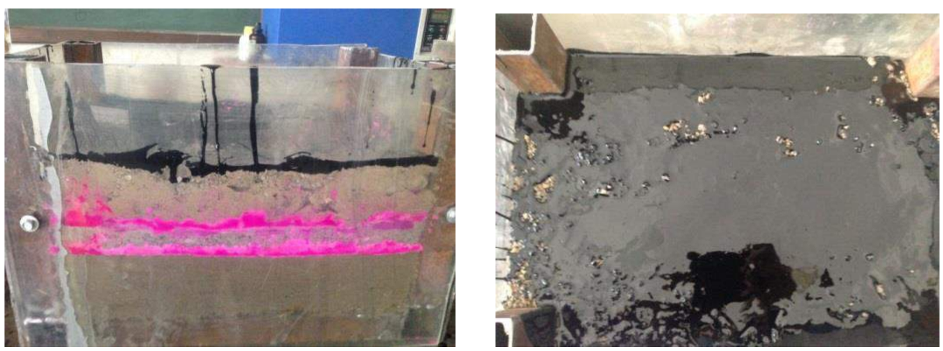


Figure 3.31 – Tack coat on binder layer

Step 8: Layer 4 – Surface course and Tack coat on surface course

Figure 3.32 – Surface course and Tack coat on surface course

3.7.3 Pavement Model (PM) 3 – With fly ash – lime - phosphogypsum binder admixture

The steps of pavement model development are same as discussed in section 3.7.1 and 3.7.2. These are shown are as follows:

Step 1: Layer 1 – Soil subgrade and tracer on it

Figure 3.33 – Soil and tracer

Step 2: Fly ash – lime - phosphogypsum binder admixture and tracer on it

Figure 3.34 – Binder admixture and tracer on it

Step 3: Layer 2 – Base course

Figure 3.35 – Base course

Step 4: Layer 3 – Binder course and tack coat on it

Figure 3.36 – Binder course and tack coat on it

Step 5: Layer 4 – Surface course and seal coat on it

Figure 3.37 – Surface course and its compaction

Figure 3.38 – Seal coat on surface course

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 General Description

As per the described methodologies the experiments were carried out to study variation of UCS and RUCS over curing time. The results obtained in different parts of the work have been discussed below.

4.2 Compaction test

4.2.1 Compaction test for fly-ash-lime-gypsum binder admixture

Compaction characteristics of Fly Ash + 8% lime with varying Gypsum content was studied. Figure 4.1 shows compaction behavior of Fly ash + 8% Lime and varying Gypsum contents. The Maximum dry density (MDD) and OMC values of 1.208g/cc and 28% respectively were obtained with Fly ash + 8% Lime +1.0% Gypsum combination. This mixture is reported as reference mix for UCS and durability studies.

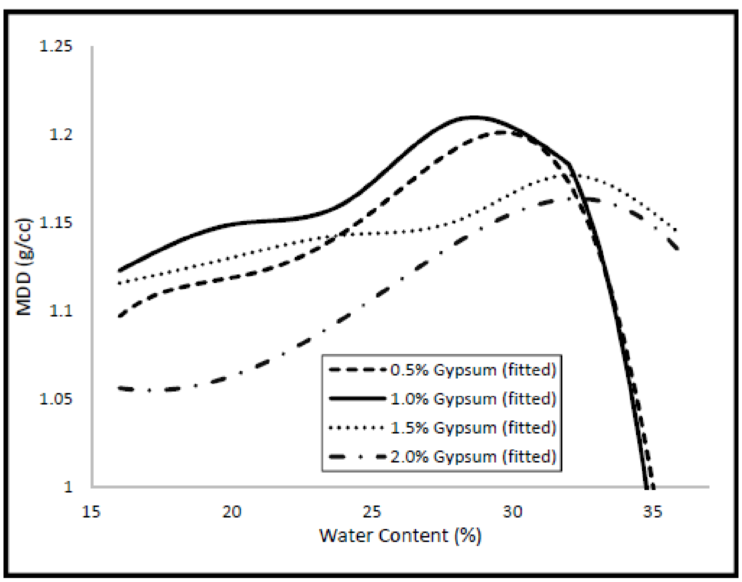


Figure 4.1 - Measurement of OMC and MDD of Fly ash + 8% Lime and varying Gypsum Content for Reference mix Preparation

4.2.2 Compaction test for Fly ash -lime-phosphogypsum binder admixture

Compaction characteristics of Fly Ash varying lime content were studied to decide appropriate composition of fly ash and lime mix. Figure 4.2 (a) shows compaction behavior of Fly ash and varying lime contents. The maximum dry density (MDD) and optimum moisture content (OMC) values of 11.79 kN/m^3 and 28% respectively were reported with fly ash + 8% lime mixture. This mixture is utilized further to prepare a reference mix of fly ash + 8% lime and varying phosphogypsum contents for unconfined compressive strength measurement and durability studies.

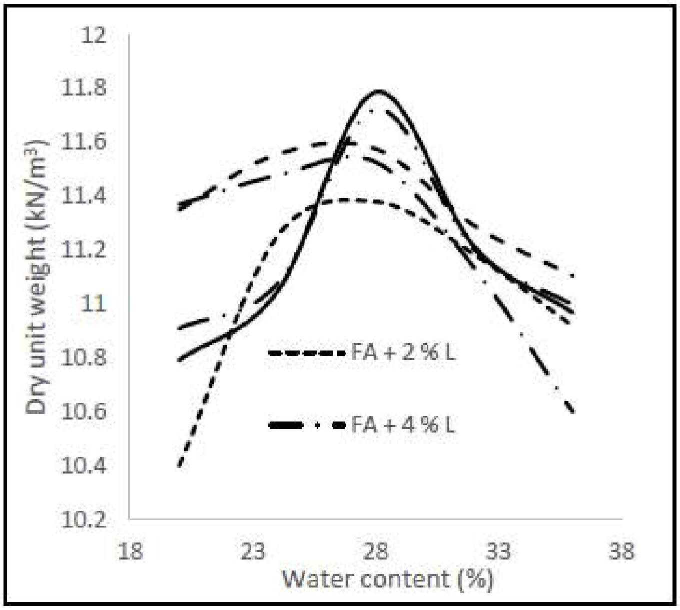


Figure 4.2 (a) – Compaction test plots for fly ash and varying lime (L) content

After this, compaction characteristics of Fly ash + 8% lime with varying phosphoypsum content was studied. Figure 4.2 (b) shows compaction behavior of Fly ash + 8% Lime and varying phosphoypsum contents. The maximum dry density (MDD) and optimum moisture content (OMC) values of 12.08 kN/m^3 and 24% respectively were obtained with Fly ash + 8% Lime +2.0% Phosphogypsum (PG) mixture. This mixture is reported as reference mix for unconfined compressive strength measurement and durability studies.

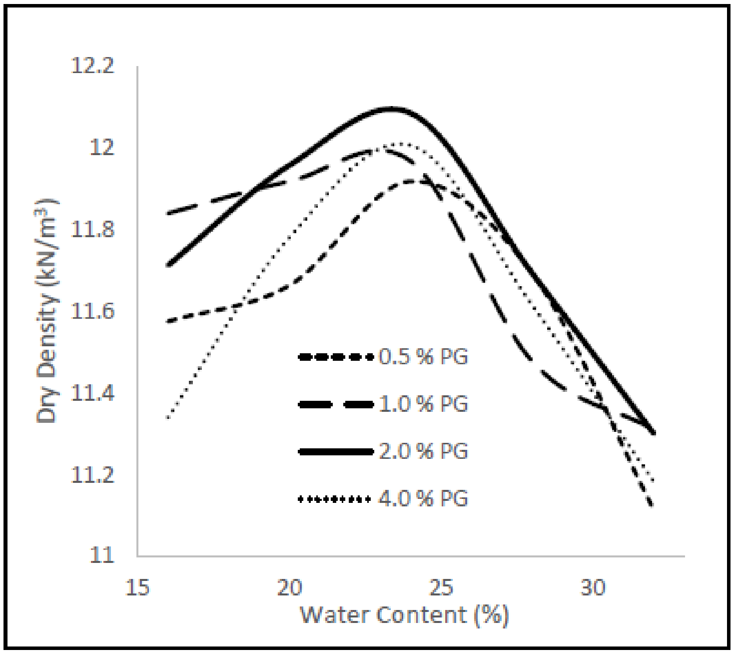


Figure 4.2 (b) – Compaction test plots for fly ash + 8% lime and varying phosphogypsum content

The compaction test results clearly indicate the increase in maximum dry density (MDD) of fly ash when mixed with lime and further increases when phosphogypsum is also added. A decreasing trend in optimum moisture content (OMC) has been observed. The observations have been tabulated below (see table 4.1).

Mix composition	**MDD (kN/m^3)**	**OMC (%)**
Fly ash only	11.41	32
Fly ash + 8% Lime	11.79	28
Fly ash + 8% Lime + 2% Phosphogypsum	12.08	24

Table 4.1 – Compaction test result trends for different mix compositions

4.3 Unconfined Compressive Strength (UCS)

4.3.1 UCS for fly ash-lime-gypsum binder admixture

Figure 4.3 shows the UCS results of 7, 14 and 28 days cured samples by burlap method and the trend shows that the reference mix is gaining strength with curing time period which is a clear evidence of increase in extent of pozzolanic reaction of lime over time forming cementations bonds with fly ash. Also, the decrease in percentage failure strain with curing period is the evidence of hardness and brittleness of the sample.

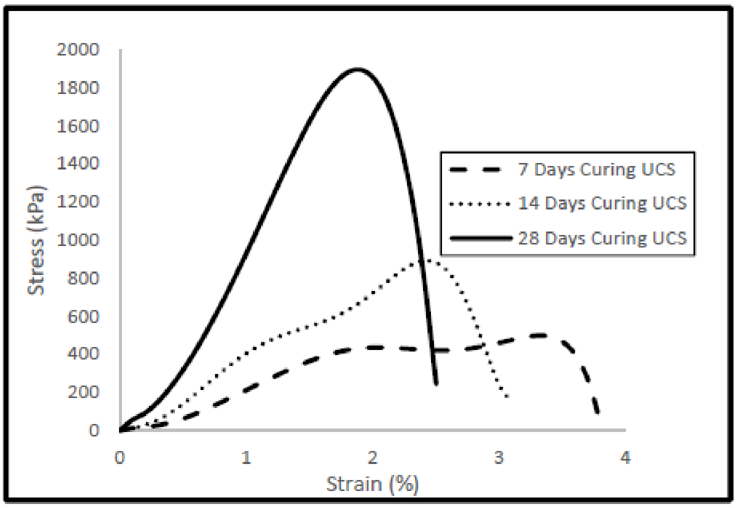


Figure 4.3 – UCS plots of 7, 14 and 28 days cured samples for Gypsum (G)

4.3.2 UCS for fly ash-lime-phosphogypsum binder admixture

Figure 4.4 shows the UCS results of 7, 14 and 28 days cured samples by burlap method and the trend shows that the reference mix is gaining strength with curing time period which is a clear evidence of increase in extent of pozzolanic reaction of lime over time forming cementations bonds with fly ash. Also, the increase in percentage failure strain with curing period is the evidence shift from brittle to ductile behavior of the sample. This may be due to porous formation with progression in curing period as evident from increase in water absorption with curing period (see figure 4.17 for PG).

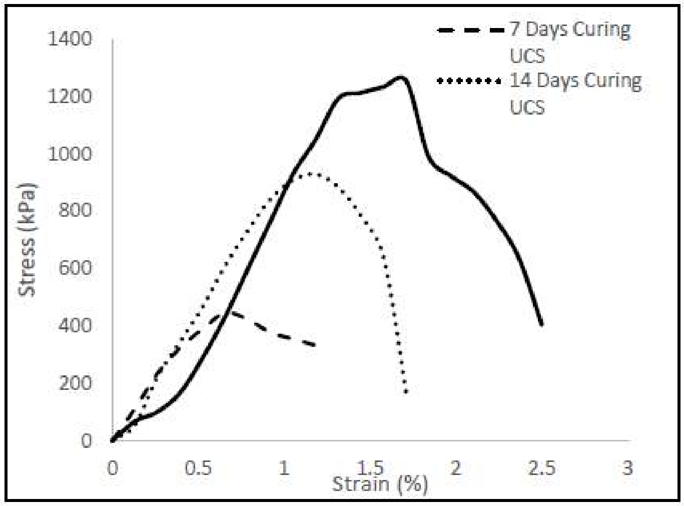


Figure 4.4 – UCS plots of 7, 14 and 28 days cured samples for Phosphogypsum (PG)

4.4 Durability Studies

4.4.1 Durability studies for fly ash-lime-gypsum binder admixture

As stated earlier in Chapter 3, the samples cured for the respective curing periods of 7, 14 and 28 days are studied for 12 cycles of drying and wetting for durability. After completion of each segment of the durability cycle the dry and wet mass of the samples were recorded. Figures 4.5 - 4.7 shows the plots for these mass measurement. The plot (figure 4.5) for 7 days cuing shows a consistent variation of the wet and dry masses. But figures (figures 4.6 and 4.7) for 14 and 28 days curing periods show fluctuations in obtained masses.

This observation can be attributed to the reduction in permeability over the curing periods leading to the conclusion that addition of lime and gypsum are causing densification of the mass along with increase in extent of pozzolanic reaction and cementation bond formation. Also, the average value of the dry as well as wet masses are almost same with respect to different curing periods pertaining to the result that degree of saturation is consistent (almost 100%) for each cycle of drying & wetting durability studies.

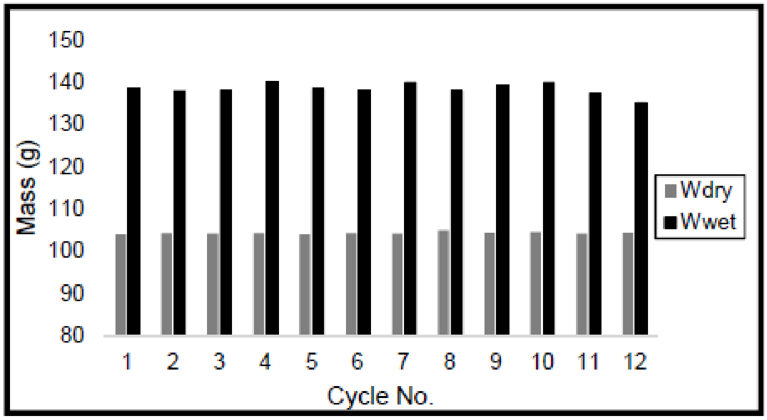


Figure 4.5 - Data of 7 Days Cured sample for Durability Cycles for Gypsum (G)

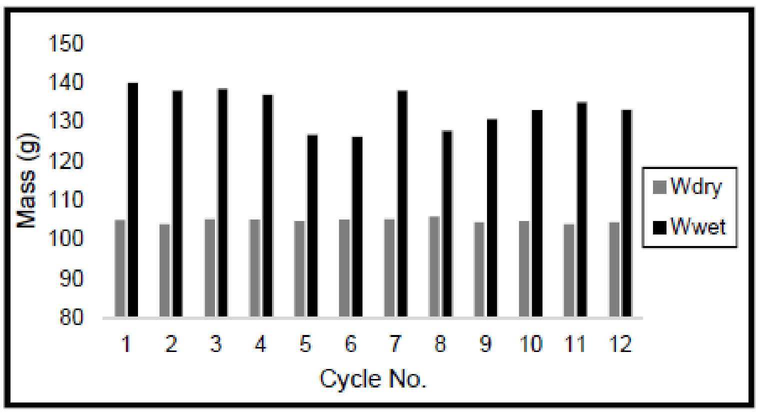


Figure 4.6 - Data of 14 Days Cured sample for Durability Cycles for Gypsum (G)

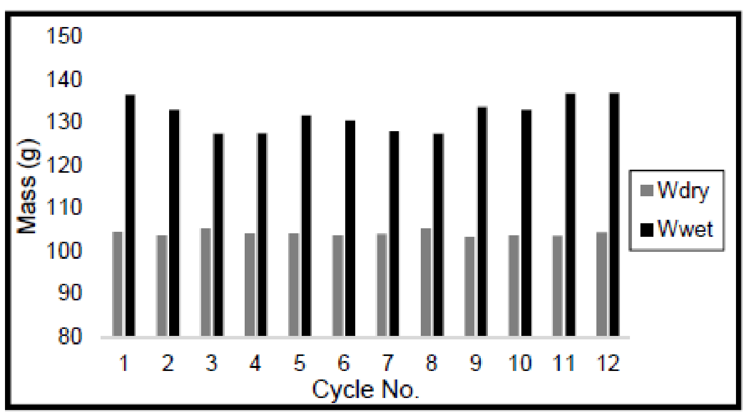


Figure 4.7 - Data of 28 Days Cured sample for Durability Cycles for Gypsum (G)

Avg. Mass (g)	Curing Period		
	7 Days	14 Days	28 Days
Wdry	104.65	105.01	104.91
Wwet	145.17	144.41	145.70

Table 4.2 – Average value of sample mass in durability studies for Gypsum (G)

4.4.2 Durability studies for fly ash – lime - phosphogypsum binder admixture

As stated earlier in Chapter 3, the samples cured for the respective curing periods of 7, 14 and 28 days are studied for 12 cycles of drying and wetting for durability. After completion of each segment of the durability cycle the dry and wet mass of the samples were recorded. Figures 4.3 - 4.5 shows the plots for these mass measurement. The plot (figure 4.8) for 7 days cuing shows a consistent variation of the wet and dry masses. But plots (figures 4.9 and 4.10) for 14 and 28 days curing periods show fluctuations in obtained masses. This observation can be attributed to the reduction in permeability over the curing periods leading to the conclusion that addition of lime and phosphogypsum are causing densification of the mass along with increase in extent of pozzolanic reaction and cementation bond formation. Also, the average value of the wet masses are almost same with respect to 7 and 14 days curing periods pertaining to the result that degree of saturation is consistent (almost 100%) for each cycle of drying & wetting durability studies. But for 28 days cured sample the water absorption is higher. This may be due to slight expansive characteristics of PG causing porous formation during higher curing periods.

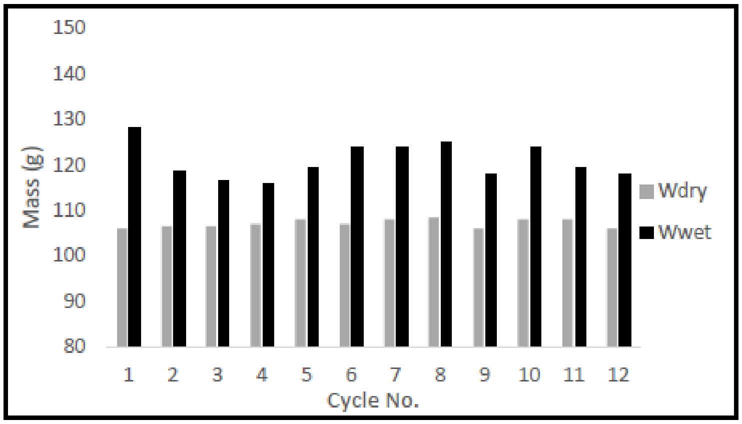


Figure 4.8 – Data of 7 days cured sample for durability cycles for Phosphogypsum (PG)

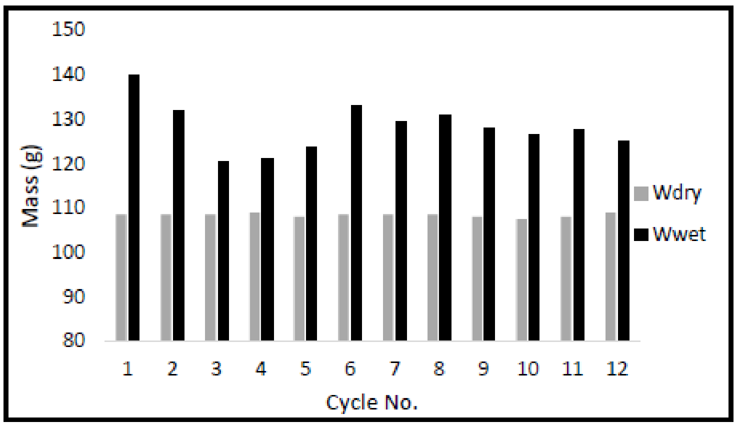


Figure 4.9 – Data of 14 days cured sample for durability cycles for Phosphogypsum (PG)

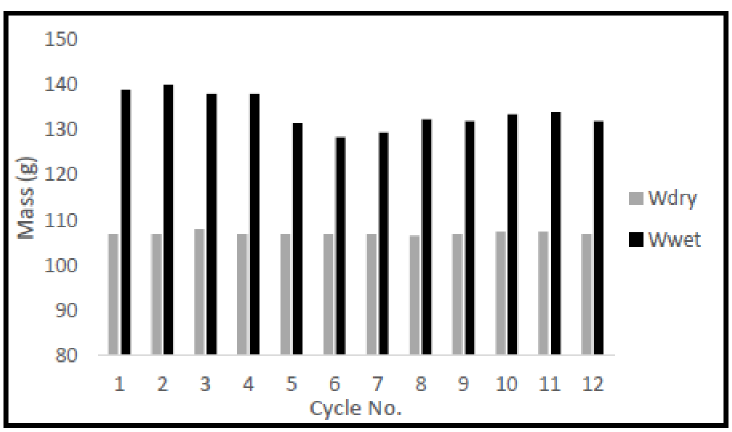


Figure 4.10 – Data of 28 days cured sample for durability cycles for Phosphogypsum (PG)

Avg. Mass (g)	Curing Period		
	7 Days	14 Days	28 Days
Wdry	107.12	107.77	106.42
Wwet	126.92	126.27	134.96

Table 4.3 – Average value of sample mass in durability studies for Phosphogypsum (PG)

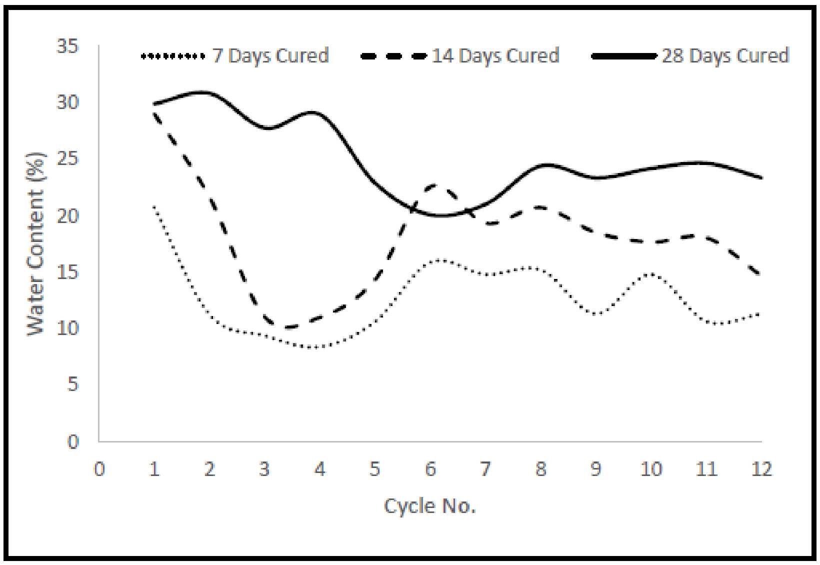


Figure 4.11 – Variation of water content with durability cycle for Phosphogypsum (PG)

4.5 Residual Unconfined Compressive Strength (RUCS)

As stated in section 3.2.3, the UCS measurements taken after 12 cycles of drying wetting can be termed as residual strength of or RUCS. Therefore, figures 4.12 and 413 show the RUCS results of 7, 14 and 28 days cured samples by burlap method and studied for 12 cycles of drying & wetting for durability and the trend shows that the reference mix is showing a consistent strength with curing time period concluding that the extent of pozzolanic reaction of lime with accelerated curing due to rapid heating and wetting leads to gain sufficient strength at early curing period of 7 days with a very little variation for more curing days. In other words, it can be said that the accelerated curing leads a sample to reach its maximum possible UCS value which is independent of curing period. It will be interesting to study a minimum threshold curing period before conducting durability studies so that in field the mixture cured for that threshold value can be directly posed to durability effects without wondering about its failure because of its low UCS at early curing periods.

Also, the decrease in percentage failure strain with curing period is the evidence of hardness and brittleness of the sample with curing period (for fly ash-lime-gypsum).

Also, the similar percentage failure strain with curing period is the evidence of hardness and brittleness of the sample with curing period (for fly ash-lime-phosphogypsum).

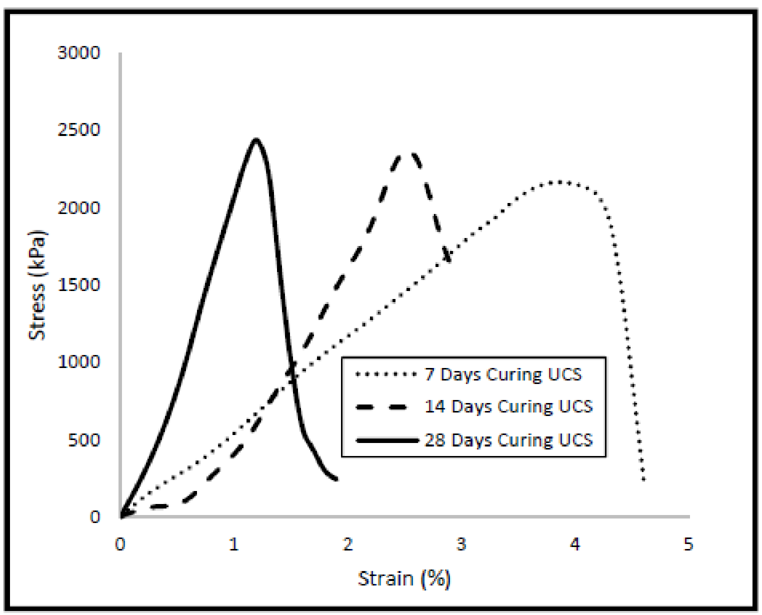


Figure 4.12 - RUCS Plots of 7, 14 and 28 days cured samples for Gypsum (G)

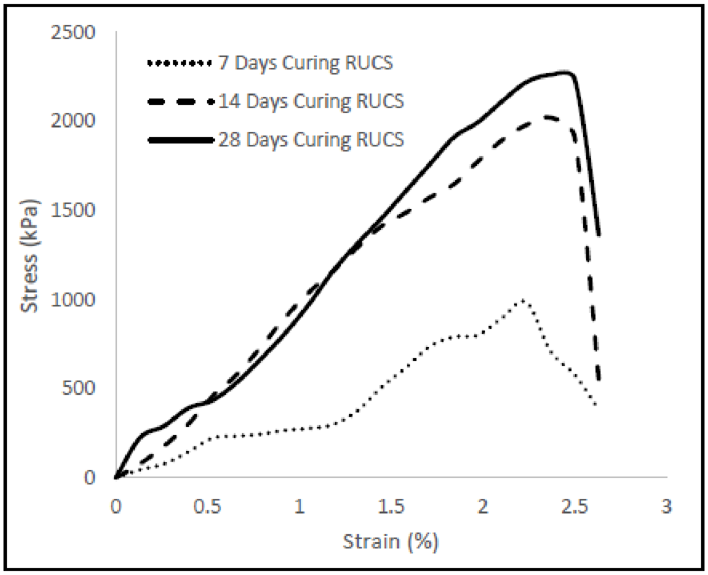


Figure 4.13 – RUCS plots of 7, 14 and 28 days cured samples for Phosphogypsum (PG)

4.6 Comparison of UCS and RUCS

As shown in section 4.3 and 4.5; the UCS and RUCS results for 7, 14 and 28 days curing periods, the figures 4.14 and 4.15 show the variation of UCS and RUCS over curing time. The slope of UCS v/s curing time period is sufficiently larger than that of RUCS v/s curing time concluding that extent of pozolanic reaction is linearly considerably increasing over time for simple curing. Whereas the variation in RUCS is very lower with curing time concluding that the accelerated curing causes sufficient completion of the extent of pozzolanic reaction. Also, the difference between the measurements of UCS and RUCS are considerably reducing with time which clearly affirms the assertion made above about the extent of pozolanic reaction.

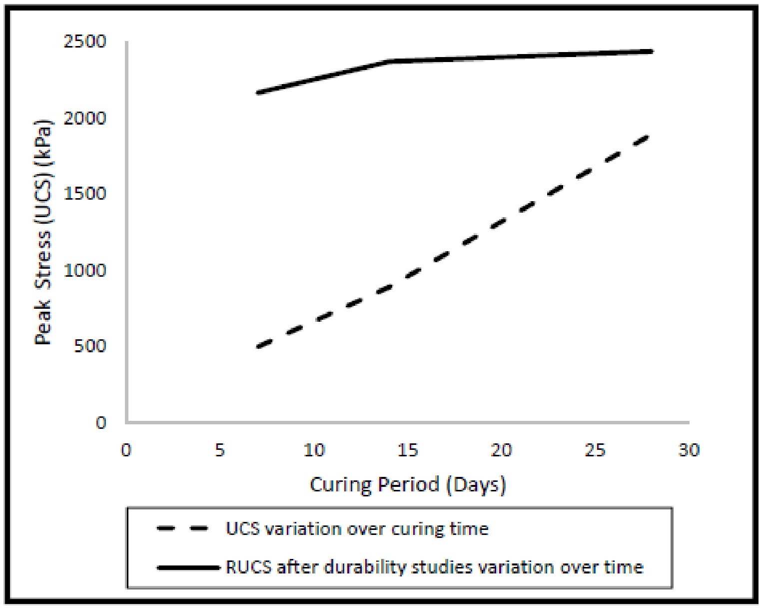


Figure 4.14 – Variation of Unconfined Compressive Strength over curing time for Gypsum (G)

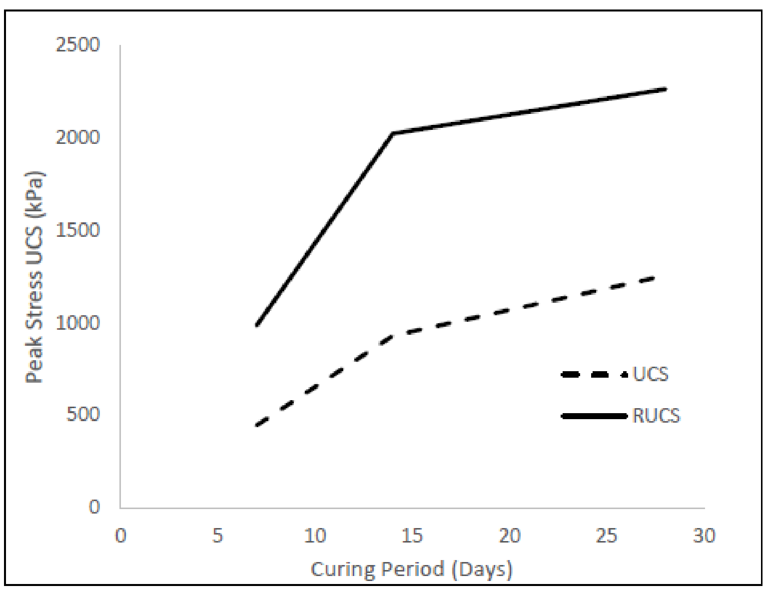


Figure 4.15 – Variation of Unconfined Compressive Strength over curing time for Phosphogypsum (PG)

In addition to this tables 4.4 and 4.5 show the values of UCS and RUCS obtained at their respective failure strains. The trend here observed reveals that during all curing periods the failure strain after the durability studies is more than that of UCS just after curing. This result attributes to the fact that the wetting drying cycles at all curing periods leads to the increase in ductility of the sample in addition to increasing its strength. This ductility increment is reduced considerably as the curing period starts increasing to higher values and the sample becomes more and more brittle with increasing curing period followed durability studies.

Unconfined Compressive Strength Readings	Curing Period					
	7 days		14 days		28 days	
	Strain (%)	Stress (kPa)	Strain (%)	Stress (kPa)	Strain (%)	Stress (kPa)
UCS	3.3	497.31	2.4	889.53	1.9	1892.33
RUCS	3.9	2164.58	2.5	2368.45	1.2	2436.69

Table 4.4 – Comparison of UCS with RUCS for Gypsum (G)

Unconfined Compressive Strength Readings	Curing Period					
	7 Days		14 Days		28 Days	
	Strain (%)	Stress (kPa)	Strain (%)	Stress (kPa)	Strain (%)	Stress (kPa)
UCS	1.05	444.98	1.18	929.51	1.71	1254.75
RUCS	2.24	985.29	2.37	2022.59	2.37	2263.12

Table 4.5 – Comparison of UCS with RUCS for Phosphoypsum (PG)

4.7 Comparison

On similar guidelines of compaction, UCS and Durability studies the mixture of Fly ash + 8.0% Lime + 1.0% Gypsum was studied in Major Project - 1 (August - December 2014). Therefore, following section is dedicated to comparison of UCS and RUCS with various facts and observations supporting the comparative studies.

4.7.1 Comparison of UCS measurement

Figure 4.16 (see below) displays the comparison of UCS studies respectively done with Gypsum (G) and Phosphogypsum (PG) modifiers.

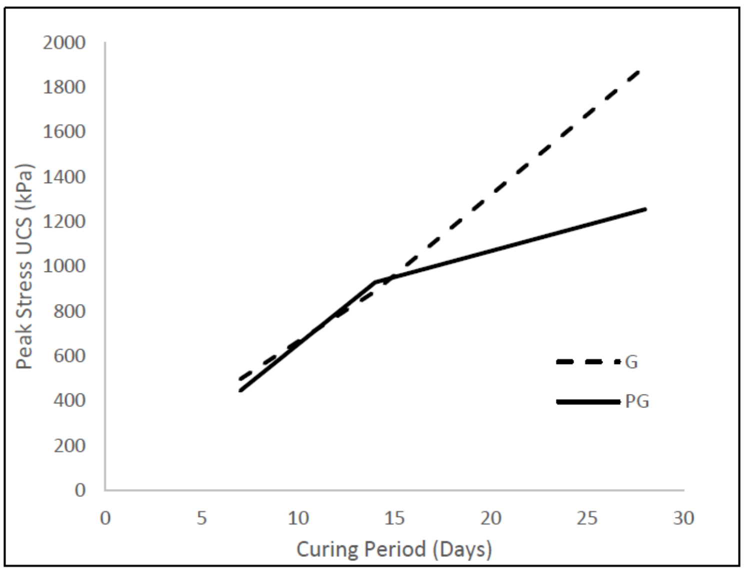


Figure 4.16 – Comparison of UCS over curing time with G and PG

The comparative studies of G & PG as a modifier in Fly ash + Lime mixture with 7, 14 and 28 days of curing reveals that the 7 and 14 days curing UCS is comparatively similar with both modifiers. Whereas the 28 days curing UCS differs significantly revealing the observation that rate of strength gain through pozzolanic reaction is less in PG in comparison to G. This may be attributed to the fact that PG is a waste material and level of impurities in PG is significantly higher than in G. Hence in early curing periods maximum amount of PG is utilized in pozzolanic reaction leading to only a slight gain in strength during 15th to 28th day of curing. Hence a gentle slope is observed in PG modifier (Figure 4.16). In addition to this the lesser extent of pozzolanic reaction in PG reveals the fact that water absorption is lesser in 7, 14 and 28 days of curing Periods. This fact is evident from Figure 4.17 which displays the water content of the 7, 14 and 28 days cured sample after completion of respective curing period.

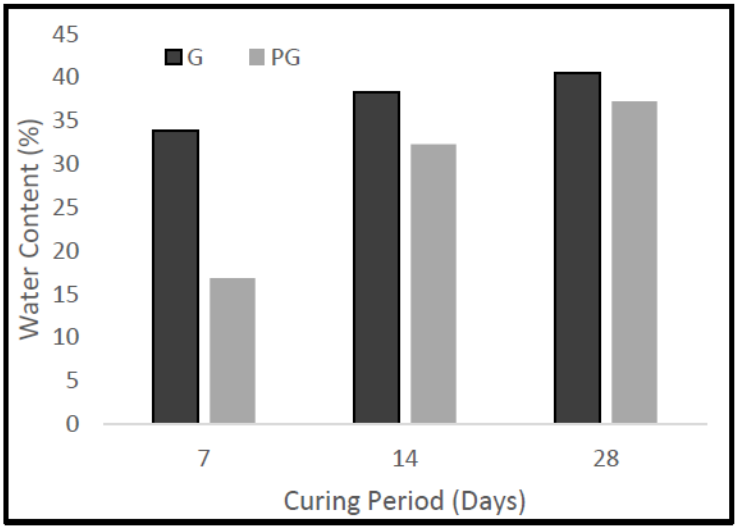


Figure 4.17 – Water content measurement just after curing

4.7.2 Comparison of RUCS measurement

Figure 4.18 displays the comparison of RUCS studies respectively done with Gypsum (G) and Phosphogypsum (PG) modifiers.

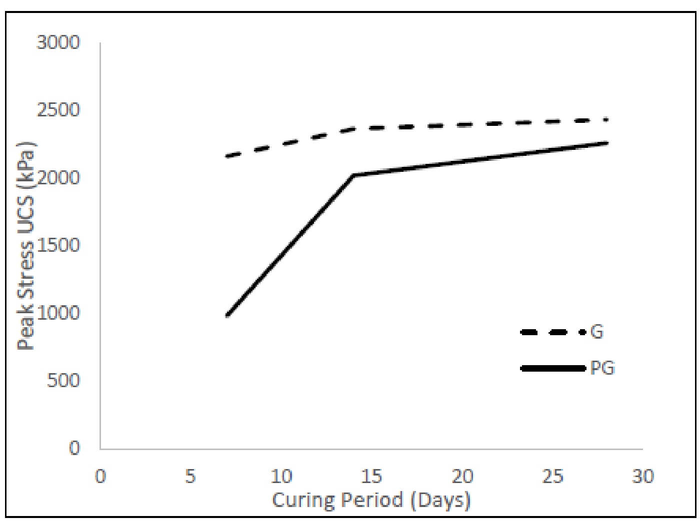


Figure 4.18 – Comparison of RUCS with G and PG

The comparative studies of G & PG as a modifier in Fly ash + Lime mixture with 7, 14 and 28 days of curing and 12 cycles of wetting, drying for durability studies reveals that the 7 days curing RUCS with PG and G is significantly different from each other. This observation indicated that the affinity for accelerated pozzolanic reaction in early curing period is much stronger in G In comparison to PG. This observation can be supported from the fact that G being a pure modifier in Fly ash and Lime mix leads to better water absorption during wetting cycle which consequently enhances the extent of the pozzolanic reaction. Whereas the water absorption and hence the rate of pozzolanic reaction is comparatively very less for PG (an impure, waste modifier) cured for 7 days. The lesser water absorption in PG is clearly visible from figure 4.19 (a).

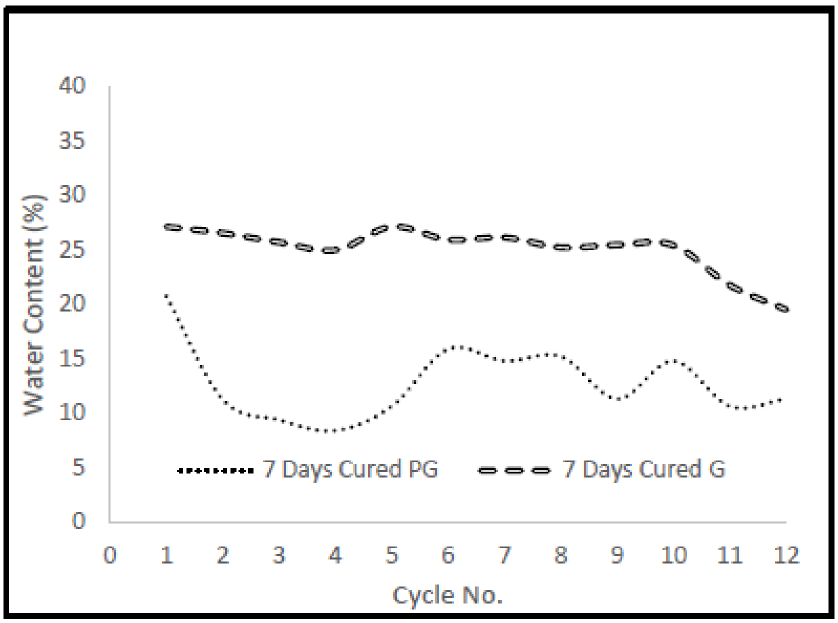


Figure 4.19 (a) – Water content plot for durability cycles with G and PG (7 days)

In addition, for 14 and 28 days of curing as evident from figure 4.18 the RUCS studies reveals that the slope is nearly horizontal indicating the observation that the extent of accelerated pozzolanic reaction has reached up to an optimum point after which the gain in strength is very less. Now, both PG and G display similar observation with only one small difference of the numerical value being slightly less for PG than G. This reveals that both these modifies accelerate the pozzolanic reaction process but PG being inferior in quality suffers from lesser numerical value of RUCS. Also, similar to figure 4.19 (a) showing variation of water content with durability studies cycle for 7 days cured sample, the same profile for 14 and 28 days of curing is also plotted as shown in figure 4.19 (b) and (c).

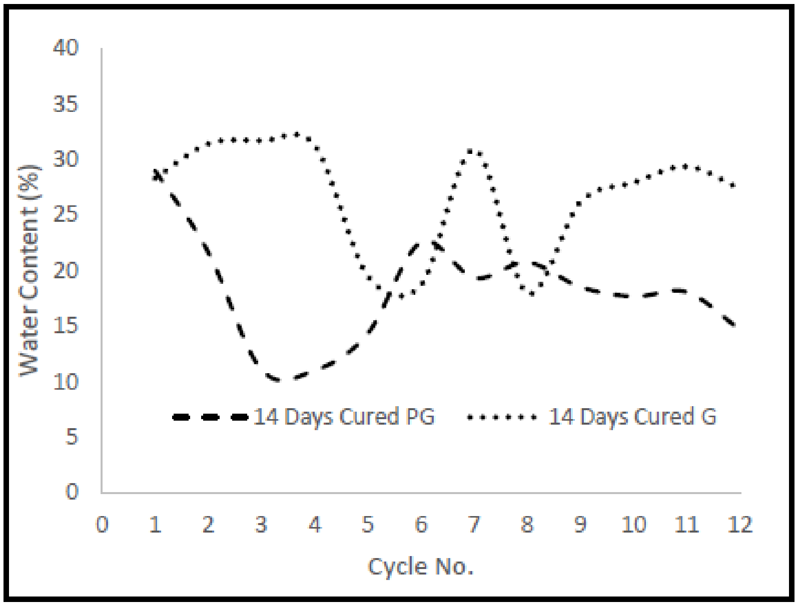


Figure 4.19 (b) – Water content plots for durability cycles with G and PG (14 days)

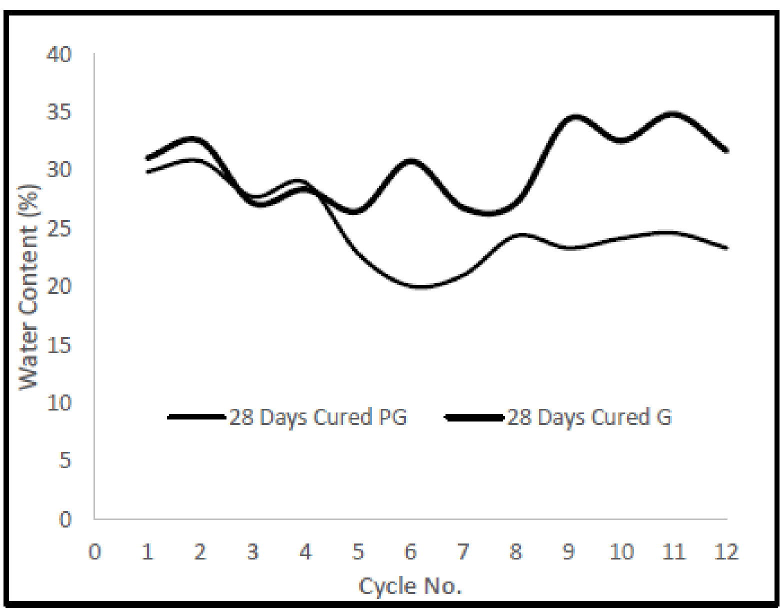


Figure 4.19 (c) – Water content plots for durability cycles with G and PG (28 days)

These figures clearly signify the observations of almost same RUCS value for 14 and 28 days of curing as water content variation is almost similar. Also, the water content profile for 14 days cured (figure 4.19 (b)) is significantly different for G in comparison to PG. Again the similar facts of impurity and affinity for water absorption are applicable here also. Whereas for 28 days cured (figure 4.19 (c)) the water content profile is almost similar for G and PG revealing that the RUCS has reached a saturation or optimum value after which the rate of strength gain is almost stagnate.

4.7.3 Sample Failure Pattern Comparison

In addition to this figure 4.20 (a) and 4.20 (b) shows the snapshots of samples under UCS & RUCS measurement for Gypsum (G) and Phosphogypsum (PG) modifiers respectively.

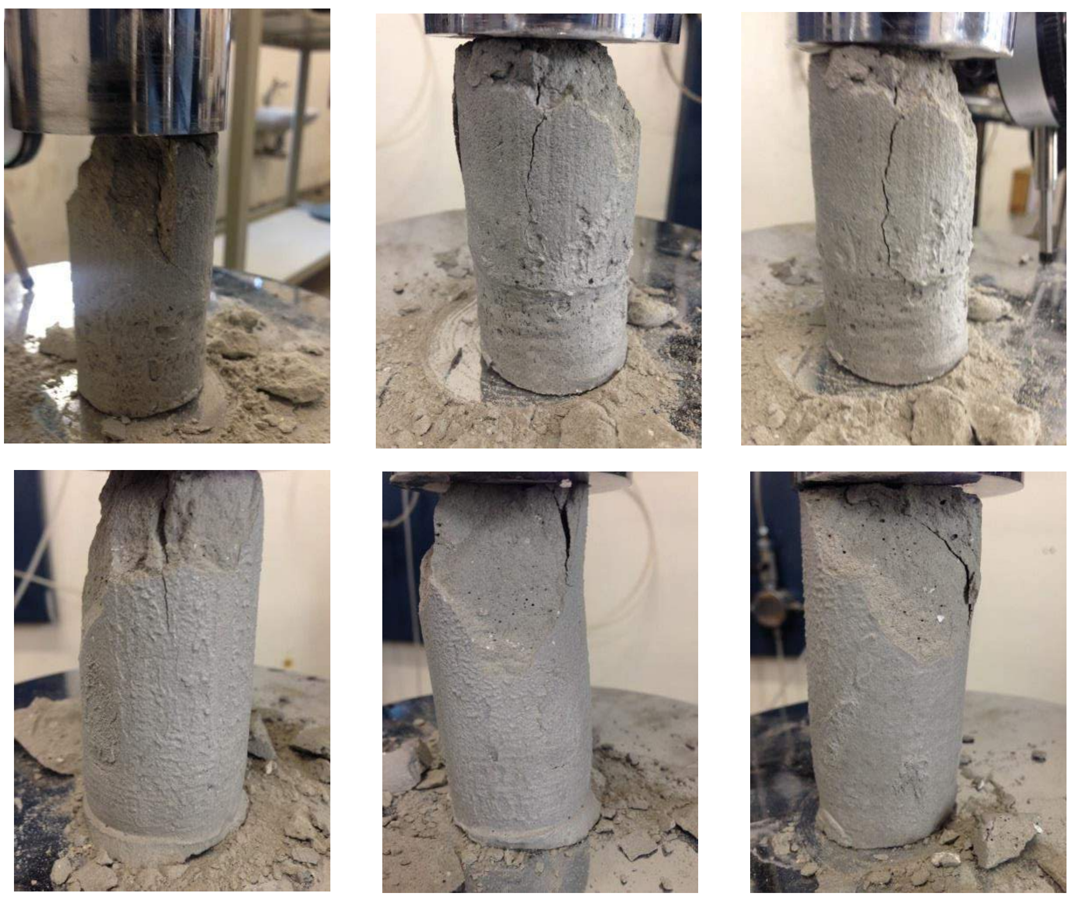


Figure 4.20 (a) – Samples under UCS (top) and RUCS (bottom) for Gypsum (G) modifier

Figure 4.20 (b) – Samples under UCS (top) and RUCS (bottom) for Posphogypsum (PG) modifier

Both these figures clearly indicate that samples tested for UCS just after curing showed nearly body failure for G as well as for PG. This indicates that the process of strength gain during curing is very much similar with G as well as PG. The only difference observed is in magnitude of UCS; being very less in PG in comparison to G probably due to lesser quality of PG in terms of purity. Whereas, samples studies for RUCS (after curing and durability studies both) displayed surface failure as a hard curst of sample. This observation clearly attributed as the surface hardness. The extent of surface hardness was higher for PG in comparison to G, as observed by thick hard crust in case of RUCS studies in PG. This is also an evidence of the lesser RUCS value of PG because the water absorption is only limited to outer surface and not up to the body center leading to higher strength gain in surface crusts in comparison to body centers. This significant strength difference led into lesser RUCS value for PG. Whereas in case of G, the water percolation was up to body center leading to almost homogeneous strength gain in RUCS for G. Though the surface crust failure is also observed in G also, but the extent of the thickness of surface crust was very less as compared to in PG.

4.8 Experimental Conclusions

On the basis of the results obtained and comparison made, following conclusions have been drawn.

4.8.1 Fly ash + 8% Lime + 1% Gypsum studies

- Addition of Lime and Gypsum admixtures leads to increase in OMC and MDD which concludes that these admixtures are serving as filler material for fly ash.
- The UCS sample failure just after curing at different periods indicate that sample is gaining strength with curing due to increase in pozzolanic reaction extent with curing.
- The sample failure under UCS studies indicates that due to accelerated curing by wetting-drying cycle causes surface hardness and the extent of hardness increases with increase in curing period leading to brittle failure.
- The UCS value is almost same after durability cycles at different curing periods indicating that accelerated curing is efficient even at early curing periods and time and cost constraints for field conditions can be properly dealt with.
- As per IRC rural road manual recommendations, any stabilized combination of non-conventional materials giving UCS of 1720 kPa after 28 days curing can be used for Base or sub base layer of pavement construction.
- After completion of water curing and drying of surface the conventional bituminous wearing coat of 25mm thick is laid. The cost is less by 30% as compared to conventional subgrades.
- Fly ash lime soil stabilization works, probably better than the crushed stone or graded gravel. Fly ash with suitable amount of lime forms a hard and

impermeable mass, which provides a very good base and sub base course for highway pavements.

4.8.2 Fly ash + 8% Lime + 2% Phosphogypsum studies

- Addition of Lime and Phosphogypsum in Fly ash leads to increase in its maximum dry density (MDD) which concludes that these modifiers are serving as filler material for fly ash.
- The UCS measurement at different curing periods clearly indicate that sample is gaining strength with curing due to increase in pozzolanic reaction extent with progress in curing.
- The sample failure under RUCS studies indicates that due to accelerated curing by wetting-drying cycle causes surface hardness and the extent of hardness increases with increase in curing period leading to brittle failure as indicated in figure 4.7 indicating sharp peak of stress as curing period is proceeded.
- The RUCS value is almost same from 14 days of curing indicating that accelerated curing is efficient even at early curing periods and hence at field the time and cost constraints can be economically dealt with by preferring 14 days of curing rather than higher curing periods for better strength.

4.8.3 Comparative studies

- As per the requirement of minimum UCS (7 days curing) reported by 'Jha et al (2009) (Table 8)' it is clear that our present studies with G or PG as a modifier are meeting the needs of UCS requirement of Road Sub-base for Light traffic (690- 1400 kPa) in pavement construction. Though the UCS value for 7 days curing is not falling within this limit both with G or PG modifier (497.31 kPa & 448.98 kPa respectively) but this limit is efficiently met with accelerated curing after 7 days of curing (2164.58 kPa and 985.29 kPa respectively for G and PG). In fact, the 7 days curing RUCS for G modifier is meeting all the UCS requirements for pavement construction.

- The lesser water absorption in PG (Figures 4.19 (a), (b) and (c)) is a positive indicator of improved durability in comparison to PG. The Sub-base constructed using PG as a modifier is safer against rupture and water percolation.

4.8.4 Effect of Lime Content

- The strength development in Class F fly ashes occurs at a slow pace owing to lower lime contents. The addition of $Ca(OH)_2$ triggers the onset of pozzolanic reaction early by increasing the solubility of silica as it breaks the Si-O bonds in the silica rich glassy phases of fly ash. The hydration of fly ash begins immediately with the depolymerization of glassy phases releasing alumina and silica. Thus for fly ashes with different lime contents the unconfined strength increases with curing period. This is also observed to increase with the increase in lime content.
- However, the rate of increase in strength with increase in lime content reduces beyond 2.5% of lime. The glassy phase corresponding to alumino-silicates in the low lime-fly ash gets activated with the addition of high lime content which increases the hydroxyl ion supply, thereby maintaining uniform fly ash hydration. The hydration of lime-fly ash pastes generates compounds similar to the ones obtained in cement hydration.
- The average Ca to Si ratio is 0.75 – 1.75 and depends on the nature of fly ash as well as the curing conditions adopted. The metastable silicates present in cementitious fly ashes react with calcium ions in the presence of moisture to form water insoluble calcium-silicate hydrates.
- With curing, $Ca(OH)_2$ is consumed by the pozzolanic reactions resulting in the precipitation of various calcium silicates, aluminates, and alumino silicates. According to Plowman (1984) the addition of lime activates the silica and alumina phases in the following manner:

  $Ca^{+2} + SiO_2 + H_2O \rightarrow xCaO \cdot ySiO_2 \cdot zH_2O$ (Eqn… 2)

  $Ca^{+2} + Al_2O_3 + H_2O \rightarrow xCaO \cdot yAl_2O_3 \cdot zH_2O$ (Eqn… 3)

  $Ca^{+2} + Al_2O_3/Si_2O_3 + H_2O \rightarrow xCaO \cdot yAl_2O_3 \cdot zSiO_2 \cdot wH_2O$ (Eqn… 4)

- The hydroxide derived calcium silicates contribute to additional strength to the stabilized fly ashes. The surfaces of nucleating fly ash particles provide sites for $Ca(OH)_2$, C-S-H, and other hydration products to precipitate, thereby leading to a more homogeneous distribution of hydration products.

4.8.5 Effect of Gypsum Content

- Gypsum is known to accelerate strength by altering the course of hydration of calcium silicate which is predominantly formed in the early hydration stages. The addition of gypsum aids in the release of sulfate ions which react with alumina phase of fly ash Plowman (1984), as shown in Eq. below:

  $$Ca^{+2} + SO_4^{-2} + Al_2O_3 + H_2O \rightarrow xCaO \cdot yAl_2O_3 \cdot zCaSO_4 \cdot wH_2O \qquad \text{(Eqn… 5)}$$

- The hydration of fly ash is better in the presence of gypsum. Apart from calcium silicate hydrate, calcium aluminate hydrate is also formed, favoring the hydration of fly ash. Also the calcium hydroxide chemically combined with fly ash in the presence of gypsum does not subject to leaching and hence, the risk of leaching of calcium hydroxide from gypsum treated lime stabilized fly ash matrix is minimized.
- The effect of gypsum is generally slower at higher lime contents. A better fly ash respond to lime can exhibit accelerated strength in the presence of gypsum early.
- The development of strength in fly ash at different lime contents is due to the formation of calcium-silicate hydrates of complex compositions. In the presence of gypsum apart from the formation of calcium-silicate hydrate, calcium aluminum silicate hydrate is also formed.
- The Scanning Electron Microscope (SEM) photographs show more densely precipitated and crystalline structure at higher lime contents particularly with gypsum. With the increase in curing period and at higher lime contents, these complex cementitious compounds are formed, increasing significantly the unconfined compressive strength of fly ash.

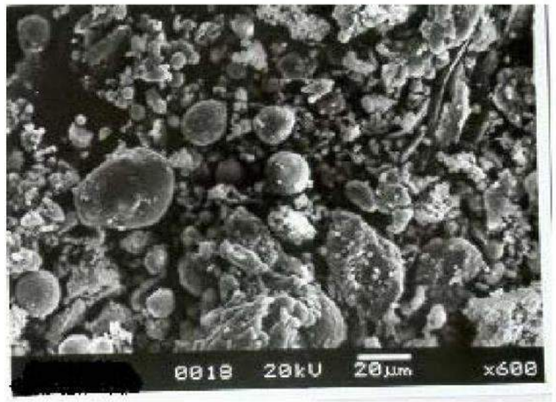


Figure 4.21 – SEM photograph of Class F Fly Ash (illustrative purpose only)

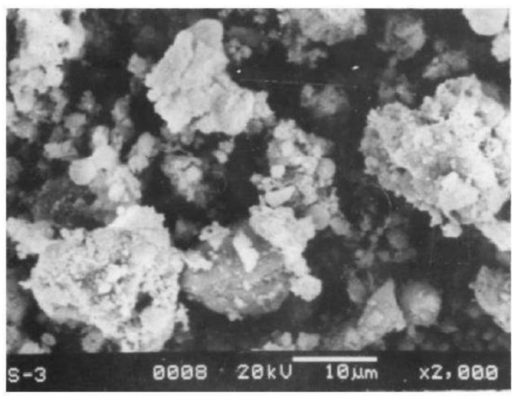


Figure 4.22 – SEM photograph of Fly Ash + Lime (8%) + Gypsum (1%) cured sample at 28 days (illustrative purpose only)

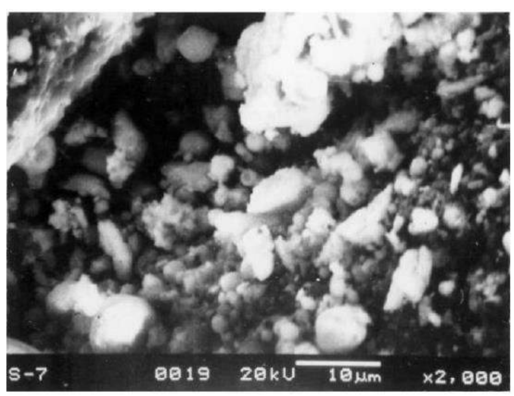


Figure 4.23 – SEM photograph of Fly Ash + Lime (8%) + Phosphogypsum (2%) cured sample at 28 days (illustrative purpose only)

4.8.6 Effect of Repeated Wet and Dry Cycles

- For stabilized materials, apart from the strength gain, durability plays a significant role. Durability against wetting and drying is important because in most of the geotechnical applications, stabilized fly ashes are subjected to repeated cycles of wetting and drying.
- Changes in compressive strength after a number of alternate wetting and drying cycles for fly ashes stabilized with lime and gypsum and cured for 7, 14 and 28 are studied and the results are presented in Tables 4.1, 4.2 and 4.3 in kilopascals respectively. There was no significant reduction in the strength of the fly ashes even at the end of 12 cycles of wetting and drying, though it is seen at a low lime content of 1%. This is because of the insufficient production of cementitious compounds in binding the particles.
- However, in the presence of gypsum, the loss of strength due to repeated wetting and drying cycles even at 1% of lime is minimized. Gypsum further reduces the loss of strength due to repeated wetting and drying cycles at higher lime contents. Actually the strength slightly increases in the presence of gypsum essentially due to the development of cementitious compounds during the process of wetting and drying.
- Thus it is interesting to note that the increased strength due to gypsum is also sustainable to repeated cycles of wetting and drying. The samples obtained after three wet and dry cycles were almost moisture resistant, enhancing the potential use of lime and gypsum stabilized fly ashes for a number of field applications.
- The strength of low lime-fly ashes which increases with lime content is significant up to an optimum lime content of about 5% and proceeds gradually thereafter.
- Addition of gypsum increases the strength of fly ashes at any lime content. At lower curing periods with lower lime contents the increase in strength with gypsum is quite significant. Increase in strength is observed at higher lime contents above 5% after a considerable period of 60 days. This increase in strength has been attributed due to the formation of calcium-sodium-aluminate-silicate hydrate along with calcium-silicate hydrate.
- Fly ash which responds readily to lime stabilization shows accelerated gain in strength due to the addition of gypsum at early curing periods.

- The increase in strength achieved with gypsum is not susceptible to the effect of alternate wet and dry cycles.

4.9 Pavement Models testing

4.9.1 Pavement Model (PM) 1 – Without fly ash-lime-gypsum/phosphogypsum binder admixture testing

The condition of Pavement Model (PM) 1 before and after test has been depicted in figures 4.24 (a) and (b). Figure 4.24 (a) shows Pavement Model (PM) 1 before starting UTM.

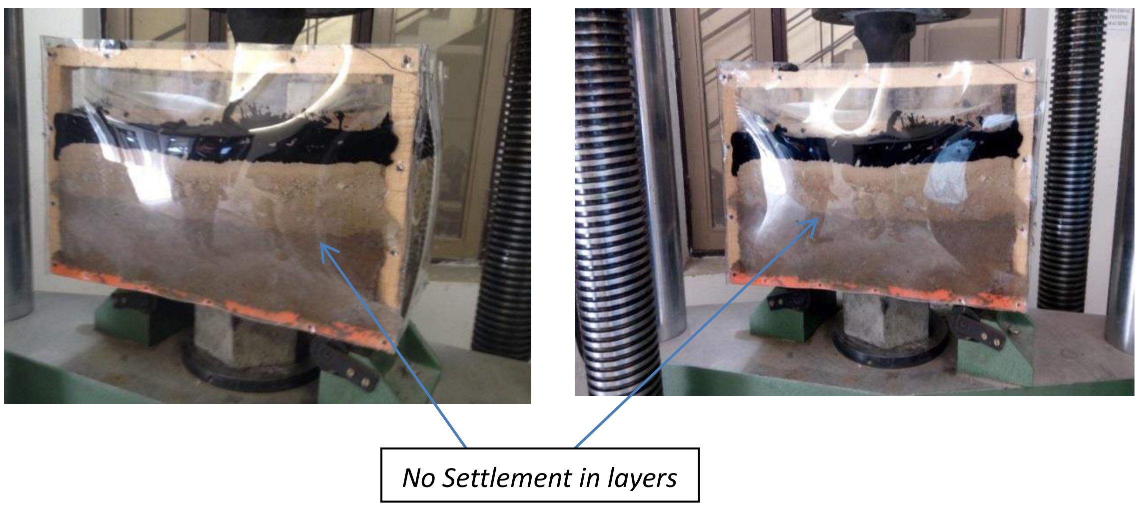


Figure 4.24 (a) PM 1 before starting UTM

Figure 4.24 (b) shows settlement in layers of PM 1 due to load after starting UTM.

Figure 4.24 (b) – PM 1 after starting UTM

The experimental readings of load (in kg) and settlement (mm) for PM 1 are shown in table 4.6.

Load (kN)	Settlement (mm)	Load (kg)
1	0	101.97
1.2	0	122.36
3.1	0	316.11
4.9	0	499.65
6.9	0	703.59
9.7	0	989.11
15.2	0.1	1549.94
15.4	0.2	1570.34
15.7	0.4	1600.93
16.2	0.6	1651.91
16.3	0.8	1662.11
16.5	1	1682.51
16.6	1.2	1692.70
16.8	1.3	1713.10
16.9	1.4	1723.29
17.1	1.6	1743.69
17.2	1.8	1753.88
17.3	1.9	1764.08
17.6	2.1	1794.67
17.8	2.3	1815.07
18	2.5	1835.46
18.2	2.7	1855.85
18.4	2.9	1876.25
19.4	4.2	1978.22
19.6	4.4	1998.61
19.8	4.7	2019.01
19.9	5	2029.20
20.1	5.2	2049.60
20.3	5.4	2069.99
20.5	5.7	2090.39
20.6	6	2100.58
20.8	6.2	2120.98
20.9	6.5	2131.17
21	5.7	2141.37
21.2	7.1	2161.76
21.4	7.4	2182.16
21.6	7.7	2202.55
21.7	7.9	2212.75
21.8	8.2	2222.95
21.9	8.4	2233.14
22	8.7	2243.34
22.2	9	2263.73
22.3	9.2	2273.93
22.4	9.4	2284.13
22.5	9.7	2294.33
22.3	10.4	2273.93

Table 4.6 – UTM Readings of PM 1

The shear box breaks completely at a load value of 2273.93 kg with a total settlement of 10.4 mm in layers of pavement model. The high value of settlement can be explained due to breaking of wooden plate and penetration of load plunger into the surface of shear box.

The load v/s settlement curves for PM 1 with and without adjustment are given in figures 4.25 (a) and (b). Figure 4.25 (a) gives load v/s settlement curve for PM 1 without adjustment. The wooden plate gets broken and the plunger of Universal testing machine penetrates the bitumen surface of the shear box. The readings are taken up to the final breaking of the wooden shear box. Figure 4.25 (b) gives load v/s settlement curve up to breaking of wooden shear box.

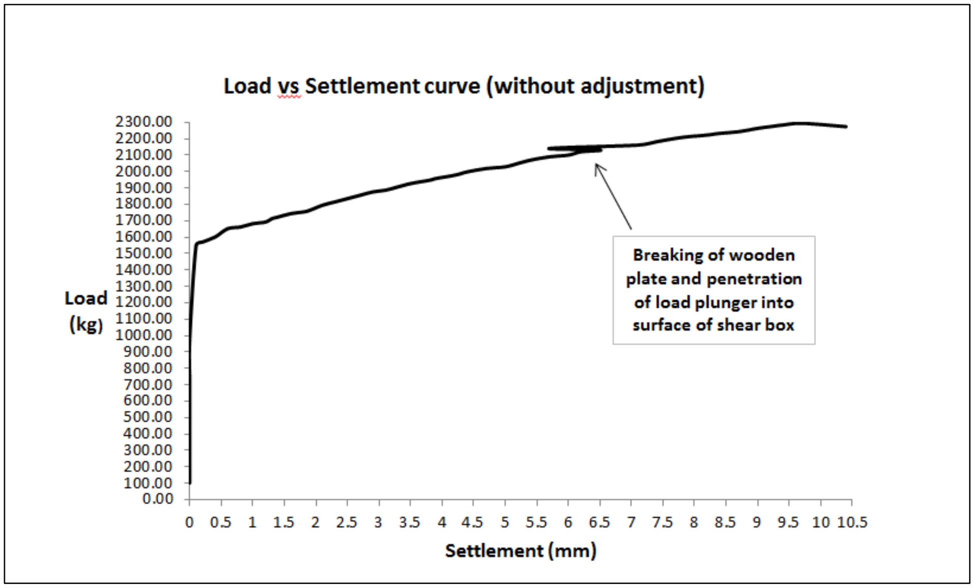


Figure 4.25 (a) – Load v/s settlement curve (without adjustment) for PM 1

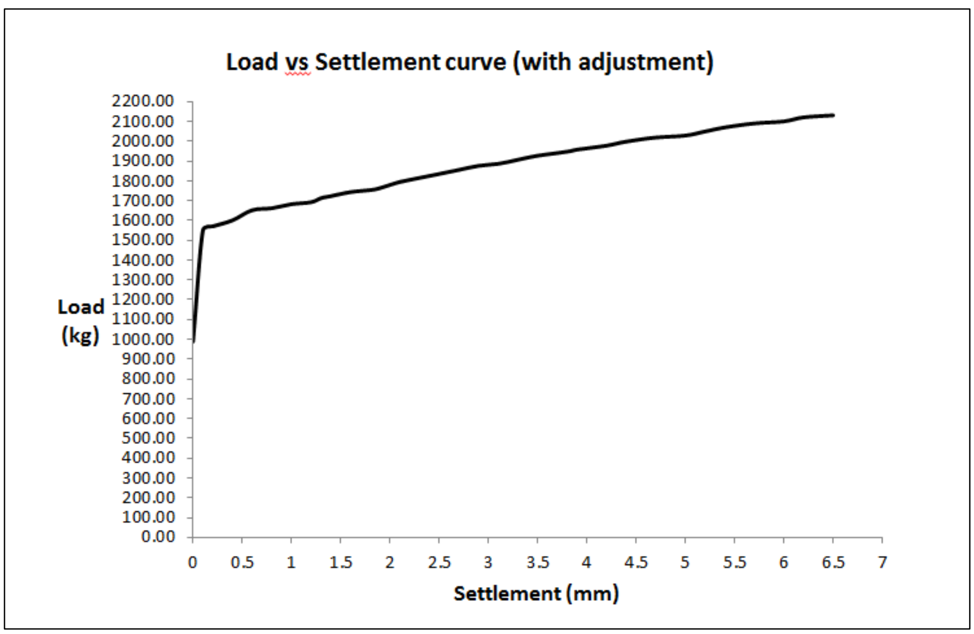


Figure 4.25 (b) – Load v/s settlement curve (with adjustment) for PM 1

The experimental readings of stress (kg/cm^2) and strain (%) for PM 1 are shown in table 4.7. The maximum value of stress is recorded at 1.89 kg/cm^2 with a corresponding value of 0.034 % strain for PM 1.

Stress (kg/cm^2)	Strain (%)
0.0850	0.0000
0.1020	0.0000
0.2634	0.0000
0.4164	0.0000
0.5863	0.0000
0.8243	0.0000
1.2916	0.0003
1.3086	0.0007
1.3341	0.0013
1.3766	0.0020
1.3851	0.0027
1.4021	0.0033
1.4106	0.0040
1.4276	0.0043
1.4361	0.0047
1.4531	0.0053
1.4616	0.0060
1.4701	0.0063
1.4956	0.0070
1.5126	0.0077
1.5296	0.0083
1.5465	0.0090
1.5635	0.0097
1.5720	0.0103

Stress (kg/cm^2)	Strain (%)
1.6825	0.0157
1.6910	0.0167
1.7080	0.0173
1.7250	0.0180
1.7420	0.0190
1.7505	0.0200
1.7675	0.0207
1.7760	0.0217
1.7845	0.0190
1.8015	0.0237
1.8185	0.0247
1.8355	0.0257
1.8440	0.0263
1.8525	0.0273
1.8610	0.0280
1.8695	0.0290
1.8864	0.0300
1.8949	0.0307
1.9034	0.0313
1.9119	0.0323
1.8949	0.0347

Table 4.7 – Stress (kg/cm^2) and strain (%) readings of PM 1

The stress v/s strain graph for PM 1 is shown in figure 4.26.

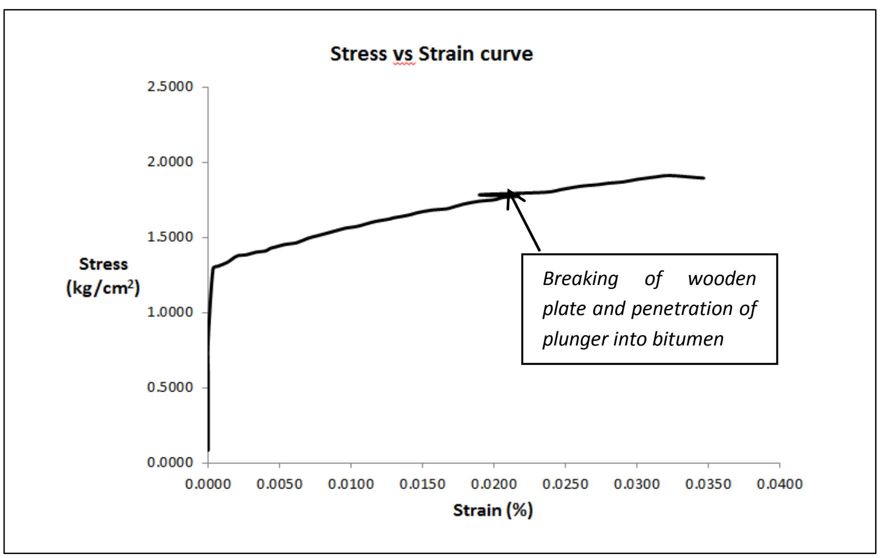


Figure 4.26 – Stress (kg/cm^2) v/s strain (%) curve for PM 1

The turn back in stress v/s strain curve (figure 4.26) for PM 1 is due to breaking of wooden plate and penetration of plunger into bitumen.

4.9.2 Pavement Model (PM) 2 – With fly ash – lime – gypsum binder admixture testing

The condition of Pavement Model (PM) 2 before and after test has been depicted in figures 4.27 (a) and (b). Figure 4.27 (a) shows Pavement Model (PM) 2 before starting UTM.

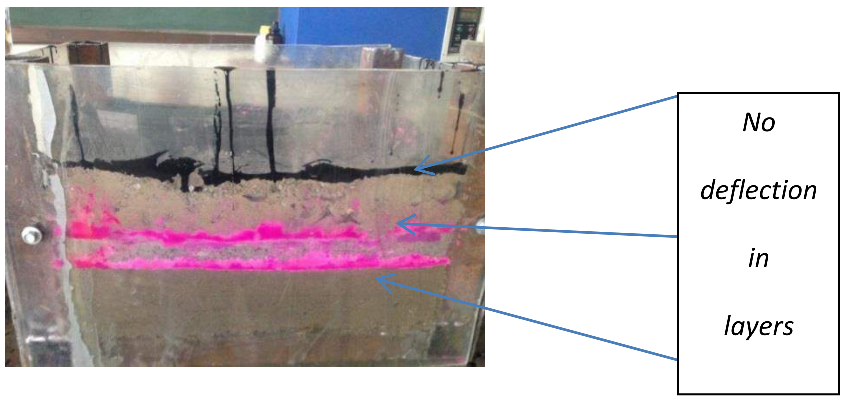


Figure 4.27 (a) – PM 2 before starting UTM

Figure 4.27 (b) shows settlement in layers of PM 2 due to load after starting UTM.

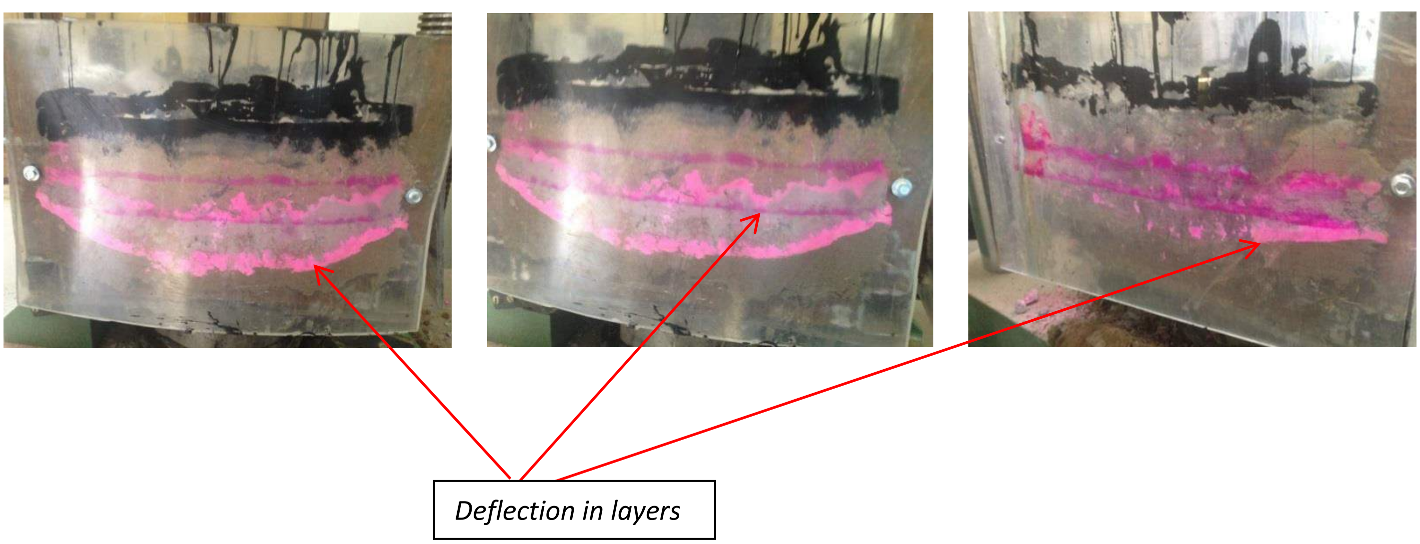


Figure 4.27 (b) – PM 2 after starting UTM

The experimental readings of load (in kg) and settlement (mm) for PM 2 are shown in table 4.8.

Load (kN)	**Settlement (mm)**	**Load (kg)**
0	0	0
5.2	0.5	530.244
10.2	0.7	1040.094
20.6	1.2	2100.582
21.3	1.23	2171.961
22.1	1.35	2253.537
23.1	1.49	2355.507
24.5	1.72	2498.265
25.7	1.97	2620.629
28.5	3.4	2906.145
35	3.6	3568.95
40	4.7	4078.8
47	6.1	4792.59
55	7.1	5608.35
57.5	7.52	5863.275
64.3	8.46	6556.671
77.6	8.8	7912.872
83.4	9.5	8504.298
87.2	10.5	8891.784
91.3	11.4	9309.861

Table 4.8 – UTM readings for PM 2

The very high value of load i.e. 9309.861 kg and settlement i.e. 11.4 mm were due to strong steel frame and steel plate. The load v/s settlement curve for PM 2 is shown in figure 4.28. For PM 2 no turn back is obtained in the curve as for PM 1 (see figure 4.25 (a)) due to only bending and non-breaking of steel plate used.

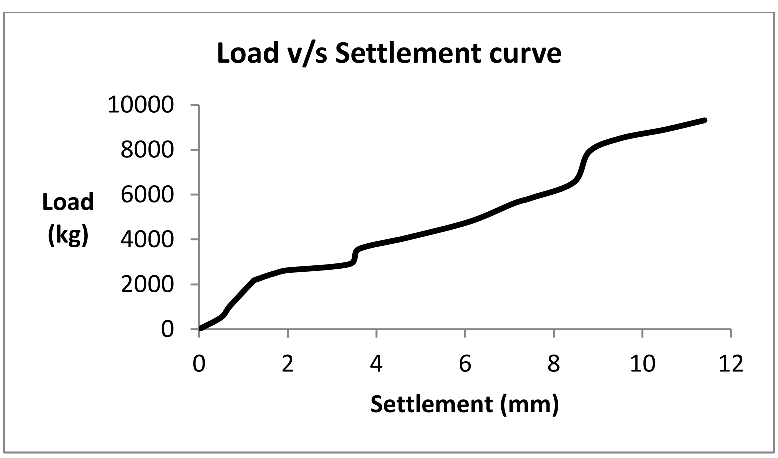


Figure 4.28 – Load v/s settlement curve for PM 2

The experimental readings of stress (kg/cm^2) and strain (%) for PM 2 are shown in table 4.9.

Stress (kg/cm^2)	Strain (%)
0.000	0.000
0.609	0.001
1.196	0.002
2.414	0.003
2.497	0.004
2.590	0.004
2.707	0.004
2.872	0.005
3.012	0.006
3.340	0.010
4.102	0.010
4.688	0.013
5.509	0.017
6.446	0.020
6.739	0.021
7.536	0.024
9.095	0.025
9.775	0.027
10.220	0.030

Table 4.9 - Stress (kg/cm^2) and strain (%) readings of PM 2

The stress v/s strain graph for PM 2 is shown in figure 4.29. The value of stress is 10.22 kg/cm^2 with a corresponding value of 0.03% strain for PM 2.

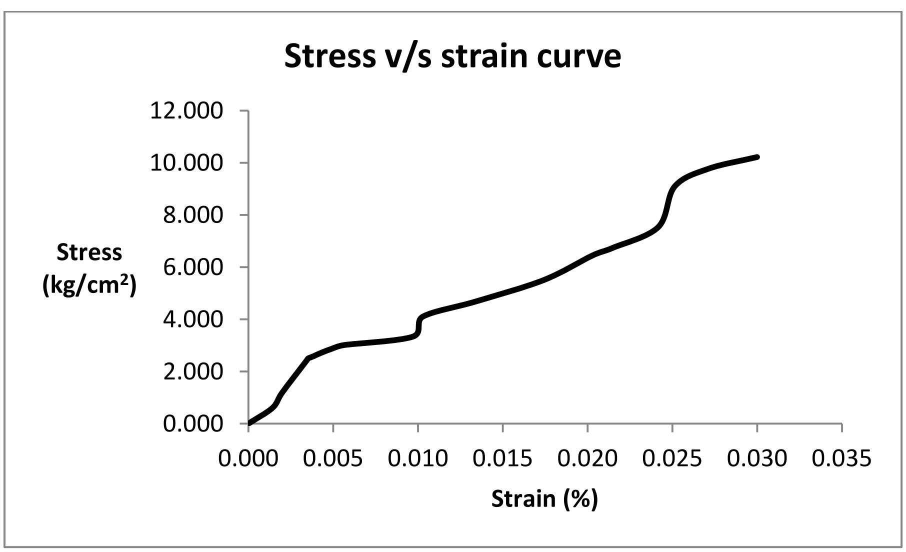


Figure 4.29 - Stress (kg/cm^2) vs strain (%) curve for PM 2

4.9.3 – Pavement Model (PM) 3 – With fly ash-lime-phosphogypsum binder admixture testing

The condition of Pavement Model (PM) 3 before and after test has been depicted in figures 4.30 (a) and (b). Figure 4.30 (a) shows Pavement Model (PM) 3 before starting UTM.

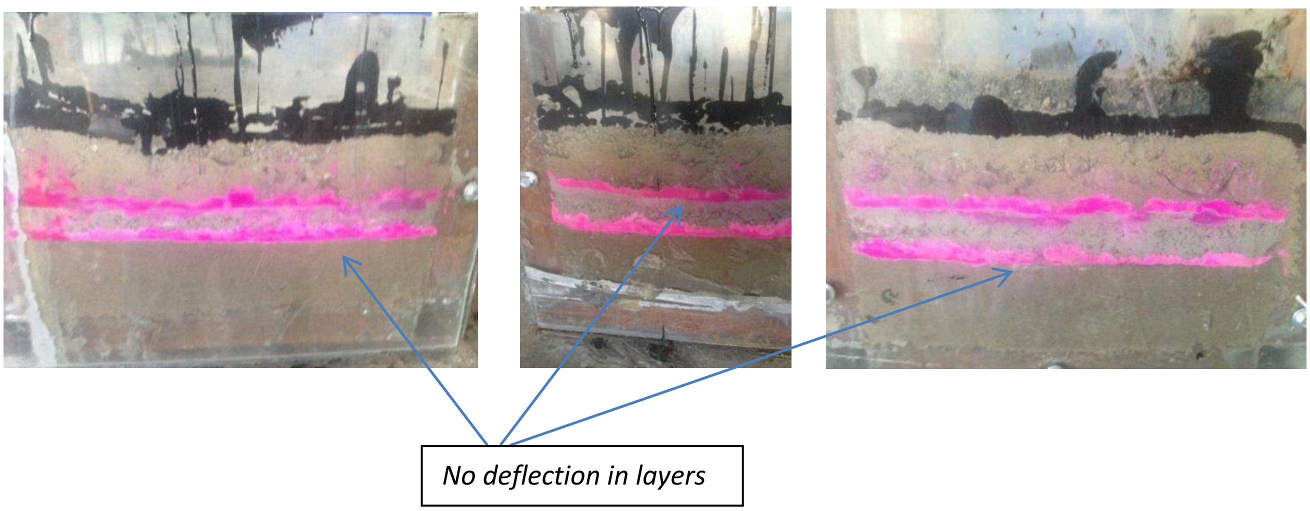


Figure 4.30 (a) – PM 3 before UTM testing

Figure 4.30 (b) shows PM 3 after starting UTM.

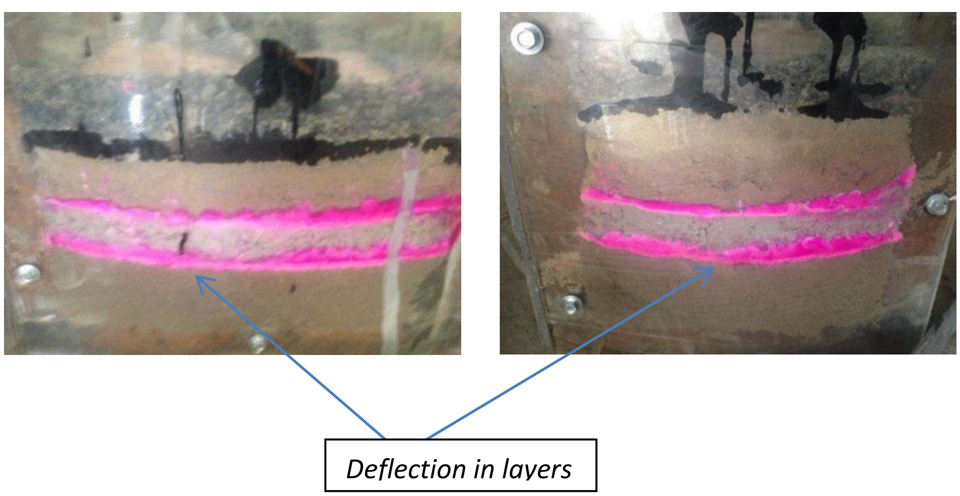


Figure 4.30 (b) – PM 3 after UTM testing

The experimental readings of load (in kg) and settlement (mm) for PM 3 are shown in table 4.10.

Load (kN)	Settlement (mm)	Load (kg)
0	0	0
2.5	0.2	254.8175
4.2	0.6	428.0934
6.1	1	621.7547
8.7	1.1	886.7649
11.3	1.3	1151.7751
14.8	1.5	1508.5196
16.9	1.9	1722.5663
20	2.5	2038.54
25.5	3.3	2599.1385
30.3	4.3	3088.3881
33	5	3363.591
40	6.1	4077.08
42.5	7.1	4331.8975
47	7.5	4790.569
51	8	5198.277
57.5	8.2	5860.8025
60	8.7	6115.62

Table 4.10 – UTM readings for PM 3

The high values of load i.e. 6115.62 kg and settlement i.e. 8.7 mm were due to steel shear box and iron plate. The load v/s settlement curve for PM 3 is shown in figure 4.31. For PM 3 no turn back is obtained in the curve as for PM 1 (see figure 4.25 (a)) due to only bending and non-breaking of steel plate used.

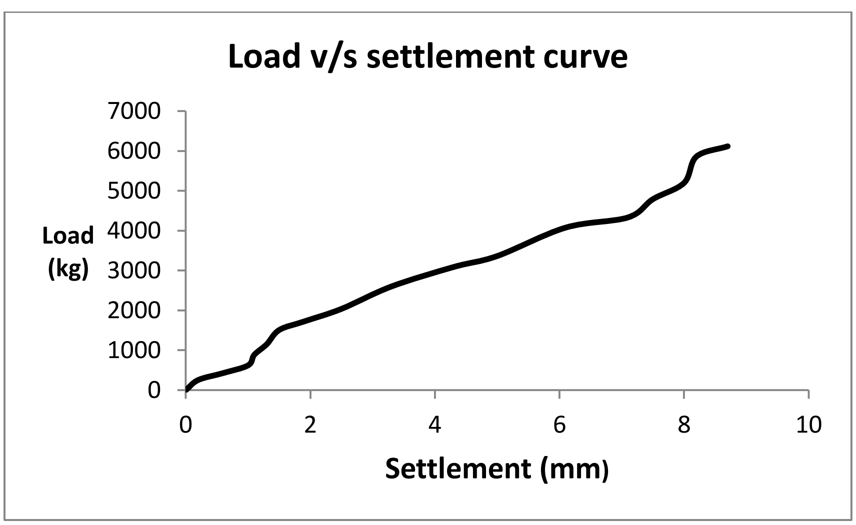


Figure 4.31 – Load v/s settlement curve for PM 3

The experimental readings of stress (kg/cm^2) and strain (%) for PM 3 are shown in table 4.11.

Stress (kg/cm^2)	Strain (%)
0.000	0.000
0.293	0.001
0.492	0.002
0.715	0.003
1.019	0.003
1.324	0.004
1.734	0.004
1.980	0.005
2.343	0.007
2.988	0.009
3.550	0.012
3.866	0.014
4.686	0.017
4.979	0.020
5.506	0.021
5.975	0.023
6.737	0.023
7.029	0.025

Table 4.11 - Stress (kg/cm^2) and strain (%) readings of PM 3

The stress v/s strain graph for PM 3 is shown in graph 4.32. The value of stress is 7.02 kg/cm^2 with a corresponding value of strain of 0.025 % for PM 2.

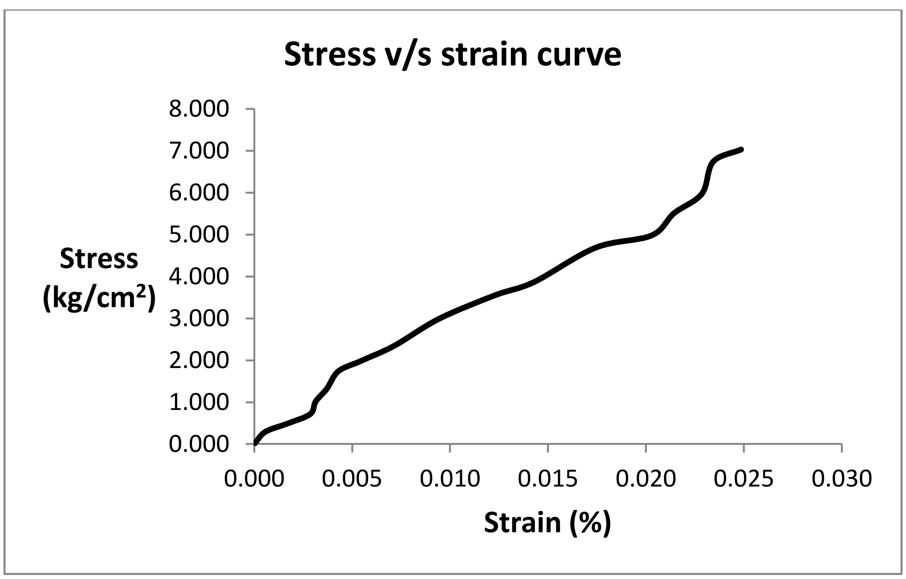


Figure 4.32 - Stress (kg/cm^2) vs strain (%) curve for PM 3

4.10 Determination of Modulus of Subgrade Reaction (k) and Modulus of Elasticity (E) of different pavement models

4.10.1 Determination of Modulus of Subgrade Reaction (k) of different pavement models

The specified deformation level is 1.25 mm and standard plate size for finding k-value is 0.75 m diameter. A plot was made between mean settlement and load (section 4.9). The pressure (p) corresponding to a settlement of 1.25 mm was calculated and the k value was determined by equation 6

k_{30} = p/1.25 kg/m^2/mm (Eqn...6)

The k value corresponding to the standard size of the plate calculated from equation 7

k_{75} = K_{30} x (30/75) (Eqn…7)

The values of modulus of subgrade reaction k_{30} and k_{75} are shown in table 4.12.

Pavement Model (PM)	k_{30} (kg/cm^2/cm)	k_{30} (kg/m^3 x 10^4)	k_{75} (kg/cm^2/cm)	k_{75} (kg/m^3 x 10^4)
PM 1	11.35	1135	4.54	454
PM 2	20.05	2005	8.02	802
PM 3	9.37	937	3.75	375

Table 4.12 – Values of k_{30} and k_{75} for different pavement models

4.10.2 Determination of Elastic Modulus (E) of different pavement models

The modulus of elasticity of subgrade and base course is determined by is determined by using Burmister's equation for a design deflection of 2.5 mm.

Δ = 1.18 x (pa/E) x F (Eqn…8)

where p = pressure (kg/cm^2)

a = radius of plate (cm)

F= displacement factor

E = modulus of elasticity of subgrade (kg/m^2)

Value of F = 1for subgrade

Value of F = 3 – 8.5 – 20 for fly ash-lime-gypsum binder admixture

The values of modulus of elasticity of different pavement models are shown in table 4.13.

Pavement Model (PM)	E (kg/m^2) x 10^4
PM 1	251.17
PM 2	2551.67
PM 3	1192.37

Table 4.13– Values of E for different pavement models

4.11 Cost Analysis and Its Comparison

The cost of materials of three pavement models calculated as per PWD Kullu schedule of rates is given below. The table 4.14 gives comparison of quantity of materials for different pavement models (PMs).

S. No.	Particulars of Materials and Details of works	Length (m)	Breadth (m)	Height or Depth (m)	Quantity	Unit
1	**Density of mix**					
	PM 1				2200	kg/m^3
	PM 2				2242	kg/m^3
	PM 3				2277	kg/m^3
2	**Quantity of gravel**					
	PM 1 (100 %)	0.4	0.3	0.04	40.56 kg	kg
	PM 2 (80 %)	0.29	0.3	0.04	23.75 kg	kg
	PM 3 (70 %)	0.29	0.3	0.04	21.11 kg	kg
3	**Quantity of coarse aggregates**					
	PM 1	0.4	0.3	0.03	5.59 kg	kg
	PM 2	0.29	0.3	0.03	4.09 kg	kg
	PM 3	0.29	0.3	0.03	4.16 kg	kg
4	**Quantity of fine aggregates**					
	PM 1	0.4	0.3	0.02	0.46 kg	kg
	PM 2	0.29	0.3	0.02	0.34 kg	kg
	PM 3	0.29	0.3	0.02	0.34 kg	kg
5	**Quantity of crushed aggregates**					
	PM 1	0.4	0.3	0.02		kg
	PM 2 (20 %)	0.29	0.3	0.02	5.93 kg	kg
	PM 3 (30 %)	0.29	0.3	0.02	9.04 kg	kg
6	**Total quantity of mix**					
	PM 1	0.4	0.3	0.175	**46.62 kg**	kg
	PM 2	0.29	0.3	0.175	**34.13 kg**	kg
	PM 3	0.29	0.3	0.175	**34.67 kg**	kg

Table 4.14 – Abstract of quantity of materials for different pavement models

As can be seen from table 4.14 the highest quantity of materials i.e. 46.62 kg are used for pavement model 1 (PM 1). This is due to larger dimensions of shear box for PM 1. The quantities of materials are nearly same for PM 2 and PM 3 due to similar dimensions of shear box.

The table 4.15 gives comparison of cost of materials for different pavement models.

S. No.	Particulars of materials and Details of works	Quantity	Unit	Rate (Rs.)	Per	Amount (Rs.)
1	**Gravel**					
	PM 1 (100 %)	40.56	kg	0.12	/ kg	4.86
	PM 2 (80 %)	23.75	kg	0.12	/ kg	2.85
	PM 3 (70 %)	21.11	kg	0.12	/ kg	2.53
2	**Coarse aggregates**					
	PM 1	5.59	kg	0.14	/ kg	0.78
	PM 2	4.09	kg	0.14	/ kg	0.57
	PM 3	4.16	kg	0.14	/ kg	0.58
3	**Fine aggregates**					
	PM 1	0.46	kg	1.2	/ kg	0.55
	PM 2	0.34	kg	1.2	/ kg	0.4
	PM 3	0.34	kg	1.2	/ kg	0.4
4	**Crushed aggregates**					
	PM 1					
	PM 2	5.93	kg	0.268	/ kg	1.58
	PM 3	9.04	kg	0.268	/ kg	2.42
5	**Binder admixture**					
	PM 1					
	PM 2	0.6	kg	5	/ kg	3
	PM 3	0.6	kg	6	/ kg	3.6
	Total cost of materials					
	PM 1					**6.19**
	PM 2					**8.4**
	PM 3					**9.53**
	Cost of materials per m^3					
	PM 1					**294.76**
	PM 2					**551.72**
	PM 3					**625.94**

Table 4.15 – Abstract of cost of materials for different pavement models

As can be seen from table 4.15 the highest cost of materials i.e. Rs. 625.94 per m^3 is for pavement model 3 (PM 3). This is due to higher cost of phosphogypsum for PM 3. PM 1 uses no binder admixture so its cost of materials is lowest. The cost of materials for PM 2 is intermediate between that of PM 1 and PM 3. This is due lower cost of gypsum in comparison to phosphogypsum.

The table 4.16 gives comparison of quantity of bitumen paint for different pavement models.

S. No.	Particulars of Materials and Details of works	Length (m)	Breadth (m)	Quantity	Unit	Rate	Per
1	**1st coat of bitumen paint**						
	Quantity of stone grit, 20 mm gauge						
	PM 1	0.4	0.3	0.0016	cu m	1.35	/ sq m
	PM 2	0.29	0.3	0.0011	cu m	1.35	/ sq m
	PM 3	0.29	0.3	0.0011	cu m	1.35	/ sq m
	Binder road tar no. 3 or asphalt						
	PM 1	0.4	0.3	0.264	kg	220	/ sq m
	PM 2	0.29	0.3	0.191	kg	220	/ sq m
	PM 3	0.29	0.3	0.191	kg	220	/ sq m
2	**2nd coat of bitumen paint**						
	Quantity of stone grit, 20 mm gauge						
	PM 1	0.4	0.3	0.0009	cu m	0.75	/ sq m
	PM 2	0.29	0.3	0.0006	cu m	0.75	/ sq m
	PM 3	0.29	0.3	0.0006	cu m	0.75	/ sq m
	Binder asphalt						
	PM 1	0.4	0.3	0.144	kg	120	/ sq m
	PM 2	0.29	0.3	0.104	kg	120	/ sq m
	PM 3	0.29	0.3	0.104	kg	120	/ sq m

Table 4.16 – Abstract of quantity of bitumen paint for different pavement models

As can be seen from table 4.16 the highest quantity of bitumen paint (stone grit and road tar) are used for pavement model 1 (PM 1). This is due to larger dimensions of shear box for PM 1. The quantity of bitumen paint is same for PM 2 and PM 3 due to similar dimensions of shear box.

It has been assumed that all other costs such as haulage cost, construction cost, etc. will be same for all three pavement models. It can be seen from the above analysis that the cost of pavement model 1 is least while the cost of pavement model 3 is maximum. Therefore, it can be concluded that by not using binder admixtures (fly ash-lime-gypsum/phosphogypsum), there is saving of cost. However, according to the results of compression testing it is better to use pavement model 2 due to (a) more taking of load and (b) 11.85 per cent savings in cost as compared to pavement model 3.

CHAPTER 5

CONCLUSIONS

5.1 Introduction

In an attempt to conserve scarce non-renewable natural resources in construction activities the search for new alternate materials is ongoing. In this major project report an attempt has been made to assess the suitability of fly ash-lime-gypsum/phosphogypsum mix for the road applications through comprehensive laboratory studies. This chapter presents an overview of the work carried out and the salient conclusions drawn from an application point of view and the conclusions drawn are highlighted.

5.2 Durability Characteristics

The quantities of these waste materials produced around the world are huge and causing disposal problems that are both financially and environmentally expensive. One method to reduce some portion of the fly ash and phosphogypsum disposal problem is by mixing them together in the presence of stabilizer like lime and utilizing the composite so produced for civil engineering applications. Civil engineers around the world are in search of new alternative materials which are required both for cost effective solution for roads and for conservation of scarce natural resources. Towards this end compaction, unconfined compressive strength, durability tests were conducted on fly ash-lime-gypsum/phosphogypsum. The studies were reported in Chapter 4. On the basis of the results and discussion presented in this chapter, the following is concluded.

1. The unconfined compressive strength of the fly ash-lime-gypsum/phosphogypsum composite increased with the increase in curing period. Specifically, the rate of strength gain is higher with gypsum as compared to phosphogypsum modifier.
2. The durability of the fly ash-lime-gypsum/phosphogypsum composite improved with the increase in curing period as reported with higher residual unconfined compressive strength (RUCS). Also, the greater RUCS is observed with gypsum as compared to phosphogypsum in all curing periods.

3. Phosphogypsum modifier is very economical to use for higher curing period because this being a waste and impure product will be highly utilized for pavement sub base course construction. Gypsum modifier is highly preferable for meeting the economy constraint as the results observed with gypsum are far better from phosphogypsum in all respects.
4. The empirical models presented are based on the experimental data within the range of parameter (curing period 7 to 28 days) and materials tested. Beyond this range of values the model may be checked with experimental results.

5.3 Pavement models (PMs) testing conclusions

1. The load carrying capacity of pavement model PM 1 is 2273.93 kg or 2.27 tones. The low strength of this model is due to absence of binder admixture (fly ash-lime-gypsum/phosphogypsum), wooden shear box, and improper compaction of subgrade and sub-base. This model is suitable to take load of bullock carts, bicycles, two wheelers (bikes, scooters) and three wheelers (auto-rickshaws). This model is suited for rural areas where the most common form of transport is bicycles and scooters.
2. The load carrying capacity of pavement model PM 2 is 9309.861kg or 9.3 tones. The very high strength of this model is provided by layer binder admixture (fly-ash-lime-gypsum) between subgrade and sub-base, steel shear box, and proper compaction of every layer. This model is suitable to take load of Light Commercial Vehicles (LCVs) that have weight upto 7.5 tones. This model can also be used for Utility vehicles (UVs), Muti-purpose vehicles (MPVs) that have seating capacity of 7 – 12 persons excluding the driver. This model is suitable for urban areas.
3. The load carrying capacity of pavement model PM 3 is 6115.62 kg or 6.1 tones. The strength of this model is intermediate between that of PM 1 and PM 2. The low strength of PM 3 in comparison to that of PM 2 is due to poor quality of phosphogypsum used in binder admixture (fly ash-lime-phosphogypsum). This model is suitable to take load of passenger cars – mini cars, compact cars, mid-size cars, etc. that have seating capacity of 4 – 6 persons excluding the driver. This model is suitable for semi-urban areas.
4. The value of modulus of subgrade reaction (k) is 137.65 x 10^4 kg/m^3 for weak subgrade to over 2753.19 x 10^4 kg/m^3 for strong subgrade. The highest value of k (k_{30} = 2005 x 10^4 kg/m^3, k_{75} = 802 x 10^4 kg/m^3) for PM 2 indicates that it has the strongest and properly

compacted subgrade and sub-base. The high strength is due to the presence of fly ash-lime-gypsum binder admixture layer between subgrade and sub-base. The fly ash-lime-gypsum provides better contact and high bond strength development between these two layers. Due to cheap quality of phosphogypsum, the value of k (k_{30} = 937 x 10^4 kg/m^3, k_{75} = 375 x 10^4 kg/m^3) is not high.

5. The typical values of subgrade modulus (k) range between 275.31 - 1101.27 kg/m^3 x 10^4 for coarse grained soils and 68.3 – 412.9 kg/m^3 x 10^4 for fine grained soils. This indicates that our soil or subgrade layer i.e. Kullu soil is coarse grained soil.
6. The suggested range of values of modulus of subgrade reaction (k) by AASHTO 1993 is shown in table 5.1.

Roadbed Soil Quality	**Range for k (kg/m^3) x 10^4**
Very Good	> 1515.2
Good	1102.2 - 1377.6
Fair	688.2 - 963.6
Poor	412.9
Very Poor	< 412.9

Table 5.1 – Range of values of k AASHTO 1993

The range of k_{30} values obtained by us is 937 – 1135 kg/m^3 x 10^4. This tells us that our soil is of good quality. The range of k_{75} values obtained by us is 375 – 802 kg/m^3 x 10^4. This tells us that our soil is of fair quality. The combined values of k_{30} and k_{75} tell us that our soil is of fair quality.

7. The values of modulus of elasticity range from 356.8 - 1529.5 kg/m^3 x 10^4 for silty soils and 356.8 - 1019.7 kg/m^3 x 10^4 for clay soils. This tells us that the subgrade of PM 1 (E = 251 kg/m^3 x 10^4) is predominantly clay soil, the subgrade of PM 2 (E = 2551.67 kg/m^3 x 10^4) is predominantly silty soil and the subgrade of PM 3 (E = 1192.37 kg/m^3 x 10^4) is mix of clay and silty soil.
8. The values of flakiness index, elongation index, crushing value, impact value, specific gravity and water absorption are within specified limits of IRC as can be seen in section 3.4.

5.4 Concluding Remarks

On the whole, the report has attempted to provide an insight into the various aspects of the suitability of fly ash-lime-gypsum/phosphogypsum mix through laboratory study and brought out its application in road pavements. The composite of fly ash-lime-gypsum/phosphogypsum satisfy the unconfined compressive strength as well as durability criteria at 28 days of curing. Therefore fly ash + 8 % lime + 1 % gypsum cured for 28 days is the optimum for use as a base/sub-base course in road pavements. As per IRC–15, 1973 the 7 days curing unconfined compressive strength (UCS) of Cement stabilized soils could be 1750 kPa to be used for base course construction. Whereas, our present studies with fly ash + 8 % lime + 1 % gypsum reports the 28 days curing UCS of 1892.33 kPa. Hence, this mixture is suitable for base course construction at higher curing period. However the postulated assessment needs to be supplemented subsequently with field trials.

5.5 Future Scope of Work

From the current study the suitability of fly ash-lime-gypsum/phosphogypsum has been demonstrated. However to enable field applications the following research is desirable.

1. Based on the values of k and E obtained, we can judge about the type and quality of soil available in an area. This makes the design of rigid pavement feasible.
2. The reduction in the thickness of various courses by using fly ash-lime-gypsum/phosphosypsum as compared to the conventional pavement design is another area of research.
3. The freeze-thaw effect study can also be conducted on fly ash-lime-gypsum/phosphogypsum binder admixtures.
4. The present studies are limited to UCS and RUCS comparisons for checking the suitability of waste and hazardous materials in road construction. However, the comparative studies of CBR & other pavement design parameters are highly desired.
5. Compaction test results have been utilised to decide the reference mix proportion to conduct the comparative studies. However, conducting similar kind of UCS and RUCS studies with varying contents of Lime and gypsum/phosphogypsum will be highly suitable for strong conclusion. Some other mix proportion may result into higher values of UCS and RUCS.

REFERENCES

- ASTM C 618 (2003): Standard specification for coal fly ash and raw or calcined natural pozzolan for use in concrete.
- Bhasin N.K, Krishan N, Mi P, Chandra J, Kumar A, “A Study on Use of Low Grade Materials for Road Construction in Rural Areas”: Indian Highways, May 1987,Vol 15,No.5 pp.35-48.
- Dempsey, B. J. and Thomson, M. R. (1973). “Effect of Freeze – Thaw parameters on the durability of stabilized materials.” *Transportation Engineering Series no 4, Illinois cooperative highway research program series no 136, Report Project IHR-401*, University of Illinois, Urbana, Illinois.
- DiGioia, A. M., and Nuzzo, W. L. (1972). “Fly ash as structural fill.” *J. Power Div.*, 98(1), 77–92.
- Franklin, J. A. and Chandra, R. (1972). ‘‘The slake durability test.’’ *International Journal Rock Mechanics Mineral Science*, 9(3), 325–341.
- Ghosh, A. (1996). ‘‘Environmental and engineering characteristics of stabilized lowlime fly ash.’’ PhD thesis, Indian Institute of Technology, Kharagpur, India.
- Ghosh, A. and Subbarao, C. (2001). “Microstructural development in fly ash modified with lime and gypsum.” *Journal of Materials in Civil Engineering*, 13(1), 65–70.
- Ghosh, A. and Subbarao, C. (2006). “Tensile strength bearing ratio and slake durability of class F fly ash stabilized with lime and gypsum.” *Journal of Materials in Civil Engineering*, 18(1), 18–27.
- Ghosh, R. K., Chadda, L. R., Pant, C. S., and Sharma, R. K. (1973). “Stabilization of alluvial soil with lime and fly ash.” Indian Road Congress, 35 (2), 489–511.
- Ghosh, A., Ghosh, A., and Bera, A. K. (2005). “Bearing capacity of square footing on pond ash reinforced with jute-geotextile.” *J. Geo-textiles and Geomembranes*, 23(2), 144–173.
- Ghosh A., Subbarao C. (2006) “Tensile Strength Bearing Ratio and Slake Durability of Class F Fly Ash Stabilized with Lime and Gypsum”, *Journal of materials in civil engineering*, Vol. 18 (1).

- Ghosh A. (2010), “Compaction Characteristics and Bearing Ratio of Pond Ash Stabilized with Lime and Phosphogypsum”, *Journal of Materials in Civil Engineering*, Vol. 22 (4).
- Husni Nuhan, “A Laboratory Study of Skid Resistance Characteristics of Pavement”, ME. Thesis, U.O.R. Roorkee, June 1987, pp.55-62.
- IRC-15 (1973) (First Reprint) (1978). Recommended Design Criteria for the Use of Cement-Modified Soil in Road Construction, The Indian Road Congress, Jamnagar House, Shahjahan Road, New Delhi – 110011.
- IS: 2386 Part I-1963, “Methods of Test for Aggregate for Concrete Part I, Particle Size and Shape”, Bureau of Indian Standards.
- IS: 4332 Part IV (1968), “Indian Standard code of Practice for Methods of test for stabilized soil (Part IV) Wetting and Drying and Freezing and Thawing tests for compacted soil cement mixtures” First Reprint April, 1984, Bureau of Indian Standards, New Delhi, India.
- IS: 4332(Part IV) (1968) (Re-affirmed on 1995): Method of test for stabilized soil, Wetting and drying, and freezing and drying of compacted soil –cement mixture.
- IS: 4332 Part V (1970), “Indian Standard code of Practice for Methods of test for stabilized soil (Part V) Determination of Unconfined Compressive Strength of Stabilized soil”, First Reprint July, 1986, Bureau of Indian Standards, New Delhi, India.
- IS: 4332(Part V) (1970) (Re-affirmed on 1995): Method of test for stabilized soil, Determination of unconfined compressive strength of stabilized soils.
- ISRM. (1981). ‘‘Suggested methods for rock characterization, testing, and monitoring.’’ ISRM Commission on Testing Methods, E. T. Brown, ed., Pergamon, Tarrytown, N.Y.
- Jha, J. N., Gill, K. S. and Choudhary, A. K. (2009). “Effect of high fraction class F fly ash on lime stabilization of soil.” *International Journal of Geotechnics and Environment*, 1(2), 105-128.
- Kaniraj, S.R. and Gayathri, V. (2004). “Permeability and consolidation characteristics of compacted Fly Ash.” *Journal of Energy Engineering*, 130(1), 18-43.
- Kim, S. S., and Chun, B. S. (1994). “The study on a practical use of wasted coal fly ash for coastal reclamation.” Proc., *13th Int. Conf.* on Soil Mechanics and Foundation Engineering, New Delhi, India, 1607–1612.

- Kumar, S. (2002). “A perspective study on fly ash–lime–gypsum bricks and hollow blocks for low cost housing development.” *Construction and Building Materials*, 16, 519–525.
- Kumar Vikas, “Study on Use of Shingle as Road Construction Material for Base Course”, M,E,Thesis, U.O.R. Roorkee, 1993, pp72-75.
- Lees G and Bindra S.P, “Some Aspects of Unbound Aggregates for Road Construction in India”: IRC 1981 - 1982, Vol. 42-2.
- Manjesh, L., Ramesh, N. H., Kumar Mohan, and Sivapullaiah, P. V. (2003). “CBR values of soil-fly ash mixture for road construction.” Proc., *Indian Geotechnical Conf.*, Roorkee, India, 451–454.
- Mishra, M. K. and Karnam, U.M. Rao (2006). “Geotechnical characterization of fly ash composites for backfilling mine voids.” *Geotechnical and Geological Engineering*, 24, 1749 -1765.
- Pandian, N. S. (2004). “Fly ash characterization with reference to geotechnical applications.” *J. Indian Inst. Sci.*, 184, 189–216.compacted soil cement mixtures” First Reprint April, 1984, Bureau of Indian Standards, New Delhi, India.
- Parreira, A. B., Kobayashi, A. R. K., and Silvestre, O. B., Jr. (2003). “Influence of Portland cement type on unconfined compressive strength and linear expansion of cement-stabilized phosphogypsum.” *J. Environ. Eng.*, 129 (10), 956–960.
- Raymond, S. (1958), “The utilization of pulverized fuel ash.” Civil Engineering and Public Works Review, London, 53, 1013–1016.
- Sharma HP, “All Weather Low Cost Roads for India”, M.E. Thesis, U.O.R. Roorkee, 1962, pp32-40.
- Shihata, S.A. and Baghbadi, Z.A. (2001). “Long term strength and durability of soil cement.” *Journal of Materials in Civil Engineering*, 13(3), 161-165.
- Singh, M. and Garg, M. (2007). “Durability of cementing binders based on fly ash and other wastes” *Construction and Building Materials*, 21, 2012–2016.
- Sivapullaiah, P.V. and Moghal, A. A. B. (2011). “Role of gypsum in the strength development of fly ashes with lime.” *Journal of Materials in Civil Engineering*, 23(2), 197-206.

- Vasquez, E., and Alonso, E. E. (1981). “Fly ash stabilization of decomposed granite.” Proc., *10th Int. Conf.* on Soil Mechanics and Foundations Engineering, Stockholm, Sweden, 391–395.

ANNEXURE 1

UNCONFINED COMPRESSIVE STERENGTH (UCS) and RESIDUAL UNCONFINED COMPRESSIVE STRENGTH (RUCS) READINGS FOR FLY ASH–LIME–GYPSUM BINDER

The Unconfined Compressive Strength (UCS) and Residual Unconfined Compressive Strength (RUCS) for fly ash-lime-gypsum binder are given in the following tables.

Vertical Deformation (ΔL)		**Vertical Strain (ε)**	**ε**	**Corrected Area**	**Compressive Load**		**UCS**
(div.)	**(mm)**	**ε = (ΔL/L)**	**(%)**	**A = Ao/(1 - ε) (mm^2)**	**(div.)**	**(kg)**	**(kPa)**
0	0	0	0	1134	0	0	0
50	0.5	0.006578947	0.657895	1141.509934	13	1.859	162.85447
100	1	0.013157895	1.315789	1149.12	20	2.86	248.8861
150	1.5	0.019736842	1.973684	1156.832215	29	4.147	358.47895
200	2	0.026315789	2.631579	1164.648649	33	4.719	405.18658
250	2.5	0.032894737	**3.289474**	1172.571429	40	5.72	**487.81676**

Vertical Deformation (ΔL)		**Vertical Strain (ε)**	**ε**	**Corrected Area**	**Compressive Load**		**RUCS**
(div.)	**(mm)**	**ε = (ΔL/L)**	**(%)**	**A = Ao/(1 - ε) (mm^2)**	**(div.)**	**(kg)**	**(kPa)**
0	0	0	0	1134	0	0	0
20	0.2	0.002631579	0.263158	1136.992084	15	2.145	188.65567
40	0.4	0.005263158	0.526316	1140	32	4.576	401.40351
60	0.6	0.007894737	0.789474	1143.023873	45	6.435	562.98037
80	0.8	0.010526316	1.052632	1146.06383	60	8.58	748.6494
100	1	0.013157895	1.315789	1149.12	72	10.296	895.98997
120	1.2	0.015789474	1.578947	1152.192513	85	12.155	1054.9452
140	1.4	0.018421053	1.842105	1155.281501	96	13.728	1188.2818
160	1.6	0.021052632	2.105263	1158.387097	110	15.73	1357.9226
180	1.8	0.023684211	2.368421	1161.509434	123	17.589	1514.3226
200	2	0.026315789	2.631579	1164.648649	133	19.019	1633.0247
220	2.2	0.028947368	2.894737	1167.804878	146	20.878	1787.7987
240	2.4	0.031578947	3.157895	1170.978261	155	22.165	1892.8618
260	2.6	0.034210526	3.421053	1174.168937	160	22.88	1948.6123
280	2.8	0.036842105	3.684211	1177.377049	170	24.31	2064.7591
300	3	0.039473684	**3.947368**	1180.60274	180	25.74	**2180.2423**

Readings of compression test (UCS and RUCS) for 7 days for Gypsum (G)

Vertical Deformation (ΔL)		Vertical Strain (ε)	ε	Corrected Area	Compressive Load		UCS
(div.)	(mm)	ε = (ΔL/L)	(%)	A = Ao/(1 - ε) (mm^2)	(div.)	(kg)	(kPa)
0	0	0	0	1134	0	0	0
20	0.2	0.002631579	0.263158	1136.992084	13	1.859	163.5016
40	0.4	0.005263158	0.526316	1140	20	2.86	250.8772
60	0.6	0.007894737	0.789474	1143.023873	29	4.147	362.8096
80	0.8	0.010526316	1.052632	1146.06383	33	4.719	411.7572
100	1	0.013157895	1.315789	1149.12	42	6.006	522.6608
120	1.2	0.015789474	1.578947	1152.192513	47	6.721	583.3227
140	1.4	0.018421053	1.842105	1155.281501	53	7.579	656.0306
160	1.6	0.021052632	2.105263	1158.387097	61	8.723	753.0298
180	1.8	0.023684211	**2.368421**	1161.509434	72	10.296	**886.4327**

Vertical Deformation (ΔL)		Vertical Strain (ε)	ε	Corrected Area	Compressive Load		RUCS
(div.)	(mm)	ε = (ΔL/L)	(%)	A = Ao/(1 - ε) (mm^2)	(div.)	(kg)	(kPa)
0	0	0	0	1134	0	0	0
10	0.1	0.001315789	0.131579	1135.494071	6	0.858	75.56182
20	0.2	0.002631579	0.263158	1136.992084	16	2.288	201.2327
30	0.3	0.003947368	0.394737	1138.494055	29	4.147	364.2531
40	0.4	0.005263158	0.526316	1140	41	5.863	514.2982
50	0.5	0.006578947	0.657895	1141.509934	55	7.865	688.9997
60	0.6	0.007894737	0.789474	1143.023873	67	9.581	838.2152
70	0.7	0.009210526	0.921053	1144.541833	78	11.154	974.5384
80	0.8	0.010526316	1.052632	1146.06383	89	12.727	1110.497
90	0.9	0.011842105	1.184211	1147.58988	97	13.871	1208.707
100	1	0.013157895	1.315789	1149.12	105	15.015	1306.652
110	1.1	0.014473684	1.447368	1150.654206	117	16.731	1454.042
120	1.2	0.015789474	1.578947	1152.192513	126	18.018	1563.801
130	1.3	0.017105263	1.710526	1153.73494	135	19.305	1673.261
140	1.4	0.018421053	1.842105	1155.281501	146	20.878	1807.179
150	1.5	0.019736842	1.973684	1156.832215	155	22.165	1916.008
160	1.6	0.021052632	2.105263	1158.387097	168	24.024	2073.918
170	1.7	0.022368421	2.236842	1159.946164	176	25.168	2169.756
180	1.8	0.023684211	2.368421	1161.509434	180	25.74	2216.082
190	1.9	0.025	2.5	1163.076923	187	26.741	2299.16
200	2	0.026315789	**2.631579**	1164.648649	193	27.599	**2369.728**

Readings of compression test (UCS and RUCS) for 14 days for Gypsum (G)

Vertical Deformation (ΔL)		Vertical Strain (ε)	ε	Corrected Area	Compressive Load		UCS
(div.)	(mm)	$\varepsilon = (\Delta L/L)$	(%)	$A = Ao/(1 - \varepsilon)$ (mm^2)	(div.)	(kg)	(kPa)
0	0	0	0	1134	0	0	0
10	0.1	0.001315789	0.1315789	1135.494071	6	0.858	75.56182
20	0.2	0.002631579	0.2631579	1136.992084	13	1.859	163.5016
30	0.3	0.003947368	0.3947368	1138.494055	25	3.575	314.0113
40	0.4	0.005263158	0.5263158	1140	36	5.148	451.5789
50	0.5	0.006578947	0.6578947	1141.509934	50	7.15	626.3634
60	0.6	0.007894737	0.7894737	1143.023873	67	9.581	838.2152
70	0.7	0.009210526	0.9210526	1144.541833	80	11.44	999.5266
80	0.8	0.010526316	1.0526316	1146.06383	87	12.441	1085.542
90	0.9	0.011842105	1.1842105	1147.58988	96	13.728	1196.246
100	1	0.013157895	1.3157895	1149.12	110	15.73	1368.874
110	1.1	0.014473684	1.4473684	1150.654206	119	17.017	1478.898
120	1.2	0.015789474	1.5789474	1152.192513	132	18.876	1638.268
130	1.3	0.017105263	1.7105263	1153.73494	144	20.592	1784.812
140	1.4	0.018421053	1.8421053	1155.281501	150	21.45	1856.69
150	1.5	0.019736842	**1.9736842**	1156.832215	153	21.879	**1891.286**

Vertical Deformation (ΔL)		Vertical Strain (ε)	ε	Corrected Area	Compressive Load		RUCS
(div.)	(mm)	$\varepsilon = (\Delta L/L)$	(%)	$A = Ao/(1 - \varepsilon)$ (mm^2)	(div.)	(kg)	(kPa)
0	0	0	0	1134	0	0	0
5	0.05	0.000657895	0.0657895	1134.746544	3	0.429	37.8058
10	0.1	0.001315789	0.1315789	1135.494071	7	1.001	88.15546
15	0.15	0.001973684	0.1973684	1136.242584	11	1.573	138.4387
20	0.2	0.002631579	0.2631579	1136.992084	25	3.575	314.4261
25	0.25	0.003289474	0.3289474	1137.742574	40	5.72	502.7499
30	0.3	0.003947368	0.3947368	1138.494055	50	7.15	628.0226
35	0.35	0.004605263	0.4605263	1139.24653	65	9.295	815.8901
40	0.4	0.005263158	0.5263158	1140	78	11.154	978.4211
45	0.45	0.005921053	0.5921053	1140.754467	90	12.87	1128.201
50	0.5	0.006578947	0.6578947	1141.509934	97	13.871	1215.145
55	0.55	0.007236842	0.7236842	1142.266402	111	15.873	1389.606
60	0.6	0.007894737	0.7894737	1143.023873	125	17.875	1563.834
65	0.65	0.008552632	0.8552632	1143.782349	134	19.162	1675.319
70	0.7	0.009210526	0.9210526	1144.541833	145	20.735	1811.642
75	0.75	0.009868421	0.9868421	1145.302326	150	21.45	1872.868
80	0.8	0.010526316	1.0526316	1146.06383	162	23.166	2021.353
85	0.85	0.011184211	1.1184211	1146.826347	175	25.025	2182.109
90	0.9	0.011842105	1.1842105	1147.58988	182	26.026	2267.883
95	0.95	0.0125	1.25	1148.35443	190	27.17	2365.994
100	1	0.013157895	**1.3157895**	1149.12	195	27.885	**2426.64**

Readings of compression test (UCS and RUCS) for 28 days for Gypsum (G)

ANNEXURE 2

ADDITIONAL PHOTOGRAPHS

Gypsum (before being finely crushed)

Fly ash – Class F

Lime

Fly Ash + Lime (8%) + Gypsum (1%) mix

Fly Ash + Lime (8%) + Phosphogypsum (2%) mix

Sample Extractor