STRENGTH AND RESILIENT CHARACTERISTICS OF POND ASH AS SUBBASE LAYER IN FLEXIBLE PAVEMENTS

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by

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CERTIFICATE

This is to certify that the thesis entitled **"STRENGTH AND RESILIENT CHARACTERISTICS OF POND ASH AS SUBBASE LAYER IN FLEXIBLE PAVEMENTS"** being submitted by **Mr. M. SUDHAKAR** for the award of the degree of **DOCTOR OF PHILOSOPHY** to the Faculty of Engineering and Technology of **NATIONAL INSTITUTE OF TECHNOLOGY, WARANGAL** is a record of bonafide research work carried out by him under my supervision and, it has not been submitted elsewhere for the award of any degree.

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This Thesis entitled "**Strength and Resilient Characteristics of Pond Ash as Subbase Layer in Flexible Pavements**" by **Mr. M. Sudhakar** is approved for the degree of Doctor of Philosophy.

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ABSTRACT

The pavement construction sector throughout the world is facing a significant challenge of non-availability of suitable materials. On the other side, many waste/ by-products are being generated from manufacturing/production units causing environmental pollution. One such waste material is coal pond ash produced from thermal power plants, which can be utilized as an alternative material in place of traditional construction materials. The use of pond ash in pavement construction would also lead to eco-friendly and profitable utilization; otherwise, it would discard as a waste product.

The design methods of flexible pavements based on strength characteristics such as unconfined compressive strength (UCS), and California bearing ratio (CBR) are mostly empirical and does not reflect any mechanistic similarity with how the pavement layers experience the various vehicle loads in the field. Even such empirical design methods are primarily confined to a range of classical pavement materials. Many previous studies have shown the critical consideration of resilient characteristics with the strength aspects, especially when alternative industrial waste materials utilized in the construction. As per AASHTO and NCHRP specifications, the empirical methods of design such as shear failure method, limiting deflection method, and regression-based method have also given way to restrict its use and finally landing up on Mechanistic-Empirical Pavement Design (M-EPD) methods; which relies mainly on resilient characteristics such as resilient modulus (M_R) and plastic strain deformation (ϵ_p).

Therefore, in the present context, investigations are carried out to understand the engineering behaviour of pond ash (P) to ascertain its feasibility as a sub-base material. The study looked at strength characteristics (UCS and CBR) and resilient characteristics (M_R and ϵ_p) of pond ash. Pond ash used in this study is collected from Kakatiya Thermal Power Plant (KTPP)-Telangana, India; and it is categorized as class F.

In general, the use of class F-based coal ashes (Fly ash, Bottom ash, Pond ash) alone could not manifest desirable strength behaviour in civil engineering works, and it needs to be modified with suitable additives to improve its overall engineering performance in the long run. Hence, in this study, two additives namely lime (L, in 4%, 6%, 8%, 10% and 12%) as a cementitious stabilizer, and randomly distributed polypropylene fibers (F, in 0.5%, 1%, 1.5%, and 2.0%) as reinforcement inclusion on dry weight basis were added as an individual and in

combined proportions to improve mechanical strength behaviour of pond ash. The physical and engineering properties of materials were determined in the laboratory in a controlled environment. Standard proctor compaction tests were performed on the mixtures to determine optimum moisture content – maximum dry unit weight (OMC-MDD). The effect of variables such as lime content, fiber content and curing period on above-mentioned properties of pond ash used were investigated. As the first research objective, an attempt has been made to evaluate the potential of additives to improve strength properties of pond ash like UCS (at a curing period of 7, 28, 56, 90 days) and CBR (at a curing period of 7 and 28days) in both untreated (P) and treated (PL, PF and PLF) forms. The effect of proposed additive contents in altering compaction and strength characteristics and morphological changes in pond ash were examined. The test results were then compared with standard IRC specifications. From the results, the strength properties of pond ash were improved considerably with an addition of lime, fiber and both (L and F). Significant improvement in the strength properties is observed with an addition of lime of 6% to 8%. The inclusion of fiber reinforcement in pond ash demonstrated the rate of enhancement in CBR values with considerable rate up to 1% fiber; however, its values are still lower than CBR required for subbase applications. The combined effect of both lime and fiber improved the performance of pond ash furthermore with enhanced load-bearing capacity, satisfied the CBR criteria as per IRC specifications.

The second objective of the study was focused on evaluating the resilient modulus characteristics of pond ash and examined the influence of additives (L and F and both in combination) on it. Based on the test results of the previous study, the following research was focused on investigating the stiffness (resilient modulus, M_R) characteristics of pond ash by conducting repeated load triaxial (RLT) tests on treated samples prepared at considered individual optimum contents (i.e. lime, L_8 and fiber, F_1) and other combined mix proportions based on optimum contents (i.e. PL_8F_X, PL_XF_1). The influence of variation in additive contents, deviatoric and confining stresses on the repeated loading behaviour of pond ash was examined. The test results were performed for statistical regression analysis by considering four well-known regression models (two-parameter based: Bulk model and Power model; three-parameter based: Universal model and octahedral shear model) reported in the literature. These studies showed that the experimental data was found a good fit with three-parameter based models. Further, with the obtained results of UCS, CBR and M_R, the correlation equations were also developed.

The third objective was to examine the permanent deformation (ϵ_p) behaviour of both untreated and treated pond ash by subjecting the specimens to a number of loading cycles (10000 N) in RLT apparatus. Effect of additives in various mix combinations (same as M_R) on the deformation behaviour of pond ash was analyzed. The influence of confining and deviator stress levels on permanent deformation behaviour is examined. Based on the experimental investigations, it was observed that compared to untreated pond ash, ϵ_p of treated pond ash was reduced by 57% in PL_8, and 43% in PF_1 specimens. Modification of pond ash with both additives further decreased ϵ_p by 65% at both optimum (PL_8F_1), which indicates the increase in the life span of the pavement structure. Four regression models (two-parameter based models: Logarithmic and Power, and three-parameter based models: Universal and Octahedral shear) reported in the literature are considered to validate the experimental data, found that fitting of 3 parameter-based models in an effective way with higher regression coefficient values.

Apart from the improved behaviour of pond ash with the addition of lime and fiber, except CBR values, the obtained results of UCS, as well as M_R values, mostly lie in the minimum requirement range; which indicating their low rate of applicability for high volume roads, but the same can be effectively used for low-volume flexible roads. In this regard, as the fourth objective, based on IRC-72, using software like KENLAYER the optimized thickness was calculated based on vertical compressive strain limitation. Further, the economic assessment studies were carried out with the proposed optimum PLF mix and showed a reduction in thickness with a saving in cost of around 2% - 5% of total costs of 1 Kilometre road (with a single lane) for the same service life compared to conventional subbase layer pavement.

Keywords: Pavements, Pond ash, Lime, Fiber, UCS, CBR, Resilient modulus, Permanent deformation

CONTENTS

CERTIFICATE .... **i**

DECLARATION .... **ii**

APPROVAL SHEET .... **iii**

ACKNOWLEDGEMENTS .... **iv**

ABSTRACT .... **vi**

CONTENTS .... **ix**

LIST OF TABLES .... **xv**

LIST OF FIGURES .... **xvi**

ABBREVIATIONS .... **xviii**

NOTATIONS .... **xix**

CHAPTER – 1 .... **1**

INTRODUCTION **1**

1.1 General 1

1.2 Pavement Construction Programs in India 1

1.3 Base/Subbase Layer in Flexible Pavements 1

1.4 Coal ash 2

1.5 Modified Coal Ash in Geotechnical Applications 3

1.6 Pavement Structural Design 4

1.7 Motivation for the Study 5

1.8 Research Goal 6

1.9 Objectives 6

1.10 Organization of Thesis 6

CHAPTER – 2 .... **8**

LITERATURE REVIEW **8**

2.1 General 8
2.2 Material Characterization for Flexible Pavement Design 9
2.3 Environmental Consideration on Using Traditional Materials 9
2.4 Secondary Materials 10
2.5 Soil Stabilization Concept 10
2.5.1 Chemical Stabilisation 11
2.5.2 Mechanical Stabilisation 11
2.6 Origin of Coal Pond ash 11
2.7 Existing Use of Coal ash 12
2.8 Coal ash as Partial Replacement Material 12
2.9 Studies Related to Coal ash 15
2.10 Modified Moal ash in Pavement 15
2.10.1 Cementitious Modification 16
2.10.2 Reinforcement Inclusion 17
2.10.3 Combined Additive Treatments 19
2.11 Determination of Resilient Modulus 20
2.12 Studies on Correlation between M_R and Other parameters 25
2.13 Numerical Models for Predicting Resilient Modulus 26
2.14 Permanent Deformation of Pavement Material 28
2.15 Existing Permanent Deformation Models 31
2.16 Summary of Literature 34
CHAPTER – 3 37
MATERIALS AND METHODOLOGY 37
3.1 General 37
3.2 Experimental Procedures 37

3.2.1 Specific Gravity 37

3.2.2 Particle Size Distribution 37

3.2.3 X-ray Fluorescence Spectrometer 37

3.2.4 Compaction 38

3.2.5 Unconfined Compressive Strength (UCS) 38

3.2.6 California Bearing Ratio (CBR) 38

3.2.7 X-ray Diffraction (XRD) 39

3.2.8 Scanning Electron Microscopy (SEM) 39

3.2.9 Repeated Load Triaxial (RLT) 39

3.3 Materials 43

3.3.1 Pond ash (P) 43

3.3.2 Lime (L) 45

3.3.3 Polypropylene Fiber (F) 46

3.4 Additive Contents and Specimen Preparation 47

3.5 Methodology 48

3.6 Summary 49

CHAPTER – 4 50

COMPACTION AND STRENGTH CHARACTERISTICS OF COAL ASH 50

4.1 General 50

4.2 Compaction behaviour 50

4.2.1 Effect of Lime on Compaction Parameters 50

4.2.2 Effect of Fiber Inclusion on Compaction Behaviour 51

4.3 Effect of Lime on UCS of Pond ash 52

4.4 California Bearing Ratio 55

4.4.1 Effect of Lime on CBR of Pond ash 55

4.4.2 Effect of Fiber on CBR of Pond ash 56
4.5 Effect of Lime and Fiber on Compaction Behaviour of Pond ash 58
4.6 Effect of Lime and Fiber on UCS of Pond ash 59
4.7 Combined Effect of Lime and Fiber on CBR of Pond ash 62
4.8 Summary 63
CHAPTER – 5 64
RESILIENT MODULUS CHARACTERISTICS 64
5.1 General 64
5.2 Specimen Preparation for RLT test 64
5.2.1 Untreated Specimen 64
5.2.2 Treated Specimen 64
5.3 RLT Test Results for M_R 65
5.3.1 Effect of Additives on M_R 65
5.4 Effect of Applied Stresses on M_R 69
5.5 Modelling Studies on M_R Response 71
5.5.1 Model 1 (M1): Bulk Stress Model (Uzan 1985) 72
5.5.2 Model 2 (M2): Power Model, (Witczak and Uzan 1988) 72
5.5.3 Model 3 (M3): Pezo Model, (Pezo 1993) 73
5.5.4 Model 4 (M4): Octahedral Shear Stress Model, (Witczak and Uzan 1988) 74
5.6 Correlation between UCS, CBR and M_R 75
5.6.1 Correlation between *CBR* and UCS 75
5.6.2 Correlation between M_R and UCS 76
5.6.3 Correlation between M_R and CBR 77
5.6.4 Correlation between M_R, UCS and CBR 78
5.7 Summary 79

CHAPTER – 6 ..... **80**

PERMANENT DEFORMATION CHARACTERISTICS **80**

6.1 General 80

6.2 Repeated Load Triaxial Test 80

6.2.1 Effect of Additives on ϵ_p Behaviour 80

6.2.2 Effect of Applied Stresses on ϵ_p 81

6.3. Modelling Studies on ϵ_p Response 82

6.3.1 Model 1 (M1): Logarithmic Model, (Barksdale 1972) 83

6.3.2 Model 2 (M2): Power Law model, Monismith et al. (1975) 83

6.3.3 Model 3 (M3): A Three-Parameter Model, (Ullditz 1993) 84

6.3.4 Model 4 (M4): A Three-Parameter Model, (Puppala et al. 1999) 85

6.4 Summary 86

CHAPTER – 7 ..... **87**

COST ESTIMATION **87**

7.1 General 87

7.2 Recommended Design Nalues for Pavement Subbase 87

7.3 Analysis for Minimum Required Thickness of Pavement 87

7.4 Economic Assessment 91

CHAPTER – 8 ..... **93**

CONCLUSIONS **93**

8.1 Brief Conclusions from Each Phase 93

8.2 Phase-I 93

8.3 Phase-II 94

8.4 Phase-III 95

8.5 Conclusions from project work: 95

8.6 Future Scope of the Investigation 96

REFERENCES **98**

Publications Related to the Work **113**

LIST OF TABLES

Table No.	Description	Page No.
3.1	Loading sequence used in the present study for MR (AASTHO T-307)	41
3.2.	Physical and chemical properties of pond ash	44
3.3.	Properties of Polypropylene fiber	47
4.1.	CBR values of lime treated pond ash	55
4.2.	Soaked CBR values of pond ash-lime mixes	57
4.3.	CBR of pond ash-lime-fiber mixture under soaked condition	62
5.1.	Regression analysis for bulk stress model	72
5.2.	Regression analysis for power model	73
5.3.	Regression analysis for pezo model	73
5.4.	Regression analysis octahedral shear stress model 1	74
6.2.	Regression analysis model constants of permanent deformation for Model 2	84
6.3.	Regression analysis model constants of permanent deformation for Model 3	84
6.4.	Regression analysis model constants of permanent deformation for Model 4	85
7.1.	Considered Pavement sections for design analysis (as per IRC 72-2015)	88
7.2.	Properties of materials considered in the study	88
7.3.	Optimised thicknesses of T6 Pavement section	89
7.4.	Optimised thicknesses of T7 Pavement section	89
7.5.	Optimised thicknesses of T8 Pavement section	90
7.6.	Optimized thicknesses of T9 Pavement section	90
7.7.	Cost per unit	91
7.8.	Cost analysis of subbase course	92

LIST OF FIGURES

Figure No.	Description	Page No.
3.1	Repeated load triaxial test setup	40
3.2	Pond ash used in the study	43
3.3	Particle size distribution of pond ash	44
3.4	XRD pattern of pond ash	45
3.5	SEM Photography of pond ash	45
3.6	Lime used in the study	46
3.7	SEM Photograph of Lime	46
3.8	Polypropylene fiber	47
3.9	Experimental Methodology	49
4.1	Compaction results of lime treated pond ash	51
4.2	Compaction behaviour of pond ash-fiber mixes	51
4.3	Variation in UCS of lime treated pond ash at different curing periods	53
4.4	SEM images of pond ash with different lime contents at 28 days curing	53
4.5	SEM images of pond ash with 8% lime content at various curing days	54
4.6	Stress-strain behaviour of lime treated-pond ash (28 days)	54
4.7	UCS sample during the test	55
4.8	CBR test of pond ash specimens	56
4.9	CBR load-penetration curves for pond ash-fiber specimens	57
4.1	Compaction results of PLF specimens at various proportions	59
4.11	UCS results of pond ash at various mix proportions	60
4.12	SEM images of PL8%F1.0% mix at 28 days curing	60
4.13	Stress-strain behaviour of lime-fiber treated pond ash (28 days)	61
4.14	Failure specimens of PLF mixes	61
5.1	Setup used in RLT test	65
5.2a	MR of untreated pond ash at various σ_c and σ_d stress levels	66
5.2b	MR of lime treated pond ash at various σ_c and σ_d stress levels	66

Figure No.	Description	Page No.
5.2c	M_R of fiber-reinforced pond ash at various σ_c and σ_d stress levels	67
5.3 (a-h)	M_R of lime-fiber treated pond ash at various σ_c and σ_d stress levels (28 days)	68
5.4.	M_R values of pond ash-lime-fiber mixes (MPa)	69
5.5	M_R of PLF mixtures for various σ_d stress levels at σ_c of 34.4 kPa	70
5.6	M_R of PLF mixtures for various σ_c stress levels at σ_d of 103.4 kPa	71
5.7	Correlation between UCS and CBR	76
5.8	Correlation b/w UCS and MR)	77
5.9	Correlation b/w M_R and CBR	78
5.1	Correlation between M_R, UCS and CBR	78
6.1	Permanent strain (ϵ_p) of untreated/ lime and fiber treated pond ash	82
6.2	Permanent deformation (ϵ_p) behaviour of PLF compositions	82
7.1	Layer thicknesses of various pavement sections as per IRC 72	88
7.2	Schematic view of input details considered in KENPAVE analysis	89

ABBREVIATIONS

AASHTO	:	American Association of State Highway and Transportation Officials
MEPDG	:	Mechanistic-Empirical Pavement Design Guide
NCHRP	:	National Cooperative Highway Research Program
RLT	:	Repeated loaded triaxial
UCS	:	Unconfined Compressive Strength
CBR	:	California bearing ratio
MDD	:	Maximun Dry Density
OMC	:	Optimum Moisture Content

NOTATIONS

M_R	:	Resilient modulus
σ_d	:	Deviatoric stress
σ_c	:	Confining stress
P	:	Pond ash
F	:	Fibre
L	:	Lime
ϵ_p	:	Permanent deformation
θ	:	Bulk stress
τ_{oct}	:	Octahedral shear stress
σ_{oct}	:	Octahedral normal stress
P_{atm}	:	Atmosperic Pressure

CHAPTER – 1

INTRODUCTION

1.1 General

This chapter presents the general information of the study, motivation for the work, research goal, objectives, and describes the organization of this thesis in the final section.

1.2 Pavement Construction Programs in India

Due to the vast geographical spread of the country and varied conditions of topography, road transportation has become an essential mode of transport in India. India consists of 58, 97,671 kilometres road network, and is the second-largest road network globally (MoRTH 2018). The Indian government is giving top priority to improve road transportation facilities countrywide by allocating enormous capital investments. The National Highway Development Project (NHDP) and Pradhan Mantri (Prime Minister's) Gram (Village) Sadak (Road) Yojana (PMGSY) programs have been proposed for implementation of the road network. However, in recent times, the most reflective challenging problem in this domain is the scarcity of conventional materials such as natural sand, crushed aggregates, gravel, bitumen due to the depletion of natural aggregate resources and the widespread demand for pavement materials in the construction fields.

1.3 Base/Subbase Layer in Flexible Pavements

A flexible pavement system generally consists of an asphalt surface layer, a base course layer, subbase layer and the subgrade. The base/subbase layer employed/placed between the surface and subgrade layers plays a significant role in transferring the loads from the surface layer to the natural soil subgrade (Kaniraj and Gayathri 2003; Patel and Shahu 2016, 2018). Thus, to distribute the traffic load, base/subbase courses must have enough strength to carry/transfer the loads without shear failure. To this, traditionally available materials derived from various source rocks have been used as a road base/subbase material (Lav and Lav 2006; Sahu et al. 2017). Environmental concerns constrain the extraction of these natural-conventional materials for road constructions; hence, many research studies have suggested replacing alternative sustainable materials as a viable option (Arshad and Ahmed 2017).

1.4 Coal ash

In the meanwhile of rapid development through industrialization and modernization many mining, manufacturing and thermal industries have been producing massive solid wastes in the form of wastes/by-products namely recycled concrete aggregates, reclaimed asphalt pavement, fly ash, bottom ash, waste rubber, waste plastics etc., and facing a shortage of disposal areas (Aboutalebi 2020; Zhao et al. 2020). Open dumping of these wastes resulted in public health and ecology threat. Hence, bulk utilization of these by-products/wastes in road construction will not only protect fast depleting natural aggregates but also preserve valuable land from huge waste dumps (MoEFCC 2015, 2016). In addition, some industrial wastes generally possess desirable engineering properties that can facilitate their use in road works (Lav and Lav 2014, Jamshidi et al. 2017; Kuntikana and Singh 2017). Today in India, only a few working sites use such alternative solid waste/recycled materials in a small amount due to lack of awareness of their engineering behaviour; and most worksites still rely on conventional materials.

One such industrial waste is coal combustion residue (CCR), generated in solid form as coal fly ash and bottom ash from coal-burned thermal power stations (Athanasopoulou, 2014). In general, the disposal of these CCR materials is done by mixing it with water in suitable proportion (1 part solid: 6 parts water) into wet-ponds/landfills, and the mixture is referred as pond ash (CEA 2017). In India, the production rate of CCR’s is about 200 million metric tons per year, consuming an estimated land of 250 million Sq. Meters for dumping purpose only (Patel et al. 2019; Arora and Kumar 2019). At present, only a small portion of coal pond ash is being utilized in various applications like foundations, road embankments, structural fill, land reclamation (Sridharan and Prakashan 2007; Ghosh 2009; Xu and Shi 2018). The remaining unused ash part is being dumped into wet-ash ponds as mountains of ashes.

While the research studies found the successful implementation of coal ash as an alternative resource material (partially/fully) in civil constructions, the coal ash status has been changed from “industrial waste” to “useful and saleable commodity”. Therefore, many Departments of Transportation (DOTs) worldwide (Texas DOT, Florida DOT, Illinois DOT, FHWA) have introduced programs to encourage the use of coal fly ash materials in pavement construction. Developing countries like India are also making great strides towards using alternate materials as raw materials in pavements. The programs like PMGSY in India proposed using new materials and technologies in pavements as part of its suggestions for rural roads

(Puppala et al. 2011; Gautam et al. 2018; Patel et al. 2019). The Ministry of Environment Forests and Climate Change (MoEF&CC) and Central Electrical Authorities (CEA) have also insisted field engineers with vide notifications from 1999 to 2003, 2009, and 2016 with the primary objective of utilizing coal ash in pavements and other civil engineering constructions to the maximum extent. In today's scenario, it has also become mandatory to use coal fly ash in road constructions and flyover embankments within a radius of 300 km of a thermal power plant. Besides, the Government of India is encouraging research studies that aim of 100% ash utilization in place of conventional materials in a sustainable basis (CEA 2017).

In India, most thermal power plants generate Class-F ashes; its use alone cannot attain desirable engineering behaviour in pavement constructions (Sivapulliah et al. 2000; Sridharan and Prakaesh et al. 2007; Bera et al. 2009; Moghal 2017; Bakare et al. 2018). These class-F ashes generally exhibit low to moderate pozzolanic characteristics due to their self-cementitious behaviour which could be improved further by adding suitable additives (Sivapulliah and Moghal 2011; Singh and Saran 2014; Samanth 2018).

Hence, maximizing the utilization of coal ash materials in pavement construction is effective and important for attaining sustainability in the construction (MoEFCC 2016). Also, achieving a realistic, stable, accurate, and cost-effective approach for assessing the performance of coal ash is yet another challenge that the road sector faces.

1.5 Modified Coal Ash in Geotechnical Applications

In the past studies, researchers have investigated the use of coal ash (fly ash, bottom ash and pond ash) as a partial substitute material (with or without adding admixtures) for the soil in various geotechnical applications (backfill material, landfill liner, land reclamation, ground improvement; stabilization, foundation base) and reported improved mechanical behaviour in terms of strength, stiffness, and failure behaviour characteristics (Arora and Ayeilek 2005; Kim et al. 2005; Consoli et al. 2009, 2010, 2017; Yadav et al. 2018). Research investigations (Chand and Subbarao 2007; Sivapulliah and Moghal 2011; Pani and Singh 2017; Suthar and Aggarwal 2018) have confirmed that coal fly ash treated with various cementitious additive blends viz., cement, lime, gypsum, GGBS, silica fume, Cement Kiln Dust (CKD), Lime Kiln Dust (LKD) showed an enhanced strength and durability characteristics due to development of pozzolanic reaction products (C-S-H, C-A-S-H). Nevertheless, in most of the cases, usage of these cemented agents with coal ash induces brittle behaviour even at low failure strains levels; which would affect the long-term stability and serviceability of pavement structure (Kaniraj and Gayathri 2003, 2006; Tang et al. 2010; Ghadakpour et al. 2019).

Researchers have also examined the effect of fibers inclusions (either natural or synthetic) in the discrete form of as reinforcement in soil, coal ash and soil-coal ash mixtures, reported the enhanced strength properties such as UCS, CBR, and shear strength, altered failure behaviour from brittle to ductile as well as reduced the post-peak losses (Kumar and Singh 2008; Bera et al. 2009; Chore et al. 2011; Tiwari et al. 2013; Dhar et al. 2018; Arora and Kumar 2019). This is because fibres inclusion in the composite material can withstand the tensile forces developed due to external loads and enhance the resistance by generating frictional forces between composite particles and reinforcement (Koerner 2012). The primary advantages of randomly oriented fiber reinforcement are i) simple in adding and mixing with soils (ii) To control the potential plane of weakness parallel to the plane of reinforcement (iii) To maintain strength isotropy in the composite mixture (iv) To modify the physical properties of virgin soil that promotes no impact on the environment (Tang et al. 2010; Li et al. 2014).

Also, various researches (Gupta and Kumar 2015; and Sahu et al. 2017; Dhar and Hussain 2018; Ghadakpour et al. 2019) have investigated the combined effect of both additives (i.e., chemical and reinforcement) on soils and other pavement related materials. They reported the enhanced peak axial stress, stiffness at large stain level, and modulus of elasticity, and reduced post-peak strength losses compared to their individual treatment. These enhanced properties of materials can reduce the required thickness of pavement layers and pose relatively low-cost alternative solutions in traditional pavement constructions where no conventional materials are available near the site.

1.6 Pavement Structural Design

Strength properties such as UCS and CBR are commonly used to characterize the pavement material for the structural design of pavements (Lav and Lav 2014). During the evaluation of these properties under laboratory conditions, the load applied to the specimen represents the static uniaxial loading condition. Researchers (Puppala et al. 1999, 2009, 2011; Arulrajan et al. 2013) have found that these strength-related parameters can only guide material selection and cannot simulate the actual mechanistic (traffic loading) behaviour. The design of pavements based on these parameters could exclusively be restricted to the range of traditional pavement materials (Lav and Himli 2014).

Therefore, for the design of coal ash-based pavements, mechanistic-empirical (M-E) design guide methods such as AASHTO (2000) and NCHRP (2004) are developed; which suggested the use of resilient modulus (M_R) as a fundamental property (i.e., stiffness) in characterizing pavements in their structural analysis and design. In general, resilient modulus (M_R) represents the mechanical response in terms of stresses, strains, and deflections caused by wheel loads. Also, permanent deformation (ϵ_p) is an essential factor in design analysis (with a specific allowable limit) to assess long-term behaviour and failure of the pavement structure (Arulrajah et al. 2013, Patel et al. 2019). The accumulation of deformations depends mainly on the effect of stress state and load repetitions by wheel movements that act on pavement structure, and the evaluation of ϵ_p can be assessed through repeated load triaxial (RLT) test.

Researchers (Kumar et al. 2006; Edil et al. 2006; Kumar and Singh 2008; Coenen et al. 2012; Lopes et al. 2012; Samar et al. 2020) have reported the improved mechanical (resilient and deformation) behaviour under repeated dynamic loading due to strain-hardening nature of additive-treated soils compared to the untreated one. Studies (Puppala et al. 2011; Arulrajah et al. 2013 and Rout et al. 2012) have shown improved M_R values and permanent strain resistance of RAP, RCA, unbound aggregates, and stabilized materials with an increase of deviatoric stress and confining stresses. Researchers (Saghafi et al. 2012; Dev and Robinson 2019; Lav and Himli 2014; Patel et al. 2019) conducted tests on cemented coal ashes and reported the improved M_R and deformation characteristic values compared to untreated materials under various stress conditions. Many research studies (Kumar and Singh 2008; Puppala et al. 2011; Dev and Robinson 2015; Patel et al. 2016, 2019) have also reported the mathematical models developed to estimate/validate the resilient characteristics of pavement materials.

1.7 Motivation for the Study

The rapid modernization and ever-increasing demand for raw materials in pavement construction are forcing to call for alternate materials through an industrial wastes/by-products to construct road pavements.

- Among all the waste materials generated, coal ash has better potential to be used as a pavement material. However, most of the coal ashes produced in India belong to class F and are not suitable for direct usage in pavement applications due to their low strength properties. Therefore, it must be modified with appropriate additives to achieve significant strength and adequate structural performance.

- The subbase layer in flexible pavements generally has a maximum thickness of all layers, provides uniform support action to the upper layers, ensures drainage, and minimizes the detrimental frost action. While several wastes/recycled materials were used more frequently during last two decades in pavement constructions as a virgin/blended with other pavement materials, their performance under repeated traffic loading, especially in base/subbase layer application is reported minimum.
- However, most of the previous works focused on the use of coal ash in embankment and subgrade construction; and its use in the subbase layer remains to be thoroughly examined.

1.8 Research Goal

This study investigates the strength characteristics (UCS and CBR) as well as resilient characteristics (resilient moduli and permanent strain) of modified coal pond ash in pavement subbase layer application.

1.9 Objectives

The primary objectives of the present study are as follows:

- To study the role of cementitious stabilization, reinforcement inclusion and their combined effect on strength characteristics of coal ash
- To determine the resilient modulus characteristics of modified coal ash under repeated load conditions
- To investigate the permanent deformation behaviour of modified coal ash under repeated load condition with multi stress levels
- To study pavement design analysis by thickness optimization and evaluate its economic assessment

1.10 Organization of Thesis

The work carried out in this investigation is presented in the following chapters.

Chapter-1: Introduction: Presents the research needs and motivation, objectives and outline of the dissertation

Chapter-2: Literature Review: Presents the review of literature relevant to this dissertation

Chapter-3: Materials and Methodology: Deals with the source of materials, laboratory evaluation of materials, and methodology adopted in the present study.

Chapter-4: Strength properties: Presents strength characteristics of untreated and treated pond ash.

Chapter-5: Resilient Modulus: Deals with laboratory experiments to find the resilient modulus of untreated and treated pond ash.

Chapter-6: Permanent deformation: Presents the evaluation of permanent strain of untreated and treated pond ash.

Chapter-7: Optimization and Economic assessment: Deals with pavement design analysis by thickness optimization with KENPAVE software tool, and evaluate its economic assessment.

Chapter-8: Summary and Conclusions: Presents the main findings, conclusions and suggestions for future study.

CHAPTER – 2

LITERATURE REVIEW

2.1 General

A pavement is a structure, which undergoes repeated loading under wheel path of varying magnitude. The pavement is structurally classified as a flexible or rigid pavement. Both types of pavements typically possess stiff soil layer as the bottom layer. A flexible pavement consists of one or two top layers made of bituminous concrete overlying base layers made of granular materials. A rigid pavement consists of stiff cement concrete layer overlying a granular layer. The load carrying capacity and the performance of pavement depends mainly on the response of these materials and their interplay in transferring the stresses from one layer to the other. Hence the material parameters are the most critical in the pavement design process (Arshad and Ahmed 2017).

The pavement granular materials are characterised by various experiments developed in the course of time. The initial experiments developed were based on the soil classification system, though not much theoretical basis was used for the development of empirical tests. Owing to their simplicity and ease of analysis, the empirical tests were prevalent for a long time. In the meantime, the industry and the development in vehicles with multi axle wheel configurations and heavy load capacities led to the changes in the pavement structural design. This has initiated the need to develop mechanistic pavement design methods, which in turn has given a way to more robust and reliable material characterisation techniques. Laboratory experiments provide data for understanding the behaviour of the pavement materials. Only a limited number of data under constrained condition can be obtained under laboratory conditions. If one needs to understand the complete spectrum of the mechanical property under realistic loading conditions, the laboratory experimental data to be conceived are considerable and such experimental data can become meaningless if not interpreted properly. A constitutive model can fill this gap and a properly calibrated and validated model can be used for predicting the material response under varying loading and environmental conditions. This chapter reviews the literature on experimental characterisation and modelling of the response of various pavement material used in pavement engineering.

2.2 Material Characterization for Flexible Pavement Design

Till early 20th century, pavements were constructed using the rule of thumb procedure. As there was no common standard procedure of pavement construction back in those days, they adopted some methods from the knowledge gathered from long term observations. Later, on the basis of performance observation, empirical methods were developed and standardised, but were confined only to the local materials available and thus could not be adopted in the case of new or alternate materials. In addition, materials characterised based on the simple index tests and methods did not consider the frost susceptibility of pavement materials. The main problem of traditional design methods is that they are restricted only to a few types of pavement materials and design procedures.

Further, due to urbanisation, the rate of availability of local virgin aggregates decreased and, as a result, engineers have been forced to adopt by-products and recycle aggregates as pavement materials. On the other side, because of increased traffic, the deformation/failures of roads increased, leading to an increase in maintenance cost. The empirical methods considered in the design of pavements are failed to adapt to the above-mentioned situations in most of the work sites. Hence, to overcome these drawbacks, mechanistic design procedures have been developed. These design methods are used to figure out the deformations of pavement materials and layers under various loading and environmental conditions. These methods provide resilient modulus factor for better understanding the characterisation and permanent deformation for finding out rutting in the pavement layers.

2.3 Environmental Consideration on Using Traditional Materials

The most common traditional materials used in road construction are crushed aggregate, crushed rock, sand and gravel. Though it requires less resources/energy to produce/get such conventional materials for the construction, transporting cost (oil, labour, and maintenance) is the single biggest variable that plays critical role in deciding the overall cost of road construction. To minimise these costs, sand and gravel mines are often opened near a specific road project and then abandoned once the project has been finished; and which led to extensive despoliation and decreased air quality at and near the mining site. The opened mines further need water to wash some of their products and to handle the dust on site. Many uses of inadequate ground water that competes undesirably with the growing demands of domestic water usage meet this need (Sarkar and Dawson 2015).

Under the scheme of extension of road network construction programme in India, thousands of kilometres were constructed and scheduled for the future. In constructing these roads, most of the work sites still rely on such traditional pavement materials. In the current scenario, the availability of such traditional materials has also become uncommon, necessitating the traditional materials to be replaced by alternative materials.

2.4 Secondary Materials

Use of waste and recycled materials as alternatives helps conserve the issue of non-availability of good-quality materials and assists in problems resulting from unwanted materials. These materials generally referred to as secondary materials and their usage as a substitute (partially/entirely) in construction for traditional materials bring greater awareness of the substantial quantities of 'stocked waste production' which arises from the extraction and construction/demolition industries. The alternative materials considered for road works include blast furnace and steel slag, spent oil shale, china clay waste, slate waste, rice husk ash, millet husk ash, corn cob ash, coconut shell ash, waste foundry powder, cement kiln dust, fly ash, bottom ash and demolition and construction waste. These materials are subjected to various laboratory tests prior to their use in construction works.

2.5 Soil Stabilization Concept

In general, the pavement bearing capacity is sustained by the subgrade, unbound base and sub base. The minimum CBR required for the sub-base is 30%, and when it is not met, the sub-base should be improved (Patel et al. 2016). Over time, researchers have concentrated on stabilising pavement materials by considering environmental and constraints which have resulted in various stabilisation/modification concepts that are both practical and economical.

The main objective of soil stabilisation is to enhance strength, durability and resistance against the external loads acting by bonding soil particles together. This concept is used to treat a wide range of materials including expansive clays to granular materials (Firoozi et al. 2017).

There are different types of soil stabilisation methods established for improving the engineering properties of pavement materials for subgrade, base/subbase course constructions by mixing/using various additives in required proportions. Stabilisation through mechanical force such as mechanical compaction/densification, soil replacement, surcharge loading, stone column, and piling has been adopted to improve soil properties by mixing or blending soils of two or more gradations obtain required strength specification. However, scarcity of conventional materials coupled with economic constraints, the research has focused toward chemical alteration using the application of chemically active materials like cement, lime, fly ash, bitumen, or in combinations of these materials, synthesised chemical additives or fibre to the soil. as an alternative option. The selection of type and amount of additive to be used depends on the soil classification and the degree of improvement in soil quality desired. In general, smaller quantities of additives are needed when it is merely desired to modify soil properties such as gradation and plasticity. The higher quantity of additive is used when it is desired to enhance the strength and durability substantially (Sherwood 1993; Afrin 2017).

Many research studies have developed a variety of approaches to achieve the benefits through the stabilisation concept, and all of these approaches fall into two broad categories namely:

2.5.1 Chemical Stabilisation

Chemical Stabilisation technique is primarily dependent on the chemical interactions between pozzolanic soil minerals and cementitious stabiliser, resulting in enhancing overall geotechnical properties. Researchers have suggested using alternative stabilisers with the growing problem posed by these secondary materials and their local supply. They have researched soil stabilisation using low-cost approaches (Wang 2002).

2.5.2 Mechanical Stabilisation

Mechanical stabilisation is accomplished by altering natural soil particles' physical structure by either induced vibration or compaction and by incorporating coarse or fine materials and geosynthetic materials. However, over the last three decades, mechanical stabilisation through geotextiles materials has been used extensively to construct pavements, which resulted in a substantial improvement in the performance of the structures by contributing higher reduction in permanent strain (Cicek 2019).

2.6 Origin of Coal Pond ash

India is the second largest source for the coal ash generation (200 MMT/year) after China, since it is the main source of energy for electricity production (Bhatt et al. 2019). Coal ash production as waste by-product can be, in any form, namely fly ash, bottom ash and pond ash (mixture of both fly ash and bottom ash). In most of the cases it is called as fly ash regardless of shape/ size of particle. Coal ash is a heterogeneous substance; which consists of SiO_2, Al_2O_3,

Fe_2O_3 and occasionally CaO as its main chemical constituents. The quality of coal ash generation is highly depends on the type of coal burned. In general, these fly ashes are classified into two classes depending on the calcium oxide (CaO) content, i.e. class C (CaO > 12%) produced by burning of anthracite and bituminous coal, and class F (Cao <10%) produced by burning of lignite and sub bituminous coal materials. Apart from the various differences, all the chemical constituents presents in coal ashes make them as pozzolans-siliceous or siliceous and aluminous materials (Sridharan and Prakashan 2007, Moghal 2017).

In view of its annual production at an alarming rate, the Government of India is at the point where it is strategically seeking ways of mitigating coal ash through treatment, re-use and beneficiation.

2.7 Existing Use of Coal ash

Across the globe, coal ash is being used in various applications such as cement production, concrete production, soil stabilisation, asphalt, embankment, flow-able fill, waste stabilisation due to its cement-like property, and in the agricultural sector. However, only 50% of the annual production is utilised in India. In comparison, coal ash has a broad application in the pavement structure integrated into sub-grade, granular base/sub-base, asphalt base/surface and structural filling. Further, it has also been paired with other by-products to improve the performance of pavement materials. Due to the high potential for sustainable use, the use of coal fly ash in road and embankment construction has been successfully demonstrated in the country. The Ministry of Surface Transport (MOST) and Central Public Works Department (CPWD) have accepted the use of fly ash and have executed many projects. However, the studies available specifically on the use of pond ash have been found to be minimal.

2.8 Coal ash as Partial Replacement Material

There are several studies presented on the use of coal ash as partial replacement to the natural soil/aggregates which have been extended to various geotechnical applications such as backfilling material, land liner, land reclamation, ground improvement, stabilisation, embankment materials (Suthar and Agarwaal 2018). The studies reported that despite of some morphological differences, coal fly ash materials exhibits improved geomechanical properties (compaction behaviour, compressibility, shear strength, conductivity, CBR, unconfined compressive strength and failure behaviour characteristics), also which are in general comparable to those observed in natural conventional materials. (Arora and Ayeilek 2005; Consoli et al. 2001, 2009, 2010, 2017; Gupta and Kumar 2017; Yadav et al. 2018).

As class C fly ashes possess self-hardening property and can be used as stand-alone stabilising agent to stabilise soils. Whereas Class F fly ash has a low lime content with pozzolanic in nature, is not ineffective as a stabilising agent by itself, and thus has to be mixed with either lime or lime and cement to be able to stabilise soil. During the stabilisation process, pozzolanic reaction between fly ash, lime in presence of water gives rise to cementitious products, which bind the soil particles. The reactive silica and alumina present in fly ash reacting with lime as given below:

$$Ca\,(OH)_2 + SiO_2 + H_2O \rightarrow CaO.SiO_2.\,2H_2O \text{ (i.e. Calcium silicate hydrate)}$$

$$4Ca\,(OH)_2 + 4Al_2O_3 + 9H_2O \rightarrow 4CaO.Al_2O_3.13\,H_2O \text{ (i.e. Calcium aluminate hydrate)}$$

$$2Ca\,(OH)_2 + 2Al_2O_3 + 2SiO_2 + 6H_2O \rightarrow 2\,CaO.Al_2O_3.SiO_2.8H_2O \text{ (Startling compounds)}$$

Nicholson and kashyap (1993), reported that addition of fly ash decreased the Plasticity Index and increased the strength characteristics such as UCS and CBR of tropical soils. Parsons and Milburn (2003), performed durability studies on soil-class f ashes mixed with additives cement and lime in various proportions by subjecting freeze-thaw cycles, reported a relative, soils treated with cement and fly ash showed lower soil losses than lime-treated soils. Kate et al. (2005), examined the effect of fly ash alone and fly ash –lime blends at different percentages on expansive clays observed that the reduction in free swell index and percent swell with improved UCS. Also, recommended that fly ash without lime also yields better stabilization of soil, but relatively lower than fly ash- lime mix. Vishwanathan et al. (1997), Arora and Aydilek (2005), Moghal (2017), stated that lime and fly ash are a good combination for stabilizing both silty and sandy soils because they considerably enhance the stiffness of the final product. Further, it was noted that the required base layer thickness decreases when treated with fly ash and lime. Edil et al. (2006), reviewed for CBR on soil-fly ash mixtures and showed a substantial increase in CBR of soils. For the addition of 7% OMC, the CBR of untreated soils varied from 1 to 5. Upon on addition of 10% fly ash CBR value enhanced to 17. Likewise, for 18% fly ash addition CBR enhanced to CBR 31. Brooks (2009), has recommended that the needed fly ash content vary based on the type of soil, for improving the engineering strength properties; and also recommended fly ash content og 15 to 30% by means of UCS or CBR varies between 15 and 30%. However, the improvements in strength properties clayey soils with fly ash addition were not sufficient for use in roadwork. Hence, they recommended an external stabilizing agent to fly ash to improve the properties of soils further. Brook et al. (2011), presents the feasibility of utilizing coal fly ash (CFA) and lime kiln dust to enhance the properties of problematic soils. They concluded that the plasticity and swelling potential of the soils were decreased by 40% and between 40 and 70% when stabilized with fly ash and fly ash-lime, respectively. The findings also showed a marked improvement in the strength of the soils for CBR and UCS. Kolay et al. (2011), Class F pond ash has been used to stabilize the highly compressible peaty soils to improve their compaction and compressive strength characteristics. It has been observed the addition of pond ash increased the MDD due to the replacement of voids in the peat matrix by the finer pond ash, and reduced OMC due to the cementitious reactions. With the curing period, the overall strength of soils (Peat) has increased satisfactorily. Yadu et al. (2011), examined the effect of rice husk ash and fly ash on geotechnical properties of expansive soil. They reported that the CBR and UCS values had shown a significant increase of around 125%–200% and 76%–192%. Mir and Sridharan (2013), performed various index and strength tests on BC soils mixed with two types fly ashes (Neyveli-Class C, and Badarpur-Class F) of various proportions (10, 20, 40, 60 and 80%). The test results showed the improved geomechanical properties of soil with fly ash addition with optimum content of 10% Neyveli fly ash and 40% Badarpur fly ash, respectively. Oormila and Preethi (2014), Native clayey soil has been stabilized for its UCS and CBR by adding fly ash and blast furnace slag. An increment of 75 and 281 %; 98 and 600 % has been observed in UCS and CBR, respectively, with fly ash and blast furnace slag, added individually, at their optimum percentage. A combination of 15 % fly ash and 25 % blast furnace slag has shown a further improvement in CBR by 800 % to that of the native soil. Thakur and Han (2015), studies performed on fly ash/cement–stabilized RAP materials, indicated a substantial increase in UCS, CBR, and stiffness. Kumar and Gupta (2016); Singh et al. (2016), carried out various experimental programs to enhance soil strength characteristics by using fibers, cement, and waste material such as rice husk ash.

In general, the mechanical strength characteristics of fly ash based stabilized soils vary with inherent soil properties, addition ratio of fly ash/fly ash-additives, delay time, water content at compaction time**.** Overall, a comprehensive evaluation of primary and secondary materials is essential to adopting type of stabilization technique for the adequate performance of pavement structure.

2.9 Studies Related to Coal ash

Pandian (2013), studied the shear strength characteristics of fly ashes mixed with soil samples and reported that fly ash improves the shear strength and bearing capacity of the soil. Hence, the soil-fly ash mixture can be used as the base materials for the roads, backfilling works. Das and Yudbhir (2005), studied the geotechnical characteristics if Indian coal ashes reported that for low lime ashes, the UCS values attained at OMC is primarily attributed to the capillary forces. Bera and Ghosh (2007), studied the compaction behaviour of 3 types pond ash samples and reported that compaction characteristics (MDD, OMC) of pond ash vary from 8.4 to 12.25 kN/m^3, and 29-46%. Madhav et al. (2008) and Trivedi et al. (2004), conducted Oedometer test to study the collapse behaviour of coal ashes and reported that ashes with more than 50% of the particle with silty size are collapsible and the lower limit of collapsible potential of ashes is 0.0075 at 80% degree of compaction. Jakka et al. (2010), examined the shear characteristics of pond ash samples collected at both inflow and outflow point and reported that the shear behaviour of ash samples collected at inflow point is identical to the sandy soils (Yamuna river) in many respects due to its good interlocking effect b/w irregular shaped coarser particle; whereas the outflow ashes showed less shear strength. Mishra and Das (2012), performed 1D-consolidation on pond ash specimens and reported that 60 to 85% settlement of pond ash is taken place within 1 min of loading, and Cv is in the range of 0.0195-0.1885 cm^2/min is comparatively low and decreases with an increase of time. Singh and Kalita (2013), performed strength studies and reported that Observed that for given compaction energy, the UCC and CBR values showing higher values with moulding water content less than OMC. Sridharan and Prakash (2007); Mohanthy (2015), studied the geotechnical characteristics of various coal pond ash samples (chemical, physical, morphological, mineralogical, and engineering properties) reported that pond ash has a good potential for the use as geotechnical applications.

2.10 Modified Moal ash in Pavement

This section presents a detailed review of literature on use of fly ash/pond ash as virgin material in particular for pavement applications. The literature is presented in three subsequent sub-sections, viz, chemical/cementitious, reinforcement inclusion, combined effect of cementitious and reinforced inclusion modification to examine its geomechanical characteristics.

2.10.1 Cementitious Modification

Wong and Ho (1989) cited in Lav and lav (2014), conducted a field study with 100% fly ash (without the addition of admixture/aggregates) as base material at Fulsher, Texas, USA with a trial track construction and reported an average UCS of about 255 kPa with 28 days of curing. Further, after four months of construction, the corrugation of the top asphaltic layer was also reported in some locations. Hence, the trial was considered as failure, and no further attempt was made to do this type of practice. Therefore, fly ash itself cannot be considered as a component material for the road base/subbase construction when it has to be placed in base/subbase layers of pavement. It should be treated with suitable additives to achieve significant strength improvement and adequate structural performance as pavement layer (Sarkar and Dawson 2015).

Many research studies (Chand and Subbarao 2007; Sivapulliah and Moghal 2011; Pani and Singh 2017; Suthar and Aggarwal 2018) reported that the coal combustion fly ash treated with various cementitious additives such as cement, lime, gypsum, GGBS, silica fume exhibited enhanced strength characteristics due to development of pozzolanic hydration products (i.e. calcium silicate hydrates C-S-H, calcium alumina silicate hydrates C-A-S-H).

Gray et al. (1994), evaluated the performance of compacted, aggregate-free, cement-stabilized fly ash base beneath a highway shoulder. Nunes et al. (1996), investigated the different mix combinations of secondary materials/aggregates and binders such as fly ash mixed with cement kiln dust, fly ash mixed with gypsum and lime, and granular blast furnace slag and some combination of china clay and coarse aggregate. They suggested various procedures and techniques for the standard examination of secondary materials for its pavement foundations. Lav and Lav (2006), studied the performance of aggregate free cement stabilized fly ash with an aim of using high volumes of this waste material as a base material in road pavements. They carried out an accelerated full-scale road test reported the cement content and layer thickness should not be less than 8% and 300 mm, respectively. They suggested that mixes with cement content less than 8% may be used as subbase materials instead of using in the base layer. Chand and Subbarao (2007), performed the strength and slake durability tests and reported that UCC of 4.8 and 5.8 MPa and durability indices of 98 and 99% (180 days of curing) when samples treated with 10 and 14% lime. Ghosh (2009), reported that pond ash stabilized with lime (> 6%) and phosphogypsum cured for 28 days meets the requirements for base course material (As per IRC 37-2001). Datta et al. (2017), conducted tests on fly ash modified with both lime and phosphogypsum and observed the improvement in strength up to 28 days, after that only marginal improvement. They also developed empirical models and recommended its use in field applications. Singh and Pani (2017), studied the behaviour of coal ash modified with lime at different compaction energy levels. Further, they also investigated that effect of curing and temperature parameters on UCS and CBR values. They found that curing and temperature favour better pozzolanic reaction for lime content of >4%. Suthar and Aggarwal (2018), reported that CBR of pond ash increased with an increase of lime content with steeper rate up to 4%. Patel et al. (2019), investigated the strength properties of fly ash-lime (FAL) and fly ash-cement (FAC) composites for subbase application, concluded that Fly ash met the minimum strength requirements recommended by the IRC 20-2002 at minimum 6% for both cement and lime.

There are also certain studies, which show that the ultimate strength of fly ash-lime blends increases with long periods of curing and high content of silica and alumina. Nevertheless, with the usage of these cemented additives in coal ash can induce brittle behaviour at low strain level, which in turn affects the long-term stability and serviceability of the pavement structure (Kaniraj and Gayathri 2003; Tang et al. 2010).

2.10.2 Reinforcement Inclusion

The introduction of reinforcement (soil reinforcement) to the pavement structural layers has also identified as an effective, reliable method (Tang et al., 2007). Over the last three decades, many experimental studies were conducted to evaluate the use of geosynthetic products as reinforcement inclusion in pavements. The beneficial behavioural effects with reinforcement inclusions of any conventional form such as strip, sheet, mat, grid and fibre have been extensively studied and reported (Koerner R.M 2012; Li et al. 2014; Jayanthi and Singh 2016; Sridhar and Kumar 2018) have reported that incorporation of reinforcement in soils/ashes and other pavement material mixes leads an increase of load-bearing capacity, stability thereby contributing to reduction in rut depth, as well as a reduction in the cost of construction and time. Generally, the reinforced technique used to improve its mechanical behaviour of soil and other similar particulate materials can be broadly divided into two types: i) systematic reinforcement ii) randomly distributed fibre reinforcement. In the case of systematic reinforcement, the reinforced elements are oriented or placed in a position so that maximum shear resistance is developed along the slip plane in a soil. Reinforcing soils using fibres/inclusions randomly in these materials is another variant and focus of research (Koerner R.M 2012). As compared to traditional systematic geosynthetic forms (strips, geotextiles, geogrids), randomly oriented fibres have the following advantage: i) simple in adding and mixing with soils, like mixing of soil with cement, lime, or other additives (ii) control the potential plane of weakness parallel to the plane of reinforcement (iii) maintaining strength isotropy in soil mix (iv) change of physical properties of soil and has no impact on the environment (Chakraborty and Dasgupta 1996; GLS Babu and Vasudevan 2008; Tang et al. 2007; Li et al. 2014). For these reasons, researchers (Kumar and Singh 2008; Tang et al. 2007; Jayanthi and Singh 2016; Kumar et al. 1999; Kaniraja and Havanagi 2001; Tiwari and Ghiya 2013) have shown an interest in finding mechanical behaviour of in soil, coal ash or soil-coal ash mixes with discrete fibre reinforcement.

Refeai and Suhaibani (1998), have performed CBR test on dune sand reinforced with polypropylene fiber, reported that the inclusion of fibers increased the CBR values of sand with an optimum content 0.4%. Kumar et al. (1999), reported that the inclusion of reinforcement in sand-pond ash composites increased the UCS and failure strain, peak friction angle, cohesion, and CBR values, especially at 0.3 to 0.4%. Kaniraj et al. (2001, 2003), conducted various geotechnical characterization tests on raw and fibre-reinforced fly ashes. They reported that the fiber inclusions increased the strength of the raw fly ash specimens and observed change in their failure behaviour. Bera et al. (2009), studied the shear strength behaviour of reinforced pond ash and reported its suitability as an alternative sustainable construction material. Chore et al. (2011), investigated the strength properties of sand such as CBR and shear strength found the optimum performance at 50% fly ash addition and 1% fibre inclusion. Sreedhar et al. (2011b), reported the increased CBR values from 35 to 59% when 1% randomly distributed fiber with an aspect ratio of 10 was used as reinforcement in pond ash are better than un-reinforced pond ash. Sarkar et al. (2012), reported the enhanced mechanical behaviour in terms of deformability, permeability, strength, volume stability (shrinkage and swelling), and durability of polypropylene fiber-reinforced pond ashes through their laboratory experiments. Singh and Sharan (2014), reported that inclusion of fibers of various lengths in pond ash give ductility to the specimens and observed lower post-peak stresses compared to un-reinforced pond ash sample, concluded that 12 mm long polyester fibers are found to be effective in improving the UCS than 6 mm fibers. Arora and Kumar (2019), have found that the provision of compacted pond ash layer and the addition of fibres into pond ash significantly influence the ultimate bearing capacity of soft soils. Besides, several authors studied the cyclic behaviour of different types of fibre-reinforced soils/soil-fly ash composites by triaxial testing; they observed an increased stiffness and resistance to liquefaction, shear modulus and damping ratio of the test specimen (Yetimoglu et al. 2005).

2.10.3 Combined Additive Treatments

Various researches have evaluated the effect of cementitious additives in combination with reinforcement inclusions the geomechanical behaviour of on soils and other pavement related materials.

Kaniraj and Gayathri (2003), Found the optimum cement content of 15 and 18% based on EPRI criteria (2760-3100 kPa after seven days and not exceed 5100 kPa) for base course applications. UCS and modulus of rupture of fiber reinforced fly ash ashes have higher failure strain than unreinforced cement modified ashes. Gupta and Kumar (2017), Performed UCS test on pond ash-rice husk ash-clayey soil specimens and investigated the impact of fiber reinforcement. They observed the more brittle behaviour in cement stabilized specimens than in un-stabilized and reinforced specimens. Maximum strength improvement of 154% is observed in the reinforced specimen with optimum pond ash and rice husk ash content in mixes 40 and 10%, respectively. Sahu et al. (2017), investigated the strength (CBR) and durability characteristics of the proposed composite mixture of (fly ash + lime sludge + lime + gypsum) for construction industries and found in a 1:1 ratio of fly ash and lime sludge, 12% lime and 1% gypsum as optimum content. It is also noted that the CBR values 48% and 65% after 4 and 7 days of soaking, and hence suggested for application as base course layer material in pavements. Xiang et al. (2018), examined that the addition of waste polyester fabric fiber improved the peak and residual strength of cemented sand and changed the brittle behaviour to more ductile one. Yadav et al. (2018), stated that the partial replacement of cement-treated clayey soil by pond ash with the inclusion of fibers caused a substantial increase in strength, decrease in the stiffness, and rate of loss of post-peak strength. It is also concluded that pond ash up to 20% in the cement-treated fibre-reinforced soil can be considered an efficient method for ground improvement. Kaniraj and Fung (2018), examined the effect of reinforcement in fibers as well as meshes forms, on UCS of both lime treated and untreated soils, and observed an increase in UCS values. In all lime treated soils the failure strain decreases with increased curing period; with the inclusion of fiber and meshes increased the failure strain and made stress-strain behaviour to ductile (fiber strain > mesh strain). Kumar and Sharma (2018), observed the increment in CBR of pond ash up to 80% by adding cement kiln dust (25%) and fiber (1%) and proposed the composition for highway pavements. Dhar and Hussain (2018), reported that modified with lime and fiber showed increased peak axial stress, reduced post-peak strength loss, improved stiffness at a significant stain level, and change the mode of failure ductile. Sarkar and Dawson (2015) and Patel et al. (2019), also studied the economic assessment in utilizing coal ash as pavement material and reported that estimated cost saving up to 10% and 26% of the cost of the project without compromising its structural performance.

Form the context of the above studies, proved that the beneficial effects on the use of coal ash mixtures for road constructions, which not only contribute to the possible application of coal ash in bulk quantity but also provide an economical solution. In addition, the enhanced properties of materials can reduce the required thickness of pavement layers and provide relatively low-cost alternative solutions in traditional pavement constructions where no conventional base/subbase materials are available near the site.

2.11 Determination of Resilient Modulus

Stiffness property is an essential mechanical characteristic used in pavement design. The relative stiffnesses of the materials used in various pavement layers dictate the distribution of stresses and strains within the pavement system. In order to characterize the subgrade/subbase/base layer support for pavement structures, the Mechanistic-empirical pavements Design (M-EPD) guidelines have recommended M_R as stiffness parameter. It is commonly defined as the unloading modulus under repeated loading. It is obtained by performing repeated triaxial tests in the laboratory. Based on precision, the M-EPD methods categorized M_R into 3 levels. Level 1 consists of the M_R values obtained from cyclic load triaxial tests. Level 2 consists of M_R values obtained from empirical correlations from the engineering properties of materials such as CBR, UCS. In Level 3, M_R values obtained from the correlations through index properties of soil have very low accuracy. In recent years, research has focused on developing test methods to evaluate the resilient characteristics of various pavement materials (Noolu et al. 2018). Several laboratory experiments, such as cyclic triaxial, resonant column, simple shear, and hollow cylinder tests, were developed to simulate the pavement loading response on compacted soils to determine the M_R of soils. (Lentz and Baladi 1981; Puppala et al. 1999).

Although the use of M_R to quantify the mechanical properties of granular material is well-developed based on sound theoretical reasoning, difficulties have arisen about the test protocol. Unlike the CBR test, the test method to find M_R is not unique. Initially, the AASHTO guide (AASHTO 1986) established a standard procedure for determining M_R of subgrade Soils (AASHTO: T274-82, 1986). Besides, many protocols were developed for the measurement of resilient modulus, and later AASHTO proposed two different procedures AASHTO: T292 (1991) and AASHTO: T294 (1992) for determination of M_R of subgrade and subbase soils and finally updated and brought out in AASHTO: T307-99 (2003).

In the following section, the literature survey conducted on various pavement materials and the influence of multiple factors which affect the resilient modulus behaviour is reported.

Monismith et al. (1967), investigated the effect of confining pressure on granular materials reported nearly 500% increment in M_R is observed with confining pressure increased from 20 kPa to 200 kPa. Morgan (1966), conducted repeated triaxial load test on granular material and observed that increment in deviatoric stress increases in M_R under constant confining pressure. Allen and Thompson (1974), reported that the constant confining pressure on specimen results in the enhancement of M_R compared to variable confining pressure. According to Smith and Nair (1973), principal stress is less effective on M_R, i.e., with a change in total stress from 70 kPa to 140 kPa there is little increment observed in the M_R. Morgan (1966), conducted the repeated triaxial test on base materials and noted that with an increase in deviatoric stress M_R values decreased.

Trollpe et al. (1962), noted 50% increment in M_R when loose soil is replaced with dense soil. Brown and Selig (1991), reported that the effect of density on M_R is less for granular material but Hicks (1970) argued that the effect of density is more on partially crushed aggregates compared to fully crushed aggregates and the reason behind it is that the fine content is more in case of crushed aggregates. Barksdale and Itani (1989), noted that the density is effective when lower stress is applied compared to higher stress level.

Hicks and Monismith (1971), noted that the addition of fines to partially crushed aggregates results in the increment in M_R. Hicks (1970), observed that the addition of fine content from 2 to 10% results is a minimal increment in M_R, but Barksdale and Itani (1989) reported that nearly 60% increment in M_R is observed 12% fines. Jorenby and Hicks (1986), noted that the addition of clay fines to crushed aggregates causes enhancement in M_R due to filling the pore spaces. Thom and Brown (1988), reported that the uniformly graded crushed aggregates possess higher M_R compared to well-graded aggregates. Heydinger et al. (1996), observed that the addition of moisture leads to the increment in M_R of uniformly graded aggregates up to the optimum moisture content and studied the effect of grading which showed a minimal effect in case of gravel whereas for limestone with open gradation showed higher M_R compared to other grades.

Haynes and Yoder (1963), stated that the increment in moisture from 70% to 97% results in 50% decrement in M_R. Hicks and Monismith (1971), observed that moisture to base materials leads to constant decrement in M_R. Thom and Brown (1988), observed decrement in resilient modulus with increasing moisture content due to the lubrication effect. A study was performed by Raad et al. (1992), on different parameters, whereas moisture content is considered as the most effective in case of well-graded compared to uniformly graded materials. Dawson et al. (1996), observed that the addition of moisture tends to increase the M_R of base materials up to the optimum moisture content. Edil et al. (2006), used fly ash to improve the M_R of fine-grained soils and concluded that fly ash stabilization is more suitable for low plasticity clayey soils. Ardah et al. (2017), studied the influence of cement-fly ash on M_R behaviour of soils at varying moisture conditions. They found that the stabilization effect was more pronounced at higher water content.

Boyce et al. (1976), conducted the cyclic triaxial test on crushed limestone specimens. They observed that the specimens subjected to preloading with few load cycles resulted in the reduction of stress history, and after that, it got nullified. Hicks (1970), observed that stress history is eliminated after 100 load cycles, but Allen (1973), reported that stress history is eliminated after 1000 load cycles.

Most of the standard testing protocols emphasize the number of load repetitions. All protocols include a conditioning sequence of 500 to 1000 cycles to eliminate errors due to improper seating of end platens and consecutively the cycles are repeated to around 100 to 500 times to remove the influence of plastic strain. However, experiments carried out by many researchers have shown that several thousand cycles of loading were required to remove the plastic strain (Seed et al. 1962; Tanimoto and Nishi 1970). The number of load applications also contributed to the material response variation. Studies by Seed et al. (1962), have confirmed that for a specific range of stresses even after 10000 load repetitions, the permanent deformation increases. Likewise, experiments carried out by Tanimoto, and Nishi (1970), have noted that the choice of an appropriate number of stress applications plays a prominent role in the determination of the actual resilience characteristics. They also indicated that the resilient strain couldn't reach a constant value within 10000 load applications. Moore et al. (1970), observed that the increase in the number of load cycles caused minimal increment in M_R due to moisture content loss. Puppala et al. (2009), performed repeated load triaxial test on three different soils and observed that the resilient modulus variation is minimal for all the soils.

A wide range of loading frequencies along with loading magnitudes has been used. A load cycle duration of 0.1 second with a 0.9 second rest period is normally adopted in most of the standards (LTPP, 1996; AASHTO: T307, 2003). Tanimoto and Nishi (1970), indicated through experiments that the resilient strain was considerably low for large frequencies. This is not surprising as large frequency results in the small load duration to be in the elastic regime.

Heydinger et al. (1996), observed that the gravel showed higher M_R compared to lime stone. Allen and Thompson (1974), noted that the crushed aggregate particles showed higher M_R compared to uncrushed aggregates. Barksdale and Itani (1989), reported that the angular particles had higher M_R properties compared to rounded particles.

Research studies have been carried out to understand M_R behaviour with the various compact efforts such as kneading compaction, vibratory compaction, and static compaction. Seed et al. (1962), studied the variation in M_R due to kneading and static compaction. Lee et al. (1995), conducted M_R test on the specimen prepared using impact and vibratory compaction methods. The specimens prepared using vibratory compaction exhibited higher dry density at increased water content, lesser permanent deformation, and 40% increment in M_R compared to that of the specimen compacted using impact compaction. The variation is attributed to the non-uniform compaction for impact compaction and due to the different fabric stress history of the compacted soil. Mamatha and Dinesh (2017), conducted a repeated triaxial test on Black cotton soil. Lime stabilized Black cotton soil with different density and observed that density is more effective in case of virgin compared to lime stabilized Black cotton soil.

Further, Refeai and Suhaibani (1998), performed M_R tests on dune sand reinforced with Polypropylene fiber and stated that The σ_c and σ_d had the least effect on the MR values of fiber-reinforced sand specimens in the fiber content range of 0.2 to 0.4%. Arora and Aydilek (2005), reported both strength (CBR, UCS), and (M_R) resilient characteristics values of soil-fly ash mixtures treated with cement. They concluded that the above-mentioned mechanical properties are increased with increased cement content; however, the rate is decreased beyond 5% cement. Kumar et al. (2006), Confirmed from his experiments that while fly ash had the lowest CBR of 9%, its behaviour under dynamic load is better than that of stone dust. Edil et al. (2006), examined the role of fly ash on inorganic soils and reported appreciable increases in M_R from ranged between 3 and 15 MPa soil alone to 12 and 60 MPa for 10% fly ash addition and 51 and

106 MPa for 18% fly ash addition. Kumar and Singh (2008), investigated the resilient characteristics of soil-fly ash composites modified with polypropylene fiber for its subbase layer application and reported improved resilient characteristics compared to an unreinforced specimen. Puppala et al. (2009, 2011), examined the resilient behaviour of various subgrade soils and recycled materials modified with lime and cements. They concluded that M_R of aggregates improved by 32-50% with cement treatement. Tilti et al. (2012), reported that mixing 50% of the bottom ash with subgrade soil is considered an optimum amount to improve the M_R of the bottom ash-soil mixtures. Rout et al. (2012), examined the role of lime and cement on soils and reported nearly 2.5 - 2.8 times increase in M_R of the soils. Lopez et al. (2012), conducted repeated load triaxial tests on soil, ashes, and soil-ash mixtures (fly ash and bottom ash) with and without lime addiction. They concluded that the mechanical behavior consistent with modified soils for low traffic roads requirements. Arulrajah et al. (2012, 2013), investigated the resilient behaviour of various recycled materials like Recycled concrete Aggregate (RCA), Crushed brick (CB), Waste rock (WR) for their applications in pavement structures. Lav and Lav (2014), conducted the cyclic triaxial test on cement and lime treated fly ash specimens after 90 days of curing. They observed that the stress-strain of stabilized fly ashes were non-linear (stress-dependent). The axial strain rate under corresponding stress paths decreases with an increase of cement and lime contents. However, the variation in strain with lime and cement was very less. Rahman et al. (2014), studied the resilient characteristics of recycled construction materials as an alternative to quarry aggregates in pavement base or subbase layer with the placement of geogrid reinforcement. They said that compared to virgin materials, the introduction of geogrids in recycled materials significantly affected their M_R values. Sarkar and Dawson (2015), reported the increased stiffness characteristics of pond ash modified with fiber and lime in fixed proportions. Dev and Robinson (2019), stated that pond ash alone is sufficient to produce a flowable fill with 2-3% cement addition (UCS < 0.7 MPa). The observed M_R values of flowable fills varied from 50-305 MPa and which are comparable with the M_R of granular aggregates used for pavement applications. Patel and Shahu (2018), studied the behaviour of industrial waste mixtures and found to be greater than conventional GSB and ranked them like BC soil-Dolime (BCD), FA-dolime (FD), Copper slag-FA (CF), GGBS-FA (GBF) based on the performance the materials. Patel et al. (2019), examined the M_R of fly ash modified with cementitious materials (cement and lime) for its use in pavement subbase layer and reported increased M_R values due to the formation of the cementitious

product due to pozzolanic reaction which results in bonding between particles, and consequence increase in stiffness.

2.12 Studies on Correlation between M_R and Other parameters

The test methods related to M_R evaluation are more sophisticated experimental and measurement system and which requires skilled technical manpower. The test can be considerably complex compared to conventional test methods such as CBR, Dynamic Cone Penetration (DCP), and Unconfined compression (UCS) test, which is not preferred by major transportation agencies. NCHRP (2004), pointed out that M_R can either obtained from laboratory studies or through correlations from index/strength-based parameters of the materials like CBR, Plasticity Index and aggregate gradation, asphalt layer coefficient and dynamic cone penetration. Several investigations also undertook different initiatives to develop statistical correlations relating M_R with alternate test methods. Kim (2007), tried to characterize M_R from Resonant Column (RC) and Torsional Shear (TS) test. Rahim et al. (2004), carried out experiments to correlate M_R and soil index properties and came out with two different equations, one for fine-grained soils and another for coarser soils.

$$M_R = 16.75\left({LL}/{Wc} * \gamma_{dr}\right)^{2.06} + \left({\#200}/{100}\right)^{K_3-0.59} \tag{2.1}$$

$$M_R = 307.5\left({\gamma_d}/{Wc}\right)^{0.86} + \left({\#200}/{log_{cu}}\right)^{-0.46} \tag{2.2}$$

Here LL= liquid limit, γ_{dr} =maximum dry density, γ_d = dry density, wc = moisture content, #200 = % passing 0.075mm sieve, and c_u = Uniformity coefficient.

Other prominent alternate test include back-calculation from non-destructive testing such as the Falling Weight Deflectometer test and estimations from the AASHTO Guide algorithm. As the CBR test is popular test for characterizing the subgrade/subbase strength, it was considered to correlate the CBR with the M_R.

Heukelom and Klomp (1962) developed correlation as M_R *(MPa) = 10 x CBR*. The equation was developed based on Rayleigh Wave and dynamic Impedance testing. The equation was developed for a modulus range of 2-200 MPa. The US Army Corps developed a similar equation () with slightly modification as $E\ (MPa) = 37.3 * CBR^{0.71}$. The South African Council on Scientific and Industrial Research (CSIR) adopted modified equations of form *E = k * CBR,* where k is the factor that responsible for local factors.

Although the number of equations emerged out from many studies, the equation developed by Heukelom and Klomp was considered a preferred relationship. Further, Transport and Road Research Laboratory, Crowthorne, UK also developed a relation E (MPa) = 17.6 x $CBR^{0.64}$. Over time, other researchers have decided that the Heukelom and Klomp equation was inaccurate. For the test data of CBR < 5, the equation underestimates the modulus, and overestimates the same for CBR values > 5. In this regard Main Roads Department, Queensland adopted the relationships: $E\ (MPa) = 21.2 \times CBR^{0.64}$ (CBR < 15), and $E\ (MPa) = 19 \times CBR^{0.68}$ (CBR > 15).

Similarly, The Indian Roads Congress (IRC) adopted a relationship by combining Heukelom and Klomp equation and the TRL equation: $E\ (MPa) = 10*CBR$ (CBR < 5), $E\ (MPa) = 17.6\ CBR^{0.64}$ (CBR > 5). And, some of the other correlations developed for soils are:

a) $M_R\ (MPa) = 37.3\ (CBR)^{0.71}$ (Green and Hall 1975)
b) $M_R\ (MPa) = 17.6\ (CBR)^{0.64}$ (Powell et al. 1984) and
c) $M_R\ (MPa) = 1.75\ (CBR)^{1.46}$ (Dev and Rabinson 2019)

2.13 Numerical Models for Predicting Resilient Modulus

From the past three decades, many research studied have made an effort to investigate the resilient modulus and proposed multiple models to predict M_R based on the test data by using different test methods and pavement materials. These M_R models predict the nonlinear behavior of a pavement layer and this help to develop more rational pavement design procedures. The use of these model relations are prevalent for their simplicity and easy implementation rather than for their reliability in predicting the realistic behavior of the material. Some models have been widely used to predict resilient modulus and examined in many research works.

Dunlap (1963), conducted several cyclic triaxial tests on base materials to find the relation between confining pressure and resilient modulus and proposed a relation equation, which is in log-log form and he also observed that cyclic deviatoric stress is less effective in finding the resilient modulus.

$$M_R = K\sigma_3^n \tag{2.3}$$

Thompson and Robnett (1979), conducted repeated triaxial test on base materials, observed that the deviatoric stress is the more predominant factor in determining the resilient modulus, and proposed a bilinear model between resilient modules and deviatoric stress.

$$M_R = K_1 + K_2\sigma_d \tag{2.4}$$

Moossazadeh and Witzcak (1981), proposed a semi log model between deviatoric stress and resilient modulus and stated that confining pressure is not effective in the determination of resilient modulus.

$$M_R = K\sigma_d^n \tag{2.5}$$

Garg and Thompson (1997), performed a large number of triaxial tests on granular materials and found out that both confining pressure and deviatoric stress are effective in finding the resilient modulus and proposed the below relation.

$$M_R = N_1 q^{N_2}\, {\sigma_3}^{N_3} \tag{2.6}$$

Hicks (1970), conducted several triaxial tests on granular materials and proposed bulk stress model, which considers both confining pressure and deviatoric stress. It is also called as K-φ model.

$$M_R = \; k_1 * \theta^{k_2} \tag{2.7}$$

Uzan (1985), performed numerous cyclic triaxial tests on granular material and proposed a model with respect to both the confining pressure and deviatoric stress and it was widely used to characterize the resilient response, whichconsidered the influence of the sum of the principal stresses. The main drawback of the K-θ model was that it assumed a constant Poisons ratio but in reality, it can vary with the magnitude of the stresses.

$$M_R = k_1 P_a \left({}^{\theta}/_{P_a}\right)^{k_1} \left({}^{\sigma_d}/_{P_a}\right)^{k_2} \tag{2.8}$$

This model uses both bulk stress and octahedral stress for the determination of resilient modulus. This model is recommended by MEPDG (Witczak and Uzan 1985). According to this model, the resilient modulus is determined using the following equation

$$M_R = K_1 p_a \left({}^{\theta}/_{p_a}\right)^{K_2} \left({}^{\tau_{oct}}/_{p_a} + 1\right)^{K_3} \tag{2.9}$$

Witczak and Uzan (1988), developed a model by considering effect of deviatoric stresses and named it as power model

$$M_R = k_3 * {\sigma_d}^{k_4} \tag{2.10}$$

Further performed large number RLT test on subgrade/subbase soils and developed a model and suggested by NCHRP, and considered in many research studies.

$$M_R = k_5 * \left(\frac{\sigma_c}{P_a}\right)^{k_6} * \left(\frac{\sigma_d}{P_a}\right)^{k_7} \tag{2.11}$$

With the help of constitutive relationships, it is possible to predict the behavior of materials. The simplest model is behavior elastic and the popular Hooke's law belongs to this category, where the stresses and strains are related with the help of a material parameter, the Young's Modulus. However, for complex materials such as soils, the material behaviour cannot be approximated to a linearised elastic one. Further, a wide range of factors such as the state of stress, residual or initial stress, volume changes under shear and stress history influences the response of granular materials. In such cases, appropriate constitutive relationships were formulated using the concepts of plasticity, taking into consideration the factors influencing the behavior of soil which provide a realistic representation of the observed behaviour.

The evolution of constitutive modelling in soil mechanics with special relevance to pavement granular materials has come from the simple elastic models to the highly complex plasticity based models. The purpose of all these models is to achieve a better agreement between the predicted and observed soil behavior. A brief review of the popular constitutive models used for predicting the behavior of pavement granular materials is described here. Broadly the models can be classified into two categories i) Elastic models and ii) Elastic-plastic models. Elastic models are the simplest of all and yet are still used widely in pavement engineering applications. The isotropic elastic model belongs to this category of linear elastic models. In elastic – plastic models, the soil behavior is characterized by the existence of recoverable and irrecoverable deformations called the elastic and plastic deformations respectively. It is observed that there exists a yield surface for soils, where the response of the soil changes from elastic to plastic. For stress changes inside a chosen yield surface, the response is elastic. As far as the magnitude of the stresses increase, and the yield criterion is satisfied, the response of the material is that of elastic – plastic material.

2.14 Permanent Deformation of Pavement Material

The permanent deformation or rutting problem in pavement system has been studied with varying successes in recent years (Puppala et al. 1999; Jegatheesan and Gnanendran 2015).

The accurate measurement/estimation of permanent strains of pavement materials would aid in the efficient design of pavement structures. If the pavement designs do not address the induced permanent strains on the pavement surface, the pavement layer would likely lead to deformation of the pavement surface. Further, it would result in billions of dollars for the rehabilitation of pavement annually (Muhanna et al. 1998; Puppala et al. 1999; Puppala et al. 2009).

Recently, significant research has been taken up to evaluate the test methods for assessing permanent deformation of soils (Monismith et al. 1975; Lentz and Baladi 1981; Ullditz 1993; Guo et al. 2006; Korkiala and Dawson 2007). Several laboratory experiments (triaxial, resonant column, simple shear, and hollow cylinder tests) have also been developed to simulate the pavement loading response on compacted soil samples. They reported that the influence of various factors such as material type and partial shape, compaction method, confining pressure, dry density, fine content, gradation size, dry density, moisture content, load duration, fine content, stress history on both M_R and ϵ_p behavioue of pavement materials. However, their influence on M_R was not same as on ϵ_p properties (Perez et al. 2006; Noolu et al. 2018; Ikeagwuani and Nwonu 2019, 2020; Rabab'ah et al. 2020). Hence, this part discusses the impact of the above-said factors on ϵ_p of pavement material.

Lekarp and Dawson (1998), argued that the accumulation of permanent strain was not a sudden process; reported that it is unnecessary to consider the static failure stage in finding the permanent deformation of the granular material. Aiban (2005), studied geotextile reinforced granular material behaviour and found out that the geotextile reinforcement does not show any significant effect in the decrement of permanent deformation above 200kPa stress level. Chahuan et al. (2008), initiated the study of permanent deformation behaviour of stabilized silty sand with fly ash and fiber. They reported that nearly 21% decrement in permanent strain with silty soil+30% fly ash stabilized with coir fiber is observed; whereas 18% percentage decrement is observed in permanent strain when stabilized with synthetic fiber. They also reported that the accumulation of plastic strain of soil (both stabilized and unstabilized soil) is directly proportional to the deviator stress level. It is also reported that at 100 repetitions fiber reinforcement decreases permanent strain by only 1.35% whereas for 10,000 cycles it was 21%. Yang et al. (2007), studied the long term behavior of cohesive subgrade soils and observed that the accumulation of permanent deformation was same from 10,000 cycles to 1, 00,000 cycles. Kumar and Singh (2008), Performed RLT tests to study the behavior of fiber-reinforced fly ash and soil–fly ash mixtures. They concluded that the resilient strain and permanent strain increased with an increase in load cycles application and decreased with confining pressure. Further, the incorporation of fiber reinforcement lowers the permanent strain from 3.82% of unreinforced to 1.91% when sample reinforced with 0.3% fiber at 1,000 load applications. Also, the results of permanent strain values obtained during repeated loading were shown to comply with simple exponential law. Mohanty (2009, 2011), studied the accumulation of permanent strains in on clayey subgrade material reinforced with randomly distributed fibers (coir and polypropylene) at optimum content. By conducting RLT tests. It is observed that the addition of fiber was able to reduce the permanent strain, and the decrease is more pronounced with coir fiber, irrespective of the presence of other parameters. Also, in another study, RLT tests were conducted on reconstituted pond ash specimens varying moisture content levels with varying dry unit weights (relative compaction of 90%, 95%, 97%), and at different stress levels under a range of initial effective confining pressure of 15, 25, and 35 kPa, by simulating the environmental and traffic conditions and reported the permanent deformation behaviour.

According to Cerni et al. (2012), if the fines have plasticity index > 10, it shows the drastic effect on permanent deformation of unbound granular material. The unbound granular material with calcareous fines (non-plastic) and silty clay fines under similar stress and moisture found that calcareous fines have less permanent strain than silty clay fines. Mishra et al. (2012), observed that 8% fines limit value for crushed aggregates whereas 4% fines are limiting for uncrushed gravel and stated that fines effect is more pronounced in permanent deformation than resilient modulus. Arulrajan et al. (2013), reported the performance of recycled construction and demolition (C&D) materials (Crushed Bricks, RCA, WR) at various moisture contents and stress levels. They said that most C&D materials perform satisfactorily about 70% of their OMC contents, and produce relatively smaller permanent strain and greater resilient modulus than commonly used granular subbase materials. Rahman et al. (2014), reported that incorporating both biaxial and triaxial geogrids showed a significant reduction in the range of 29 to 37% on permanent deformation of C&D materials. Abu-farakh et al. (2015), studied the plastic response of five different soils in Louisiana with cement and lime stabilization and concluded that cement stabilization was more effective for silty and sandy soils whereas lime stabilization was suitable for high plasticity soils. Kumar et al. (2016), performed a series of triaxial tests to evaluate the accumulation of plastic strain of unreinforced and fly ash-rise husk ash stabilized low plasticity clayey soil. They observed about 64% reduction in plastic strain when the soil is stabilized with fly ash, whereas 67% reduction is observed when the soil is stabilized with rice husk ash. One more observation noted that up to 1000 load cycles, the accumulation of permanent strain follows the same trend for both unstabilized and stabilized low plasticity clayey soil. Patel and Shahu (2018); Patel et al. (2016, 2019), reported the resilient characteristics of various waste materials to be used in pavement applications. Based on experimental findings, they reported the optimal mix of 75% steel slag and 25% fly ash with 15% dolime (SFD) which exhibit 60% higher M_R and 83% lower ϵ_p values than the conventional Wet Mix Macadam (WMM) for 28 days curing period. Further, they also noted the patterns of ϵ_p characteristics of SFD mixes as opposite to the M_R and UCS. Bhuvaneswari et al. (2018), studied the effect of lime (4, 6 and 8%) on clayey soils and observed the reduction in the plastic strain values, indicating the increased resistance of the material to permanent deformation. Georgees et al. (2018), Investigated the effect of polyacrylamide (PAM) on three types of granular materials (with an optimum content of 0.002% by dry weight of the soil based on UCS testing) to assess its engineering performance to simulated traffic. They conclude that PAM in the pavement subbase or select filling applications would lower rutting potential after long-term repeated loading. Puppala et al. (2009), Noolu et al. (2018, 2020), Addressed the influence of moisture content, applied stresses on the permanent deformations of all three soils including clay, silt, and sandy soils and stabilized soils. They concluded that the application of deviatoric stress has more influence on measured ϵ_p for all tested soils. Rabab'ah et al. (2020), evaluated the effect of curing on the permanent deformation behaviour of soil stabilized using by-product mill scale (MS) and cementitious materials (lime and cement); and concluded that soil stabilization with a combination of cement and lime demonstrated higher resistance to ϵ_p than lime stabilization alone

This observation indicates that for a complete characterization of the material. a need to evaluate the permanent deformation behaviour of pavement materials along with resilient modulus under various stress levels.

2.15 Existing Permanent Deformation Models

Many researchers have established constitutive relationships for pavement structures to estimate the long-term characteristics of pavement materials in terms of accumulated permanent strain in pavement systems (Sweere 1990; Barksdale 1972; Rada and Witczak 1981; Morgan 1966; Sharp 1985; Lekarp and Dawson 1998). In such model relationships, the state of stress applied and the number of load applications involved are substantial factors determining and predicting the gradual accumulation of plastic strains (Lekarp et al. 2000). Several researchers

(Morgan 1966; Barksdale 1972; Lekarp 1998) have documented the effect of permanent strain response with increased loading cycles. They confirmed that increases in loading cycles would continue to increase permanent strains.

In general, the models for predicting deformations are primarily divided into two categories: 1) Incremental models, which is based on elastoplastic theory, and it can precisely quantify the effects of stresses applied and paths on permanent deformation, but the complication and time-consuming nature often makes them difficult to implement. 2) Mechanistic-empirical models, which consume less time, predict results with greater accuracy even with fewer parameters. Thus it is commonly used in the design of pavements. While different researchers have proposed several empirical models, only some popular and widely used models are summarised below.

The first well-known prediction model is that proposed by Barksdale (1972), identified linear relationship and proposed the equation between permanent deformation and the load repetitions as

$$\epsilon_p = \ a + b \ logN \tag{2.12}$$

Later, Sweere (1990) performed a large number of cyclic triaxial tests on granular base materials using 10, 00,000 load repetitions and found semi-log model as they didn't agree regression equation which was proposed by Barksdale (1972) proposed another equation which is in log-log form as

$$\epsilon_p = \ a \ N^b \tag{2.13}$$

Wolff and Visser (1994), studied the long-term deformation behaviour of the material using heavy vehicle simulator (HVS) and differentiated the accumulation of plastic strain into two stages. In the first stage, quick development was seen in plastic deformation, but the deformation rate constantly decreased to 12, 00,000 repetitions. In the second stage, plastic strain development is prolonged, and there is no development in deformation rate. Log-Log model didn't satisfy the results, and hence below model was proposed to calculate the accumulated strain.

$$\epsilon_p = (cN + a).(1 - e^{-bN}) \tag{2.14}$$

Khedr (1985), observed permanent strain accumulation of crushed limestone using cyclic triaxial test and proposed an equation using the number of load cycles under variable confining pressure (VCP).

$$\frac{\epsilon_p}{N} = A_1 N^{-b} \tag{2.15}$$

To take into account the stress dependence in prediction models, Li and Selig (1996) evaluated the various influencing factors and obtained the following model:

$$\epsilon_p = a * \left(\frac{\sigma_d}{\sigma_s}\right)^{\alpha_9} * N^{b.} \tag{2.16}$$

However, Korkiala-Tanttu (2009) have performed similar research and proposed model as (5):

$$\epsilon_p = CN^b \frac{R}{1-R} \tag{2.17}$$

$$\text{b} = d\left(\frac{q}{q_f}\right)^{\cdot} + C \tag{2.18}$$

Where R = q/q_f is shear failure ratio; q_f = shear failure line in p-q space; d and c' are material parameters

Further, along with growth-type prediction models, a model was proposed by Paute et al. (1996) with a stabilization prediction value. This model depicted the role of stress levels and the number of load cycles on the accumulation of permanent strain:

$$\epsilon_p = \epsilon_{acc,100} + A\left[1 - \left(\frac{N}{100}\right)^{-B}\right] \tag{2.19}$$

Gidel (2001), conducted several cyclic triaxial tests on UGM by varying number of load cycles, stress rate, and proposed a model using maximum deviator stress and mean stress.

$$\epsilon_p(N) = \left[\left[1 - \left(\frac{N}{100}\right)^{-b}\right]\left(\frac{L_{max}}{p_a}\right)^u \left(m + \frac{s}{p_{max}} - \frac{q_{max}}{p_{max}}\right)\right] \tag{2.20}$$

$$L_{max} = \sqrt{p_{max}^2 + q_{max}^2}$$

A three-parameter model recommended by Ullditz (1993) to account for the influence of deviator stress:

$$\epsilon_p = \alpha_5 * \left(\frac{\sigma_d}{P_a}\right)^{\alpha_6} * N^{\alpha_7} \tag{2.21}$$

A three-parameter model proposed by Puppala et al. (1999) for subgrade soil:

$$\epsilon_p = \alpha_8 * \left(\frac{\sigma_{oct}}{P_a}\right)^{\alpha_9} * N^{\alpha_{10}} \tag{2.22}$$

By accounting the influence of octahedral shear ($\sigma_{oct/atm}$) stresses, normalized octahedral normal ($\tau_{oct/atm}$), and the load cycles Puppala et al. (2009) formulated a four-parameter permanent strain model

$$\epsilon_p = \alpha_1 * N^{\alpha_2} \left(\frac{\sigma_{oct}}{\sigma_{atm}}\right)^{\alpha_3} * \left(\frac{\tau_{oct}}{\sigma_{atm}}\right)^{\alpha_4 \cdot} \tag{2.23}$$

To predict the permanent strain response of a soil from its physical properties, a model was developed by Ullidtz (1993) and modified by Puppala et al. (1999) as

$$\epsilon_p = AN^{\alpha} \left(\frac{\sigma_{oct}}{\sigma_{atm}}\right)^{\beta \cdot} \tag{2.24}$$

Huurman (1997), used RLTT results from different types of sand to explain the relationship between ϵ_p and the number of cycles with the log-log approach

$$\epsilon_p = A * \left(\frac{N}{100}\right)^{b} + C \left(e^{D\frac{N}{100}} - 1\right) \tag{2.25}$$

Yang et al. (2008), performed cyclic triaxial tests on cohesive soils and derived a model based on the influence of stresses and load cycles with different moisture conditions w.r.t. M_R

$$\epsilon_p = A \times SL^b \times \left(\frac{M_R}{M_{R,i}}\right)^{c} \times N^D \tag{2.26}$$

Mohammad et al. (2006), studied both resilient and plastic deformation on treated as well as non-treated base material and correlated with the below equation.

$$M_R = 225\epsilon_p^{-0.25} \tag{2.27}$$

2.16 Summary of Literature

Based on the above-detailed literature survey, the literature review summary is listed below.

- With its intrinsic self-hardening properties, coal ash has many potential applications in civil practice. However, the self-hardening properties of coal ash depend on the amount of free lime present in it. In India, most power plants produce low lime fly ashes called class F ashes, and their application in pavements is not encouraged due to lack of adequate strength and durability. Hence, stabilization with suitable admixtures is a promising method for improving the properties.

- Soil, fly ash, or soil-fly ash mixtures with cementitious additives show improved mechanical properties and durability characteristics under adverse conditions due to the formation of cementitious products. However, such stabilization may lead to failure in a brittle manner, without showing significant plastic deformation behaviour and prone to fracture due to repeated wheel loading of vehicles. This would, therefore affect the safety and stability of the pavement structure. One way to improve this aspect is to include fibre reinforcement in the stabilized mixtures
- Fibre reinforcement in soil/ash has enhanced the improved tensile strength, coupled with the reduction in post-peak strength loss which is attained due to ductility induced by greater frictional and interlocking forces. Therefore, the addition of reinforcement has prevented the occurrence of sudden failure in pavement structures with the impact of wheel loading.
- Adding fibre to cemented soil/ash mixtures improved geomechanical strength, post-break load capacity, and changed the behaviour from brittle to ductile. Such composite mixtures present a relatively low-cost alternative solution for pavement constructions.
- Most of the experimental research reported so far is the mechanical behaviour of fibre-reinforced cemented soils. Very few laboratory investigations have been reported concerning the use of coal ash stabilized with both cementing agent and fibre inclusions for their use in pavements applications.
- Resilient modulus (M_R) and permanent deformation (ϵ_p) are generally important parameters, used in the mechanistic-empirical design of flexible pavements to characterize the non-linear response of pavement layers under repeated loading. The addition of admixtures to soil or other pavement materials could significantly increase M_R values and reduce deformation. These improvement rates are highly dependent on added admixture quality & content, curing time, state of stress levels (σ_d and σ_c) acting on the specimen.
- Although many researchers have reported the stabilized/modified coal ash application in as pavement structures, the studies to evaluate the stiffness characteristics (resilient and deformation behaviour) for various stress levels under repeated loading conditions are not well addressed.

CHAPTER – 3

MATERIALS AND METHODOLOGY

3.1 General

In order to accomplish the present research objectives, the basic and engineering properties, and resilient characteristics of both untreated and treated pond ash specimens are to be investigated thoroughly. Hence, this chapter covers the materials and methodology, including information about the materials used and its characteristics, sample preparation techniques, equipment/test setups used and testing procedures followed. The thrust of the experimental program includes specific gravity, particle distribution, X-ray fluorescence spectrometer (XRF), proctor compaction, unconfined compression, California bearing ratio, X-ray diffraction (XRD), scanning electron microscopy (SEM) and repeated load triaxial (RLT) tests.

3.2 Experimental Procedures

Procedures for carrying out the properties of the experimental materials are given below.

3.2.1 Specific Gravity

Specific gravity test was conducted using specific gravity bottle as per standard test method IS 2720 (Part 3), in which an average of three trials was reported.

3.2.2 Particle Size Distribution

The particle size distribution was conducted in accordance with IS 2720 (Part 4). The coarse fraction, i.e., the fraction larger than 75 µm, was analyzed by the dry sieving method. The fraction finer than 75 µm was collected by washing the samples through a No. 200 (75 µm) sieve. The collected samples were oven-dried and analyzed using the hydrometer method via sedimentation.

3.2.3 X-ray Fluorescence Spectrometer

Chemical compositions of test materials were analyzed using XRF spectroscopy technique. For the XRF study, Phillips PW 2404 X-ray fluorescence spectrometer was used.

Approximately 5 g of dried sample finer than 75 µm were put on glass holders and scanned for chemical composition.

3.2.4 Compaction

The compaction parameters of pond ash of various mix proportions were determined by following IS: 2720 (Part 7). The mould with a standard volume of 1000cc was used, and the material was compacted in three layers by giving 25 number of blows per layer. Standard hammer of 3.6 Kgs weight falling from a height of 36 cm was used for compaction, and the test was repeated with an increase in water content. Dry density was calculated for all water contents to obtain the compaction curve. The water content at maximum dry density (MDD) was considered as optimum moisture content (OMC).

3.2.5 Unconfined Compressive Strength (UCS)

The unconfined compressive strength test is a standard method of testing, to measure the resistance of a material against external load acting in a single axial way when the sample is not subjected to any confining stress. The test results UCS determines the relative response of treated material over a period of time (7, 28, 56 and 90 days). For this purpose, specimens of size 50 mm diameter and 100 mm height were prepared. The prepared specimens were then wrapped in an airtight plastic cover (vinyl bags) to minimize the loss of moisture under a controlled temperature of $25 \pm 2^{o}C$ and humidity, and placed in desiccators for respective curing periods. Before the testing time, samples were submerged in water for 4 hours for saturation to minimize the matric suction, and then UCS test was performed at a strain rate of 0.6 mm/min as per IS 4332-V (1970).

3.2.6 California Bearing Ratio (CBR)

California bearing ratio is essentially a penetration test to determine the strength of pavement layers. The test measures the load needed to penetrate the plunger of standard diameter into the specimen. The harder the surface, the higher the value of CBR. As per IRC 37, CBR parameter is used in the analysis and design of pavements to evaluate the suitability of the material for utilization in pavement layers. In this study, CBR specimens were prepared in the standard mould and cured in plastic bags at room temperature of $25 \pm 2^{o}C$ for 7 and 28 days. Consequently, the specimens were immersed in water for four days to study the soaking effect as it represents the worst possible scenario of pavement structures in the field conditions. After that, the CBR test was carried out at a strain rate of 1.2 mm/min as per IS 2720-16 (1987).

A surcharge of weight 25N was used throughout the testing. A metal plunger of 50mm in diameter and 100 mm in height was used for penetration purpose.

3.2.7 X-ray Diffraction (XRD)

X-ray powder diffraction is the most common method used to study the characteristics of crystalline structure and to determine the mineralogy of fine-grained soils. In the present work, powder X-ray diffraction technique was used to determine the mineral phases present in ash which caused by added additives during the curing process. X-ray diffraction analysis was carried out by using PANanalytical X-ray diffractometer. Representative samples obtained at the end of UCS tests were oven-dried for 24 hours. Then, the dried samples were prepared by manually grinding the specimen in a porcelain mortar and pestle to powder form and pressing the material lightly into rectangular glass holders, which were then scanned between two theta values of 6º to 70º with a step size of 0.02º. The X-Ray tube operated at 60 KV, and 55 MA is using an accelerator ultra-fast detector. Qualitative identification of minerals was performed using X'pert high score plus database software provided by the Joint Committee of Powder Diffraction Data Service (PCPDFWIN 1999).

3.2.8 Scanning Electron Microscopy (SEM)

SEM test is used to study the individual morphological characteristics (surface and shape) and to gain some insight into the behaviour of mixture. Elemental chemical composition was assessed with the help of energy dispersive analysis of X-ray. In this study, TESCAN VEGA 3LMU microscope with conventional tungsten heated cathode with live stereoscopic imaging using 3D beam technology was used to obtain high-resolution pictures of specimen microstructure. Before the scanning, the dried ash samples were mounted onto the double-sided carbon tape glued to the flat surface of SEM stub and then coated with a thin layer of gold for 120 s using the SC7610 magnetron sputter coater. The SEM images of the selected samples were taken, and the micrographs shown in the study reveals the typical microstructure of the specimen.

3.2.9 Repeated Load Triaxial (RLT)

The resilient characteristics of the pond ash were determined by an automated pneumatic cyclic triaxial apparatus with servo control and data acquisition system. Figure 3.1 shows the schematic figure of triaxial setup. In general, two-way repeated loading on specimen replicates principal stress reversal caused by compression and extension in the field condition; in contrast, one-way repeated loading only deals with compression (Pillai et al. 2011, Puppala et al. 2009). In the present study, one-way repeated loading was used to simulate the repeated loading on pavements.

Fig. 3.1. Repeated load triaxial test setup

3.2.9.1 Resilient Modulus (M_R):

Resilient modulus (M_R) is an elastic modulus based on recoverable strain under repeated loads. In simple words, it is the ratio of applied deviator stress to recoverable (resilient) strain. M_R is a stress-dependent measure. It indicates the elastic response of a material under different stress conditions; which replicates how a pavement system responds to various traffic wheel loads under various field conditions. M_R is typically determined through laboratory tests by measuring the stiffness of specimen subjected to a cyclic loading using RLT testing apparatus, as shown in Figure 3.1. For this, specimens of size 75 mm diameter and 150 mm height were prepared by compacting mix proportions in 8 layers with 25 blows/layer. A set of two specimens was prepared for each combination. For RLT testing, a triaxial pressure chamber was used to accommodate the sample, and water was used as a confining fluid to apply confining stress (σ_c) around the sample. The cyclic loads were applied as a haversine function with 0.1 sec loading time and 1.0 Hz loading frequency as it was decided on the basis of implementation of average traffic density in typical flexible roads. The test was conducted as per AASTHO T-307 protocol. During the test, the sample was subjected to repeated cyclic deviatoric (σ_d) and static confining stresses (σ_c). At first, the experiment begins with a conditioning phase by implementing 500 load repetitions at σ_d and σ_c of 103.4 kPa each, to minimize the imperfect contact between test specimen and sample cap. Subsequently, the test specimen is subjected to 15 various stress levels with 100 load repetitions each, as shown in Table 3.1.

Table 3.1 Loading sequence used in the present study for M_R (AASTHO T-307)

Sequence No	**Confining Stress,** σ_c	**Deviatoric Stress,** $\sigma_{d\,max}$	**Cyclic stress,** $\sigma_{d\,cyclic}$	**Contact stress,** $0.1 \times \sigma_{d\,max}$	**No of load cycles**
	(kPa)	(kPa)	(kPa)	(kPa)	N
0	103.4	103.4	93.1	10.3	500
1	20.6	20.6	18.6	2.1	100
2		41.3	37.3	4.1	100
3		62.1	55.9	6.2	100
4	34.4	34.4	31.0	3.5	100
5		68.9	62.0	6.9	100
6		103.4	93.1	10.3	100
7	68.9	68.9	62.0	6.9	100
8		137.9	124.1	13.8	100
9		206.8	186.1	20.7	100
10	103.4	68.9	62.0	6.9	100
11		103.4	93.1	10.3	100
12		206.8	186.1	20.7	100
13	137.9	103.4	93.1	10.3	100
14		137.9	124.1	13.8	100
15		275.8	248.2	27.6	100

Finally, the modulus at each stress level was calculated by taking an average value of moduli of the last five cycles for each sequence using equation (3.1).

$$M_R = \left(\frac{\sigma_d}{\epsilon_r}\right) \tag{3.1}$$

Where M_R = Resilient modulus;

σ_d and ϵ_r = Deviatoric stress and resilient deformation at a given load pulse.

Here, σ_c represents the overburden pressure on the specimen within the pavement layer. The axial σ_d is generally composed of two components, cyclic stress ($\sigma_{d\ cyclic}$) and contact stress, of which cyclic stress is the applied deviatoric stress and contact stress represents the seating load which was applied by placing a vertical load on the sample to maintain positive contact between the specimen cap and the specimen. The contact stress is 10% of total axial stress.

3.2.9.2 Permanent Deformation/strain (ϵ_p)

Permanent deformation (ϵ_p) is a crucial factor in assessing the long-term behaviour and failure of the pavements (Patel et al. 2019; Puppala et al. 2011). In the field condition, the accumulation of deformations in pavement layers depends on stress state and load repetitions of wheel movements. As per FHWA NHI 05-037 (Christopher 2006), there were no standard specifications established for determining permanent deformation of base/subbase layers based on RLT testing and suggested to consider first 500-1000 preconditioning load cycles of resilient modulus testing for ϵ_p calculations. However, relying on such fewer load cycles and single stress level to find ϵ_p behaviour of specimens may not represent the actual field behaviour of pavements. Further, the selection of stress levels for deformation analysis mainly depends on the type of material and their depth under the pavement surface (Pidwerbesky 2004). Hence, researchers (Patel et al. 2018, 2019, Arulrajah et al. 2013) have performed RLT tests of permanent deformation, which involve applying various repeated deviatoric stress levels at static confining stress around the test specimen.

Further, NCHRP 1-28 A (2004) determined stress condition of pavement layers based on stress analysis conducted to compute a field representation and stated that the base/ subbase materials of any typical flexible pavements are likely to experience a stress level σ_c of 34.5 kPa and σ_d of 103.4 kPa. Based on this stress condition, to investigate further the effect of expected wheel loads on ϵ_p, the deviatoric stresses of 100, 200 and 300 kPa were considered under multi loading stages for each test specimen. These deviatoric stresses also represent the effective range of the deviatoric stress levels (103.4 kPa to 275.8 kPa) considered in finding the M_R of the specimen; which are expected to closely reflect the actual stress occurrence in the subbase layers of pavement (Pidwerbesky 2004). Therefore, in this study, RLT tests for ϵ_p were performed at σ_d of 100, 200, and 300 kPa with static σ_c of 34.5 kPa for 3000, 3000 and 4000 number load cycles to a total of 10000 load repetitions applied in 1, 2 and 3 stages respectively. Procedure for the preparation of specimen is the same as that of resilient modulus test. The deformations were assessed using a linear variable differential transducer (LVDT) with a precision of 0.01 mm. A load cell with a capacity of 5kN was used to measure the load acting on the specimen. In the present study, the all the RLT testing specimen were maintained at a temperature range of 27 ± 2^0 C from preparation stage to testing stage.

3.3 Materials

3.3.1 Pond ash (P)

Coal ash, referred to as pond ash (P) in this study was collected by excavating the top surface ash pond to a depth of 0.5m at Kakatiya thermal power plant (Latitude: 18.3832° N & Longitude: 79.8260° E), Telangana, India. The colour of the pond ash was observed as grey (Fig. 3.2). The basic properties and chemical characteristics of pond ash are presented in Table 3.2, and the particle size distribution curve is shown in Fig. 3.3. Pond ash was characterized for its mineralogy and morphological microstructure by XRD and SEM studies (Fig. 3.4, Fig. 3.5). From X-ray diffraction pattern, it can be seen that pond ash mainly consists of quartz (SiO_2), mullite (3 Al_2O_3 2 SiO_2) along with a peak pertaining to magnetite spinel (Fe_2O_3, Fe_3O_4). The percentage of lime present in pond ash is less than 15%; therefore, as per ASTM C 618-89 specifications, it can be categorized into Class-F ashes. Fig. 3.5 shows the SEM image of pond ash, which reveals that the ash particles used in the study are composed of spherical in shape, irregular texture with the presence of pore structure on its surface.

Fig. 3.2. Pond ash used in the study

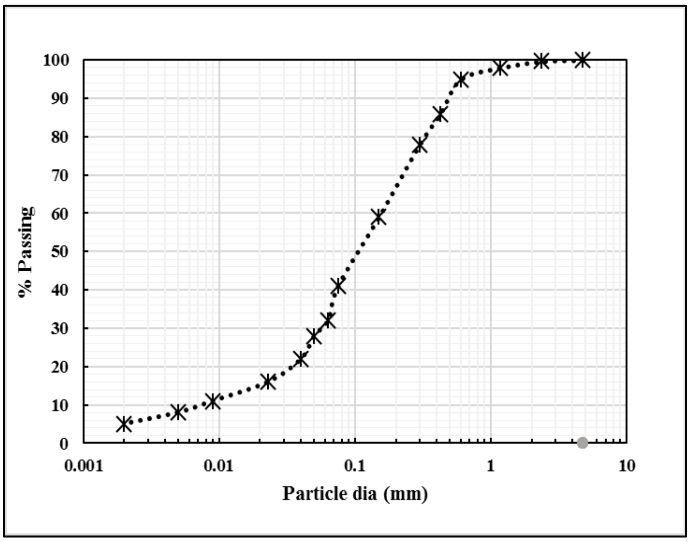

Fig. 3.3. Particle size distribution of pond ash

Table 3.2. Physical and chemical properties of pond ash

Property	Value	Major Compounds	% by mass
Specific gravity	1.93	SiO_2	62.1
Plasticity index	Non-Plastic	Al_2O_3	13.6
Grain size distribution		Fe_2O_3	2.56
i) % Gravel	0	SiO_2+ Al_2O_3+ Fe_2O_3	78.26
ii) % Sand	65	CaO	1.2
iii) % Fines	35	SO_3	0.25
Group symbol	SM	LOI (loss on ignition)	11.43
Maximum dry density (kN/m^3)	11.21	Others	8.86
Optimum moisture content (%)	34.02		
Angle of internal friction(φ°)	32.1		
CBR (%)			
i) Unsoaked	21.3		
ii) Soaked	4.2		

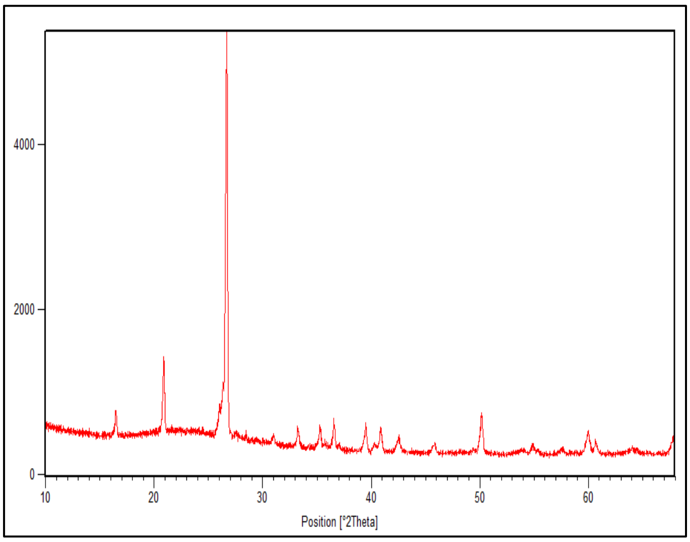

Fig. 3.4. XRD pattern of pond ash

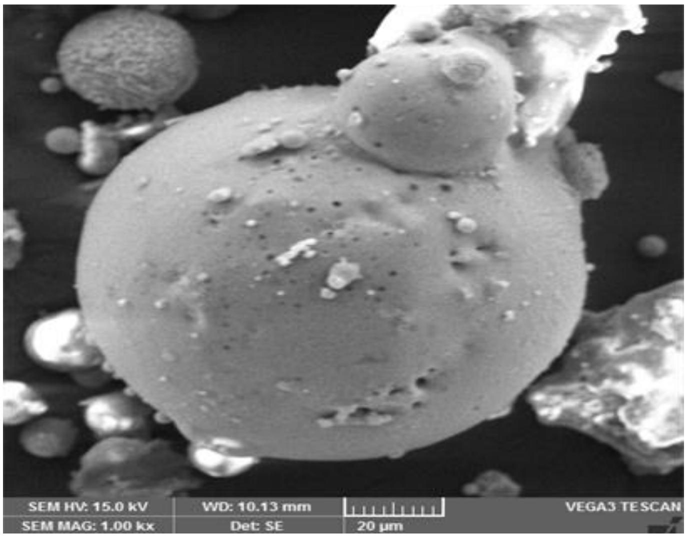

Fig. 3.5. SEM Photography of pond ash

3.3.2 Lime (L)

The hydrated lime ($CaOH_2$) is more commonly used stabilizing agent with appropriate precautions in most of the geotechnical applications. In the present study, commercially available lime (purity of 72.13%), supplied by Super Lime Traders, Hyderabad, India is used to modify pond ash (Figure 3.6). Before use, lime was sieved through 600µ to remove impurities any present in it. The chemical composition of the lime on dry weight basis is: SiO_2 = 4.5%; Al_2O_3 = 4.63%; Fe_2O_3 =2.3%; MgO = 9.2%; CaO = 72.13% and others = 7.24%. The SEM image of lime is shown in Figure 3.7.

Fig. 3.6. Lime used in the study

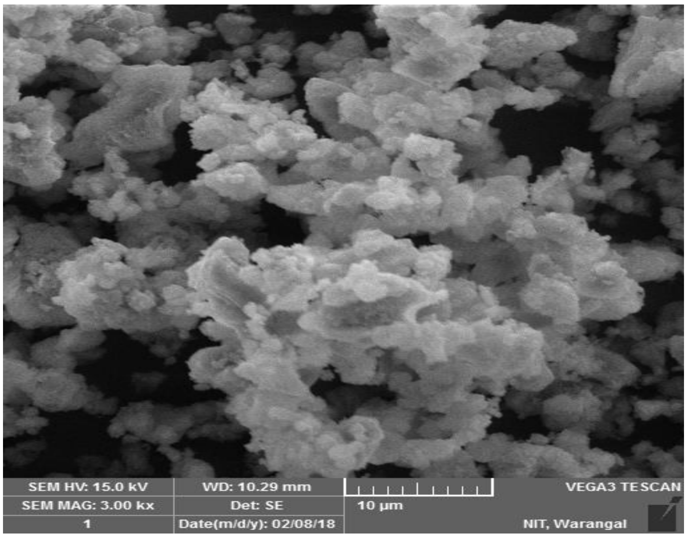

Fig. 3.7. SEM Photograph of Lime

3.3.3 Polypropylene Fiber (F)

In general, polypropylene fiber has distinct technical features like lightweight, strong, flexible, excellent chemical resistance to acids and alkalis, hydrophobic in nature, high abrasion resistance, and has low thermal conductivity. It also has low moisture absorption. Compared to other synthetic fibers, PP fiber is easy to process and inexpensive. In the present study, fibers used are monofilament/fibrillated type (Figure 3.8) with a diameter of 0.2 mm and length of 12 mm. Its physical properties supplied by the manufacturer Nina Concrete System Pvt. Ltd, Hyderabad, India are presented in Table 3.3.

Fig. 3.8. Polypropylene fiber

Table 3.3. Properties of Polypropylene fiber

Properties	**Value**
Colour	white
Length, (mm)	12
Modulus of elasticity (MPa)	4000
Tensile strength (MPa)	450
Specific gravity	0.91
Melting point (^{0}C)	>250
Diameter (mm)	0.2
Aspect Ratio	60

In the present study, for characterization purpose, distilled water was used, and for preparation of the specimen, normal tap water was used.

3.4 Additive Contents and Specimen Preparation

For the preparation of test specimens, the notations considered in the study are: Untreated pond ash sample (P), Lime-treated pond ash (PL), Fiber-reinforced pond ash (PF), and Fiber-Lime treated pond ash (PLF).

Before selecting and preparing test specimens with lime, based on the methodology proposed by Rogers et al. 1997, pH tests were conducted on several pond ash-lime mix proportions to define minimum lime content. From the results obtained, the ideal peak constant values (pH = 12.3) were observed from 4% lime, and it is considered the initial lime addition to the ash material. Besides, to study the effect of variation in lime contents of 6%, 8%, 10% and 12% were considered in the study.

For fiber reinforcement, four different fiber contents on a dry weight basis of pond ash were selected. Most of the previous studies also reported the optimum fiber contents varying from 1.0% to 1.5% for the effective performance of reinforced specimen. Accordingly, the fiber contents of 0.5%, 1.0%, 1.5% and 2.0% were considered in this study.

During the preparation of test specimens, the required quantity of raw materials in the dry state was measured and mixed thoroughly until a uniform colour was obtained in the dry mixture. Then, the necessary amount of water was added to the composite mix. All the mixing process was done manually. During the mixing process, the fibers tend to come together to form a lump; hence, it requires a due care to separate fibers and to ensure a uniform distribution of fibers in the mixture.

3.5 Methodology

This study divided the total experimental program into five phases (Figure 3.9). The first phase concerns the physical and engineering properties of materials used (i.e. pond ash, lime, and fiber). The second phase deals with the compaction and strength characteristics (UCS, CBR) of both untreated and treated pond ash; and studies the effect of additives addition, curing period and comparison of the test results with IRC specifications. The third phase deals with the resilient modulus characteristics (M_R) of untreated and treated pond ash. The effect of lime, fiber, lime-fiber, confining and deviatoric stresses on M_R were examined. Further, statistical regression analysis studies on selected models were performed with experimental M_R values. Similarly, the fourth phase deals with the permanent deformation characteristics (ϵ_p) of untreated and treated pond ash. The effect of lime, fiber, lime-fiber, deviatoric stresses and load cycles on ϵ_p were investigated. Further statistical regression analysis studies on selected models were performed with experimental ϵ_p values. Phase five deals with pavement design analysis by thickness optimization using KENLAYER software, and the same is carried out for economic assessment.

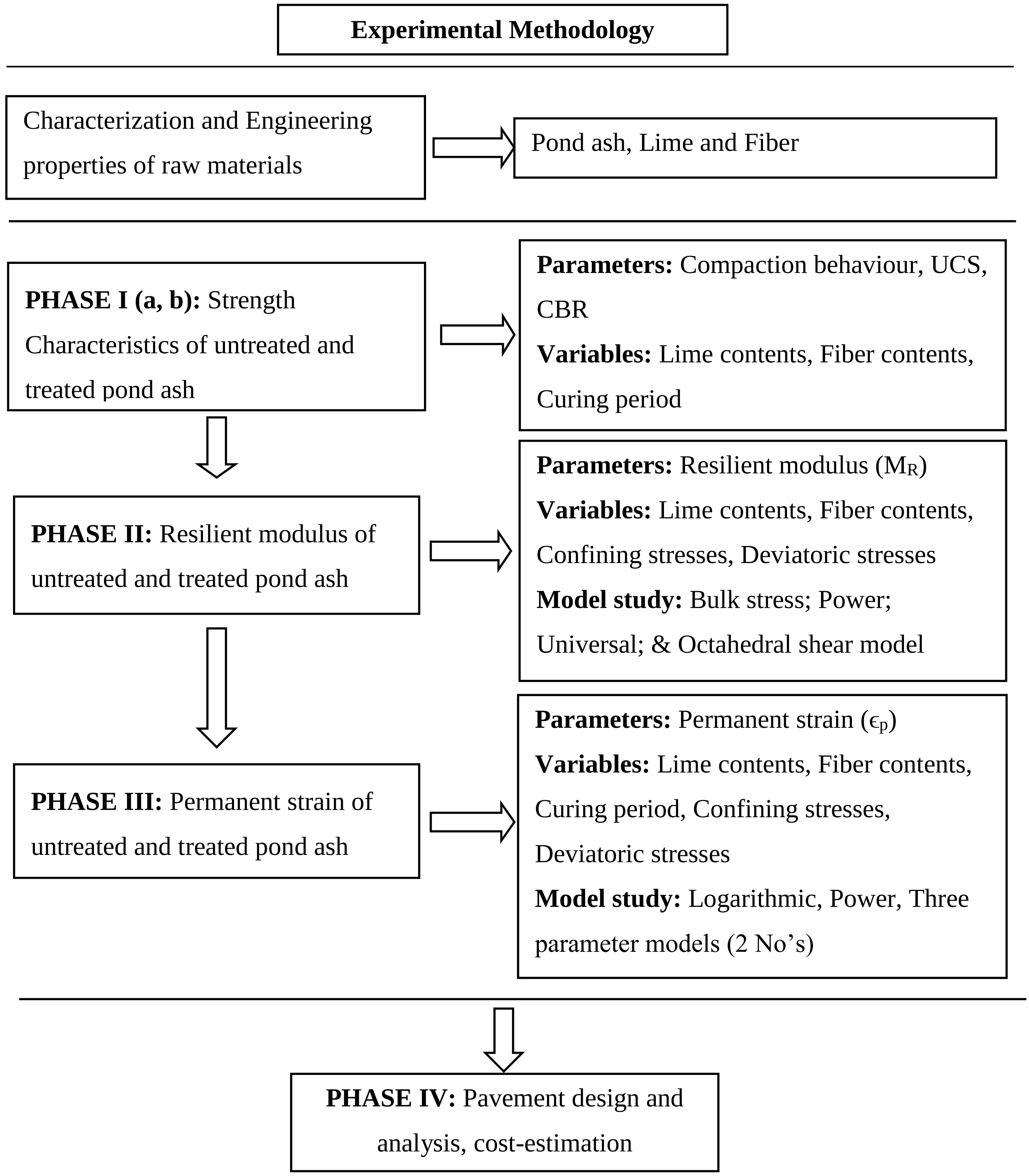

Fig. 3.9. Experimental Methodology

3.6 Summary

This chapter provides a detailed description of the basic properties of the materials used. Also, the details of the experiments conducted on the samples and the methodology followed are briefly explained.

CHAPTER – 4

COMPACTION AND STRENGTH CHARACTERISTICS OF COAL ASH

4.1 General

This chapter describes the impact of lime and fiber inclusion on the compaction and strength characteristics (UCS and CBR) of pond ash in various proportions. The test results presented in tables and figures are discussed in the following subsections.

4.2 Compaction behaviour

The compaction results (MDD and OMC) obtained by standard compaction tests for pond ash treated with lime and fiber in various mix proportions are presented in Figure. 4.1 and Figure. 4.2.

4.2.1 Effect of Lime on Compaction Parameters

The MDD and OMC of compacted pond ash observed as 11.21 kN/m^3 and 34.02%, respectively. These MDD and OMC values are well within ranges of Indian coal ashes (Mohanthy and Patra 2015; Prakashan and Sridharan 2007). The MDD of pond ash mixed with lime increased from 11.12 kN/m^3 to 12.71 kN/m^3 with the increment in lime content from 4% to 12%, whereas the corresponding OMC decreased from 34.02% to 30.29%. The reason for an increase in MDD could be attributed to the lime with better plasticity at OMC, facilitates the rearrangement of pond ash particles in a better way, and resulting in an increased MDD (Sreedhar et al. 2011). Similarly, the decrease in OMC of pond ash with lime addition is likely due to reduced water ingress into cavities caused by sealing of cavities present on the surface of pond ash (Bera et al. 2007; Ghosh and Subbarao 2007; Ghosh 2009).

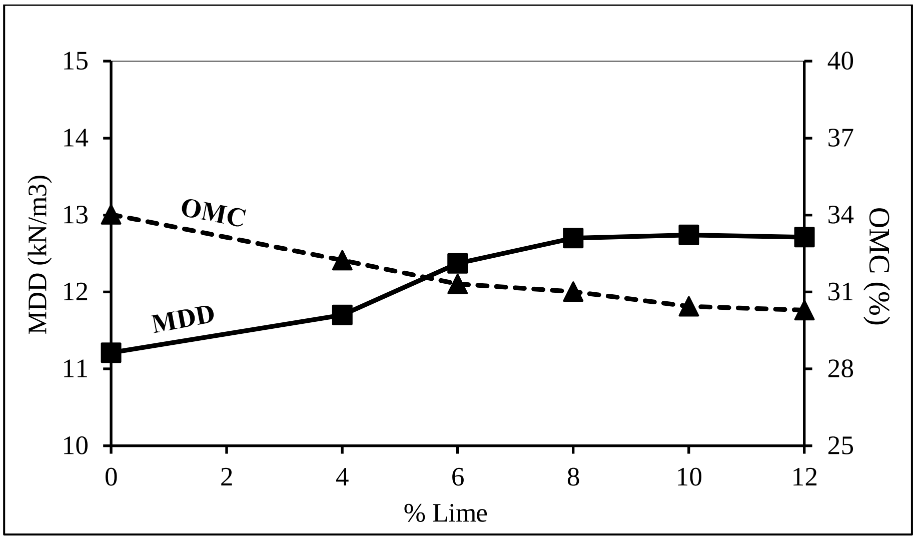

Fig: 4.1 Compaction results of lime treated pond ash

4.2.2 Effect of Fiber Inclusion on Compaction Behaviour

The compaction behaviour of pond ash reinforced with fiber is shown in Figure. 4.2. The Figure clearly shows that compared to untreated pond ash, the inclusion of fibers does not show the ample changes in compaction behaviour of pond ash. Though a marginal increase in MDD (up to 3.5%), and a slight decrease in OMC (up to 3%) was observed with an increased fibre content from 0.5% to 2.0%; these variations could be owing to the presence of fibers in pond ash. Similar results were reported in previous studies for various fibre-reinforced cohesionless soils (Kaniraja and Gayathri 2003; Sreedhar et al. 2011; Singh and Sharan, 2014).

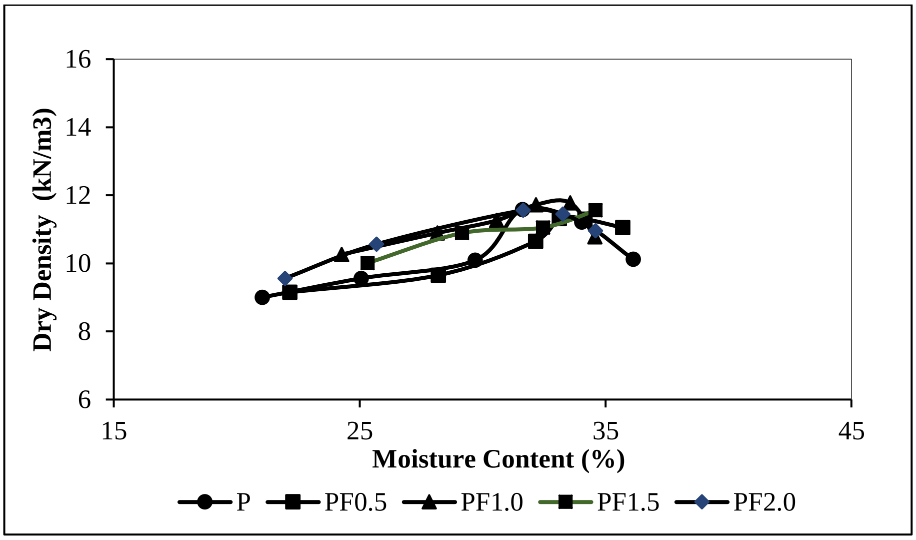

Fig. 4.2. Compaction behaviour of pond ash-fibre mixes

4.3 Effect of Lime on UCS of Pond ash

The variation in UCS of lime treated samples at different curing periods is shown in Figure. 4.3. As reported in many previous studies, due to the absence of cohesion, the UCS value of untreated class F coal ashes is very low (assumed to be zero). Hence, it may be considered the control value for the study. Figure 4.3 shows that the UCS of pond ash linearly increased with increased lime content for all curing periods. The variation in UCS of pond ash is 110 kPa to 389 kPa, 236 kPa to 1136 kPa, 526 kPa to 3012 kPa, 728 kPa to 4216 kPa at 7, 28, 56 and 90 days curing for lime contents of 4% to 12%, respectively. This increase in UCS of pond ash is caused by the pozzolanic reaction due to the formation of cementitious products between added lime and silica-alumina present in pond ash (Samanta 2018; Sahu et al. 2017). Figure 4.4 shows that the untreated pond ash is initially round in shape, with a little rough surface, absence of hydration products on the surface. After addition of lime, the formation of cementitious compounds (CSH, CASH) on the surface of pond ash can be observed; and these gels cause to bind the adjacent ash particles firmly and enhance its strength. However, the improvement in UCS of pond ash at 4% and 6% were observed to be low; which is due to the insufficient lime availability for pozzolanic reaction development, and the amount of lime added primarily is mostly utilized in initial colloidal type reactions (Sivapullaih et al. 2000). Figure. 4.4, shows that the disjointed particles on the surface of pond ash for 4% and 6% lime contents through the SEM images indicating a lower amount of hydration/pozzolanic products formation. It is observed that the UCS increased with increase in lime content from 6% to 8%, especially at longer curing times (> 28 days). This indicates that the slow rate of pozzolanic reaction between pond ash and lime during the initial days gradually accelerates along with the curing period and reaches the equilibrium state of ultimate strength at more extended curing periods (Chand and Subbarao 2007). In Figure. 4.3, such substantial improvement can be observed between curing periods of 28-56 days. Also, Figure. 4.5 shows the rate of change of pozzolanic activity concerning time for 8% lime treated pond ash. The SEM images of specimens with lime content more than 6% show a higher amount of gel formations in a dense continuous form on the surface of pond ash particles. It resulted in more bonding nature between the particles leading to significant improvement (Figure. 4.4). However, the increase of lime content from 8% to 10% and 12%, the relative increase in strength is observed to be marginal. The possible reason for the lower gain rate is the non-availability of reactive silica for excess added lime to form more cementitious products (Pani and Singh 2017). Also, lime has no appreciable friction or cohesion; thus, excess lime in the specimen can decrease strength (Bell 1996).

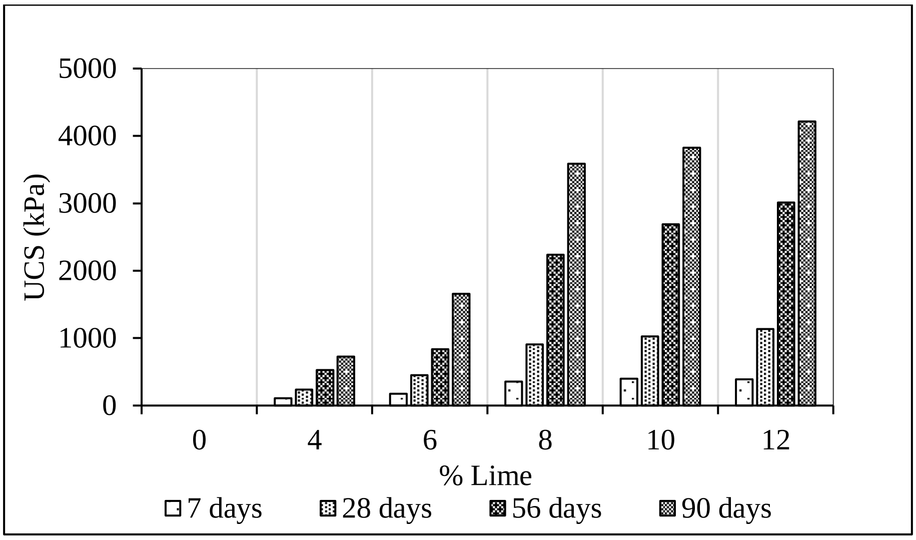

Fig. 4.3. Variation in UCS of lime treated pond ash at different curing periods

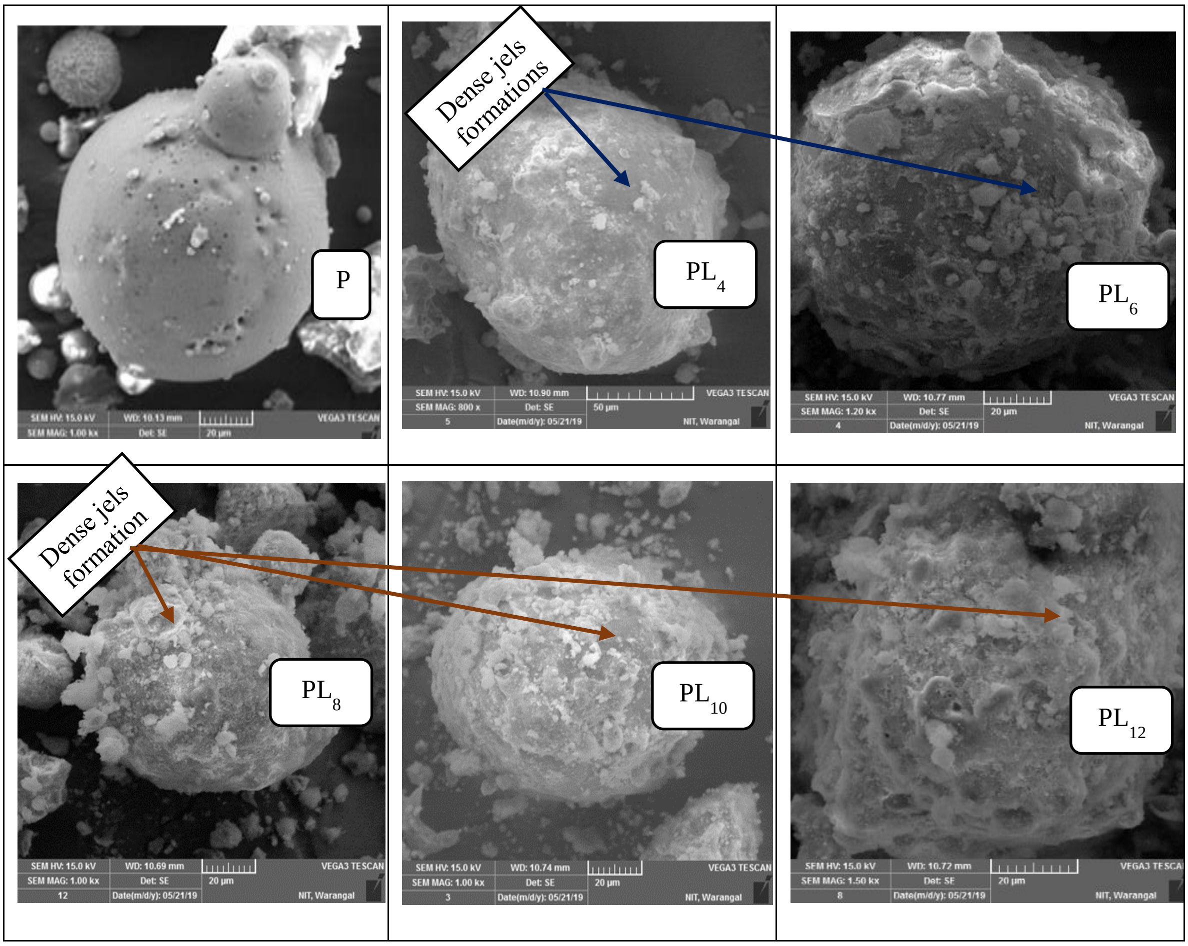

Fig. 4.4. SEM images of pond ash with different lime contents at 28 days curing

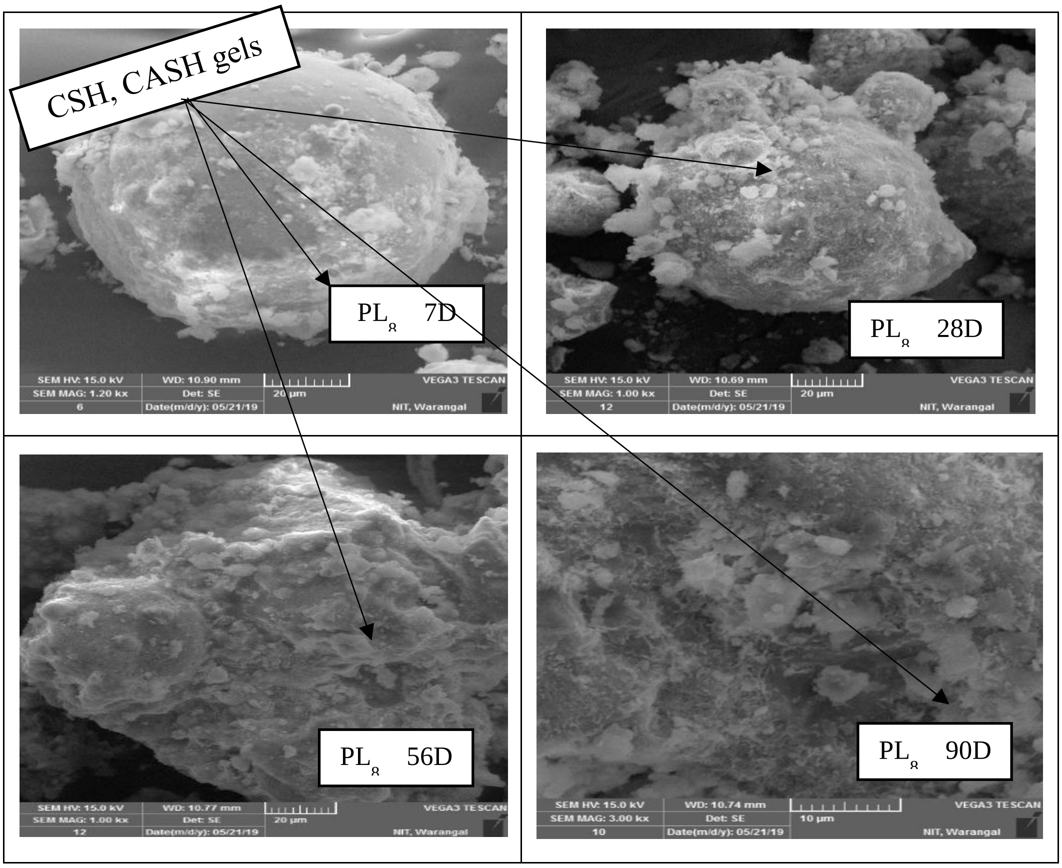

Fig. 4.5. SEM images of pond ash with 8% lime content at various curing days

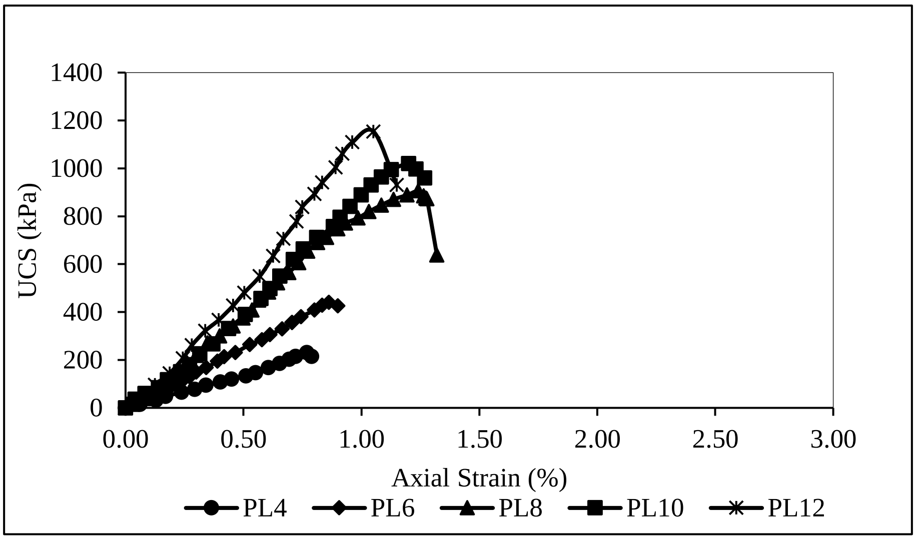

Fig. 4.6. Stress-strain behaviour of lime treated-pond ash (28 days)

The stress-strain curves of lime treated pond ash at 28 days curing period is presented in Figure. 4.6. The stress-strain curves are similar for lime treated specimens at curing periods of 7, 56 and 90 days. It is understood that the peak stress and stiffness of pond ash increased significantly with an increase of lime content. However, after attaining peak stress, the specimen failed suddenly indicating a substantial loss of energy in immediate strength drop. Also, the failure strain range of the tested specimens was observed as low failure strain of 0.75% to 1.2%, indicating brittle nature of failure. Figure. 4.7 shows the testing of lime treated sample during the UCS test.

Fig. 4.7. UCS sample during the test

4.4 California Bearing Ratio

4.4.1 Effect of Lime on CBR of Pond ash

The CBR test is one of the most common indentation tests in highway-geotechnics as it guides the design of pavement layers based on strength criteria (Ghosh 2009). The complete inundated condition of pavement layers is the worst possible scenario in the field, and it is represented with the soaked condition in the laboratory (Figure. 4.8). Upon soaking, ingress of water takes place, which significantly reduces the CBR to a considerable extent (Sridharan and Prakashan 2007).

Table 4.1. Soaked CBR values of lime treated pond ash

Mix	7 days	28 days
P	4.2	4.2
PL_4	15.1	23.6
PL_6	21.2	32.6
PL_8	27.2	39.7
PL_{10}	29.6	43.1

PL_{12}	28.1	45

The CBR test results of both untreated and treated pond ash are presented in Table 4.1 and Table 4.2. It is noticed that the CBR of pond ash is 4.2. The addition of lime and fiber increased the CBR values of pond ash continuously with increased additive content.

Compared to untreated compacted pond ash (P), the CBR gain rate for lime treated specimens varies from 2.60 to 6.05 and 3.07 to 6.76 times higher for 7 and 28 days curing respectively. This improvement in CBR is attributed to the formation of cementitious products (C-S-H and C-A-S-H) due to pozzolanic reactions which aid in binding the particles effectively, thus resulting in higher CBR (Suthar and Aggarwal 2018). Higher the lime content, higher is the generation of cementitious compounds, resulting in higher bearing ratio. Further, the CBR values of 28 days cured specimens were observed to be 30-60% higher than seven days cured specimens. Based on the CBR values it may be stated that except PL_4, remaining all lime treated specimens had reached the capacity equivalent to that of silty sand (20%-40%) or sand/gravelly sand (20%-50%) (Naganathan et al. 2012; Schaefer et al. 2008).

Fig. 4.8. CBR test of pond ash specimens

4.4.2 Effect of Fiber on CBR of Pond ash

The load-penetration behaviour of pond ash reinforced with fibes of various proportions are shown in Figure 4.9, and the corresponding soaked CBR values are shown in Table 4.2. From Figure 4.9, it can be observed that the load values have been increased with an increase in penetration and reach its maximum at a certain point. The load values tend to decrease after reaching a maximum, in case of unreinforced pond ash exhibiting typical strain-softening behaviour. However, the load remains constant or slightly increases in fibre reinforced pond ash, especially at the higher fibre content. Similar observations were made by (Refeai and Suhaibani 1998) for sands reinforced with polypropylene fibres.

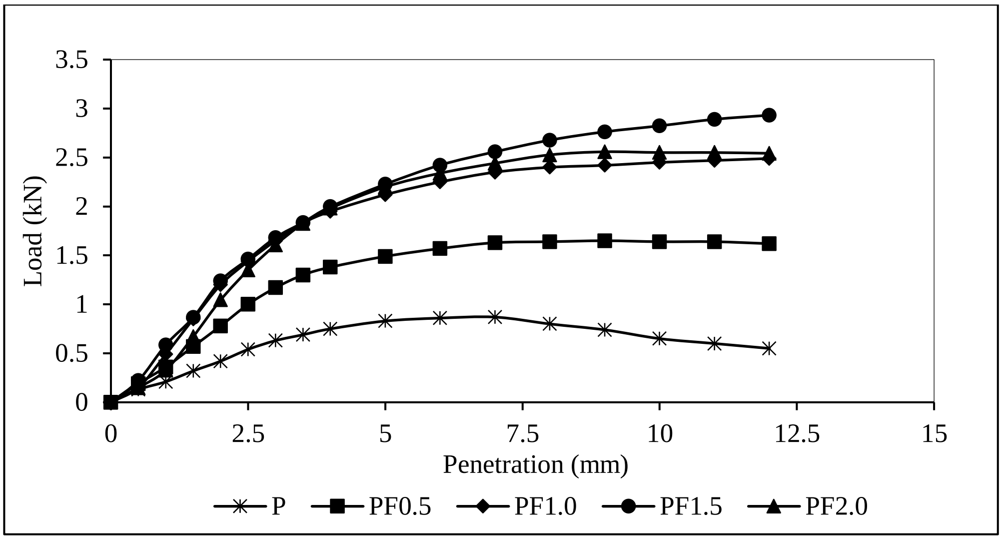

Fig. 4.9. CBR load-penetration curves for pond ash-fiber specimens

Table 4.2. Soaked CBR values of pond ash-lime mixes

% Fibre	Soaked CBR	Improvement w.r.t. unreinforced Pond ash (%)	Rate of gain in CBR (%)
0	4.2	-	-
0.5	7.3	73.8	-
1	10.52	150.5	104
1.5	12.56	199	32.2
2	13.16	213.3	7.2

Table 4.2 shows the variation in increased CBR values of pond ash with an increase in fibre content, and the values are improved by 0.73 to 2.13 times compared to compacted pond ash. This is because of the enhanced interaction mechanism of incorporated fibers with pond ash particles through the surface frictional bond and interlocking forces. The function of bond or interlock is to transfer the stress from pond ash to the fiber inclusions by mobilizing the tensile strength of fibers. Thus, fibre reinforcement works as frictional and tension resistance element, which leads to an increased load-carrying behaviour of composite (Li et al. 2014). It

is also observed that the gain rate in CBR was significant up to 1.0% fibre (104%), above that the gain rate is decreased (i.e., 32.2% for 1.5% fiber and 7.2% for 2.0% fiber). This is because, beyond 1.0%, the quantity of fiber is high enough to affect more fibre-fibre interaction than fibre-pond ash interaction, resulting in the formation of slippery surface (Li et al. 2014; Sreedhar et al. 2011).

As IRC: 89-2010 and IRC 37-2012 specifications recommends a minimum UCS of 750 kPa and CBR of 30% required for sub-base construction with stabilized/modified pavement materials. It is observed that pond ash with 8% lime meets the above requirements, and hence 8% lime is considered an optimum content for pond ash.

The fiber-reinforced pond ash show exhibited the enhanced CBR of pond ash (maximum 10.52% for $PF_{1.0}$). However, the required CBR is still higher than the attained CBR; hence, its application in the subbase layer may not be suitable.

Further, in the event of repeated wheel loading of vehicles, safety and stability of the pavement structures may be affected by its brittleness and prone to fracture (Tang et al. 2007; Chauhan et al. 2008). One way to improve this aspect is to include fiber reinforcement in the stabilized composites. The inclusion of reinforcement would prevent the sudden failure of the pavement structures due to the impact of wheel loads and increase its serviceability (Sarkar and Dawson 2015). In this connection, based on the strength results of pond ash, the influence of both lime and fiber on pond ash at their optimum contents levels by mixing succeeding additive in different proportions.

The combined effect of lime and fiber on compaction, strength, and resilient characteristics of pond ash is studied in the following sections.

The nomenclature of Pond ash-Lime-Fiber (PLF) samples prepared are shown in Table 4.3. The lime-fiber treated specimens are designated as PL_XF_Y, where P denoted pond ash, L means lime, and the subscript X indicates lime content in PLF mix, F represents fiber, and subscript Y indicates fiber content in PLF mix.

4.5 Effect of Lime and Fiber on Compaction Behaviour of Pond ash

The compaction results (MDD and OMC) of pond ash treated with both lime and fiber ate presented in Figure 4.10. It can be seen that compared to PL_8 specimen, the MDD of corresponding PLF mixes increased by 0.4% to 2.2 %, and OMC increased by 0.7% to 4.8%.

This increase in the OMC is attributed to the absorption of water slightly by polypropylene fiber resulting in marginal changes in compaction parameters of lime treated specimens. Similar observations were reported in previous studies by Setty and Rao, 1987 for silty sand mixed with polypropylene fiber, and Chore and Vaidya, 2015 for fly ash with cement and polypropylene fiber. However, this behaviour is somewhat different from the results reported by Kaniraj and Gayatri (2001, 2003) for fly ash-soil-cement-fiber mixtures (Singh and Kumar 2019) for MSWI bottom ash-cement-fiber composites. In the present study, however, for simplicity purpose, PL_XF_Y samples were prepared at MDD and OMC of corresponding PL mixtures.

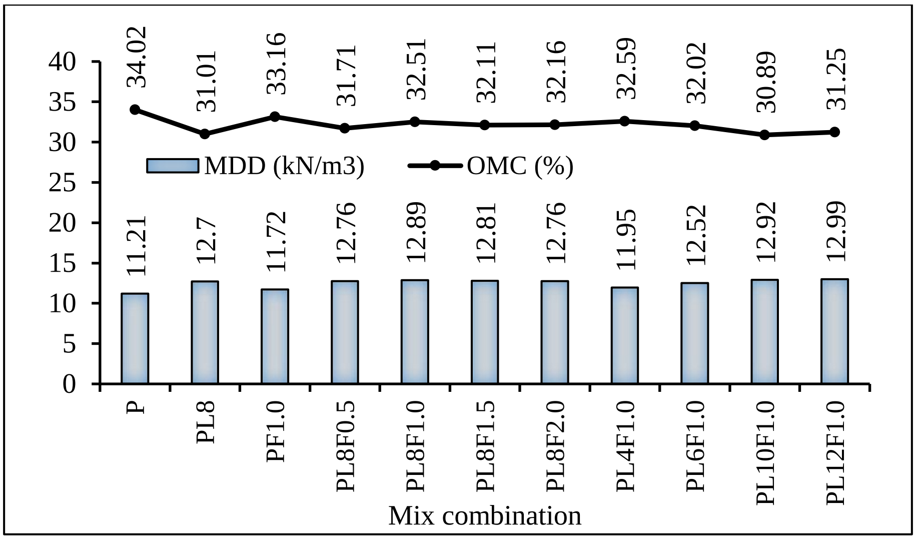

Fig. 4.10. Compaction results of PLF specimens at various proportions

4.6 Effect of Lime and Fiber on UCS of Pond ash

The combined effect of lime and fiber on UCS of pond ash cured for 7 and 28 are presented in Figure 4.11. It is observed that the inclusion of fibers in lime treated specimens increased the UCS values furthermore at both curing periods. For instance, with the inclusion of 1.0% fiber in PL_8, UCS values increased from 356 kPa to 562 kPa for 7 days and 906 kPa to 1612 kPa for 28 days, respectively. This is due to the formation of the cementitious product by pozzolanic nature with lime addition, which binds the fiber inclusions and ash particles together and provides a compact matrix structure resistance (Figure 4.12). These formations increase the effective contact area and interlocking forces that allow mobilized frictional bonding to transfer stresses from ash to fibers by exhibiting ductile behaviour with enhanced load (Park 2011; Dhar and Hussain 2018; Syed et al. 2019). However, at low lime-fiber combinations (i.e., L_4, L_6 and $F_{0.5}$) the increased rate in UCS showed less than the required UCS of 750 KPa. There is a substantial increment up to $PL_8F_{1.0}$, later which the observed increase only marginal increase or decrease (in some cases). The cause for a decrease in gain rate might be attributed to the deficiency in contact between ash particle and fiber due to increased fiber content after 1% which results in the formation of lumps, tend to reduce the friction coefficient and thus decreases UCS (Dehghan and Hamidi 2016; Wei et al. 2018). In addition to that the higher the lime content in composite contributed to development of micro-cracks, which leads to a decrease in the efficiency of matrix structure (Wang et al. 2019).

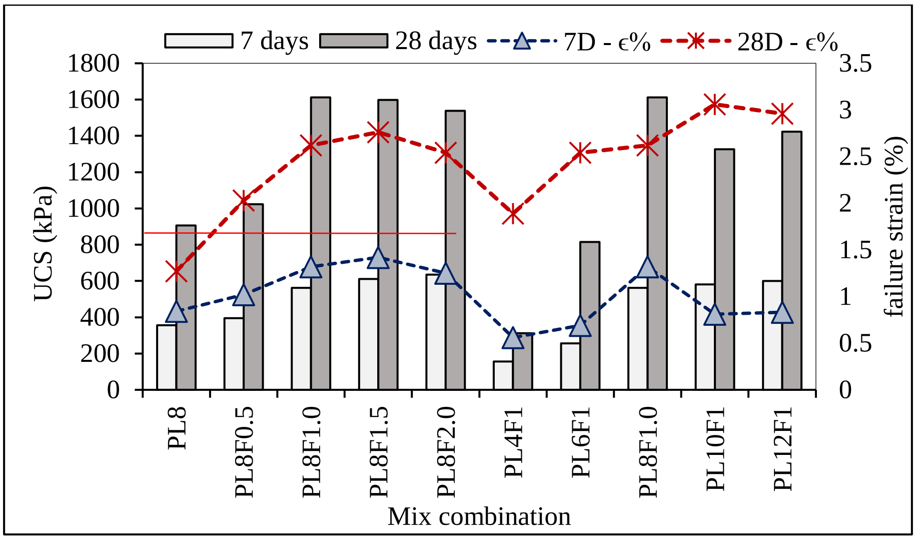

Fig. 4.11. UCS results of pond ash at various mix proportions

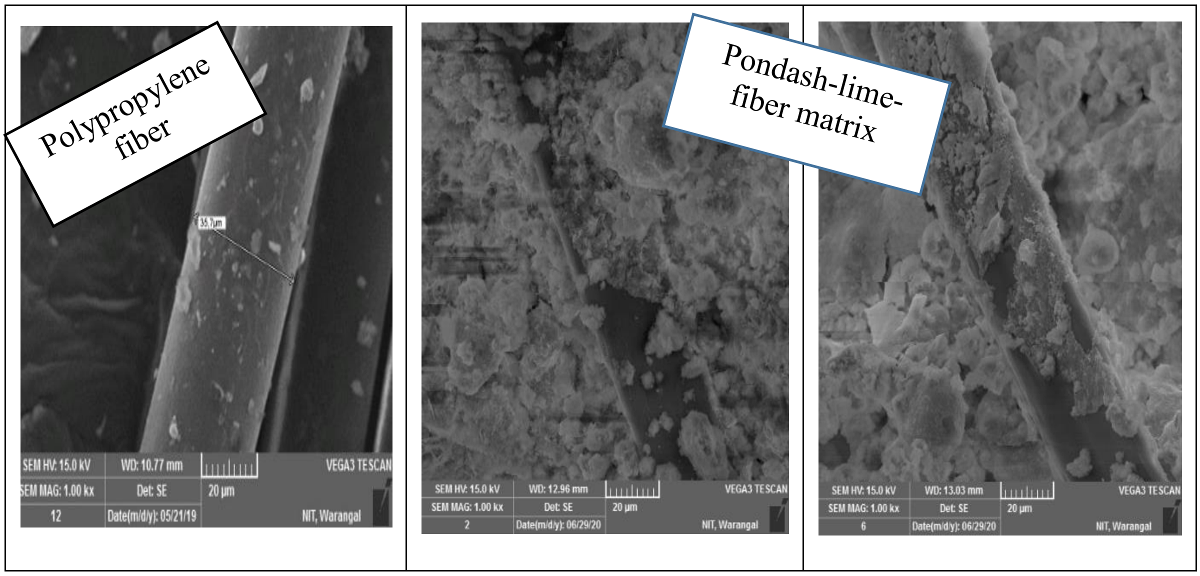

Fig. 4.12. SEM images of $PL_{8\%}F_{1.0\%}$ mix at 28 days curing

Figure 4.13. shows the stress-strain behaviour of 8% lime treated pond ash with different contents of fiber cured for 28 days. Similar stress-strain curves were observed for other PLF

specimens. As it is already discussed that the addition of lime increased the peak stress with brittle failure in nature. The inclusion of fiber in lime treated specimens improved the failure strain by altering from a sudden fall (in lime treated pond ash) to a little gradual with reduced post-peak stress loss. For instance, the axial failure strain of PL_8 specimens increased from 1.27% to 2.03%-3.06%, depending on fiber content. Figure 4.14 show the failure specimens of PLF specimen during testing.

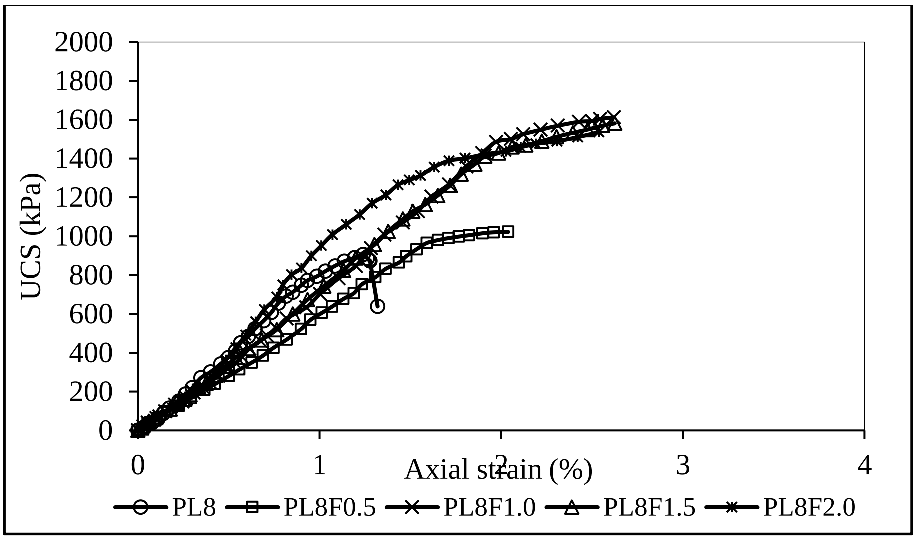

Fig. 4.13. Stress-strain behaviour of lime-fiber treated pond ash (28 days)

Fig. 4.14. Failure specimens of PLF mixes

From the test results obtained, the specimen mix combinations PL_8 (906 kPa) and above, as well as PL_6F_1 (815 kPa) and above, satisfied the UCS criteria as per IRC specifications. Also, the specimen with $PL_8F_{1.0}$ reported the UCS of 1612 kPa, indicating the optimized strength compared to all other combinations.

4.7 Combined Effect of Lime and Fiber on CBR of Pond ash

The CBR test results of pond ash treated with both lime and fiber for curing 7 and 28 days are presented in Table 4.3. It is observed that the CBR of pond ash increased substantially with the addition of both lime and fiber compared to its untreated and individual contents. Compared to the untreated pond ash, CBR values of PLF specimens found to be increased by 407% to 802%, and 640% to 1140% for 7 and 28 days, respectively. Similarly, when compared to unreinforced lime treated pond ash, the CBR values of reinforced specimens increased by 9% to 41%, 10% to 32%, for 7 and 28 days, respectively. The cause of an increase in CBR is due to the enhanced bearing capacity of fiber inclusive-stabilized pond ash (Dhar and Hussain 2018).

Table 4.3. CBR of pond ash-lime-fiber mixture under soaked condition

Mix	**CBR (%)**		**% gain in CBR w.r.t pond ash**		**% gain in CBR w.r.t lime (8%)**		**% gain in CBR w.r.t fiber (1%)**	
	7 days	28 days	7 days	28 days	7 days	28 days	7 days	28 days
P	4.2		-					
PL_8	27.2	39.7	548	845				
$PF_{1.0}$	10.52		150					
$PL_8F_{0.5}$	29.7	43.5	607	936	9	10		
$PL_8F_{1.0}$	**32.1**	**45.6**	664	986	18	15		
$PL_8F_{1.5}$	35.8	50.6	752	1105	32	27		
$PL_8F_{2.0}$	37.9	52.1	802	1140	39	31		
$PL_4F_{1.0}$	21.3	31.2	407	643	41	32	102	197
$PL_6F_{1.0}$	26.5	37.5	531	793	25	14	152	256
$PL_8F_{1.0}$	32.1	45.6	664	986	18	15	205	333
$PL_{10}F_{1.0}$	33.1	47.8	688	1033	12	10	215	352
$PL_{12}F_{1.0}$	34.2	49.6	714	1081	19	15	225	371

As per the Indian Road Congress (IRC) specifications, the minimum bearing ratio required for subbase material shall be 30% for cumulative traffic loads of 3 msa (million standard axles). The study findings showed that the specimen proportions of $PL_8F_{1.0}$ (at 7 days) and above satisfy the pavement sub-base application criteria.

4.8 Summary

The findings showed that the individual optimum contents for improving strength characteristics (UCS and CBR) of pond ash were considered to be 8% lime and 1% fiber. Although the fiber addition enhanced the CBR values of pond ash, its application alone in pond ash-based pavement is not recommended due to lower CBR values (< 30). Modification of pond ash with both lime and fiber enhanced the strength properties further, especially at combined optimum contents. The mode of failure is changed from brittle to ductile, and the strength requirements as per IRC specification are satisfied. The formation of cementing agents such as CSH and CASH due to pozzolanic reaction and mobilization of frictional bonds between fiber and ash particles are liable for the improved strength.

CHAPTER – 5

RESILIENT MODULUS CHARACTERISTICS

5.1 General

Resilient modulus is a fundamental stiffness property of the material similar to the concept of modulus of elasticity, which refers to the stress-strain relationship of material under the applied stress conditions. This chapter presents the repeated load triaxial test results for resilient modulus (M_R) of pond ash (P) treated with different proportions of lime (L) and fiber (F). Two replicates were tested for each mix proportion. Hence, the M_R values represent the average value obtained from the two replicates.

5.2 Specimen Preparation for RLT test

5.2.1 Untreated Specimen

In the study, the moist under compaction method was adopted for reconstituting the untreated pond ash specimens. The moist under compaction method introduced by Ladd (1978), was the revised method for moist tamping. In this method, the targeted density of specimen can be achieved by compacting initial layers to a looser density. The final desired value is to account for the increase in density of the lower layers while the upper layers are placed. This procedure is observed to produce more consistent and repeatable results. Clean, dry pond ash was used to prepare the 75 mm diameter and 150 mm length specimen. A rubber membrane was stretched tight to the inner wall of a split mould to maintain the vacuum, which was then seated on base pedestal of triaxial apparatus, as shown in Figure 5.1. This method is carried in layers forming the entire specimen. A light twist was applied while seating the top cap to maintain full contact between the cap and the specimen. The tests were then performed on unsaturated/partially saturated samples, i.e. sample was kept the same during the testing, without back pressure saturation.

5.2.2 Treated Specimen

Before preparing treated specimens, dry pond ash was thoroughly mixed with the desired amount of additives and water. The mixture was then statically compacted in a cylindrical mould to obtain a sample with the required density with 75 mm diameter and 150 mm height.

The inner surface of the mould was lubricated to reduce the side friction during compaction. After moulding, the specimens were immediately extruded from the split mould and then placed in plastic bags and stored in desiccators to avoid significant moisture content variations before testing.

Fig. 5.1 Setup used in RLT test

5.3 RLT Test Results for M_R

The RLT test was performed as per AASHTO T-307 with 15 different stress levels of confining and deviatoric stresses by implementing on the specimen. The results were then analyzed for the dosage of additives, confining and deviatoric stresses on both untreated and treated pond ash. Further, regression analysis was carried out with the experimental data using selected models available in the literature, and the corresponding coefficients were calculated.

5.3.1 Effect of Additives on M_R

The M_R was determined as the average of the last five cycles of each load sequence according to AASTHO T307. Figure 5.2a illustrates the relation between M_R and applied stresses deviatoric stresses at different confining stresses for the untreated compacted specimen (P). It is noted from the fig that the M_R of compacted pond ash varied from 13 to 25 MPa, indicating the insignificant variation in MR with increased stress levels. As the deviatoric stress increases, the M_R values are observed to be decreased. The impact of confining stress was found to be less pronounced for pond ash. This kind of findings was observed in previous studies for fine-grained silty type soils compacted to their MDD at OMC condition (Arab et al. 2019), and coal ash-based materials (Dev and Robinson 2019).

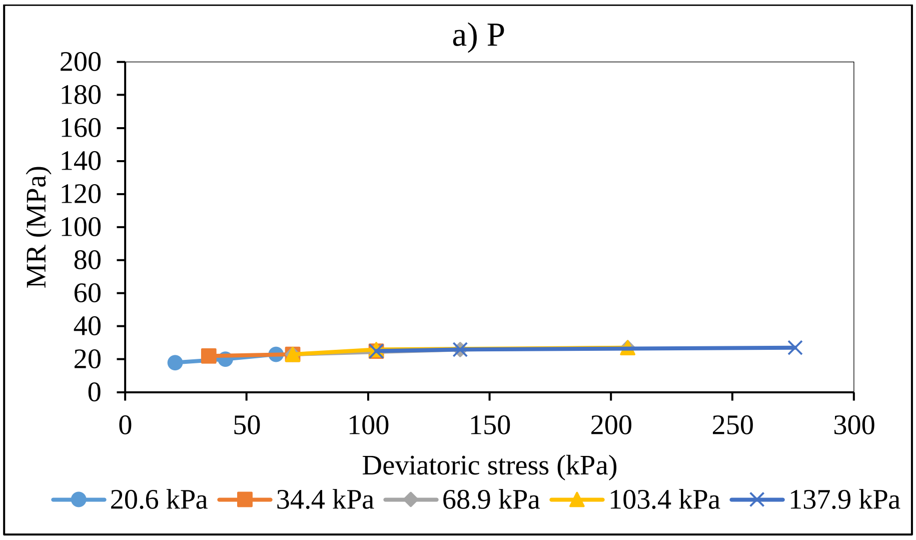

Fig. 5.2a. M_R of untreated pond ash at various σ_c and σ_d stress levels

Conversely, the M_R values for treated pond ash increased with an increase of both deviatoric and confining stresses during the test sequences.

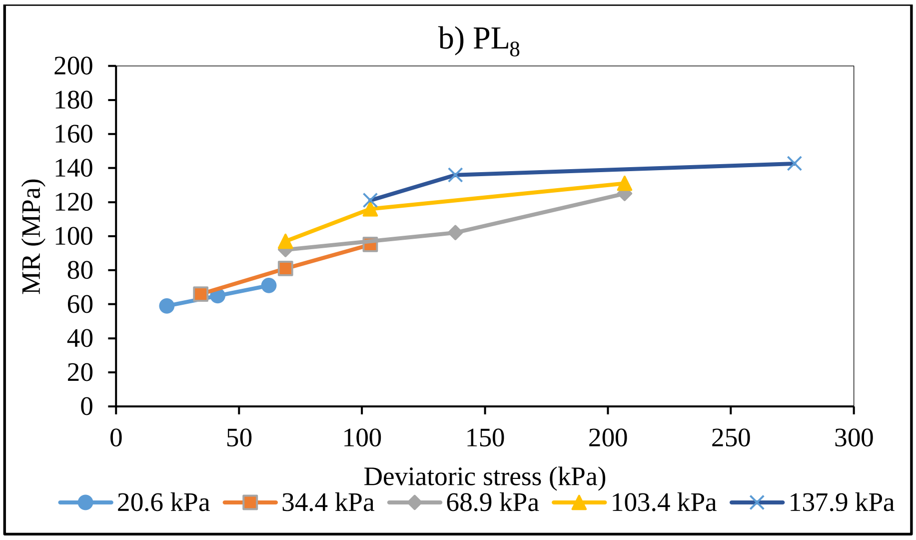

Fig 5.2b. M_R of lime treated pond ash at various σ_c and σ_d stress levels

Figure 5.2b illustrates the relationship between M_R and applied stresses deviatoric stresses at different confining stresses for lime treated pond ash (PL_8). It is observed that the MR of PL_8 shows the significant variation in M_R values with increased stress levels in an incremental way; i.e. from 59 MPa to 143 MPa compared to M_R of untreated pond ash. This is due to the formation of cementitious bonds between pond ash particles with the passage of time; which exhibits strain hardening with a slight effect of confining stresses on the specimen (Patel and Shahu 2016).

Similarly, Figure 5.2c illustrates the relation between M_R and applied stresses deviatoric stresses at different confining stresses for fiber-reinforced pond ash (PF_1). As shown in fig, the MR values of PF_1 shows a considerable variation with increased stress levels, i.e. from 28 MPa to 63 MPa compared to M_R of untreated pond ash; this is due to mobilization of tensile resistance, which allows a significant contribution of the fibres to the rigidity of the composite specimen (Kumar and Singh 2008; Heineck et al. 2005).

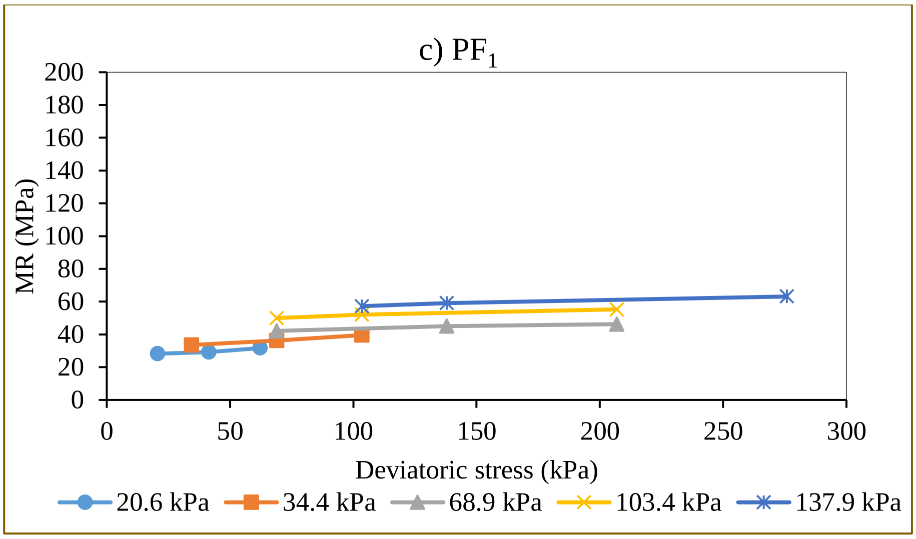

Fig. 5.2c M_R of fiber-reinforced pond ash at various σ_c and σ_d stress levels

Figure 5.3(a to h) show the variation in M_R values with the combined effect of lime and fiber on selected mix proportions for deviatoric and confining stresses. The range of M_R values obtained, along with M_R value at particular stress level-6 (σ_c of 34.4 kPa and σ_d of 103.4 kPa, NCHRP 2004) are presented in Figure 5.4. It can be seen from the figure that compared to untreated (P), lime treated (PL) and fiber treated (PF) specimens, the M_R values of PLF composites observed to be increased furthermore. However, an optimal increment in M_R of pond ash with increased stress levels can be observed at a combination mix of $PL_8F_{1.0}$ (82-196 MPa); after that, with an increase of added additives contents, the increment rate in M_R observed was only marginal. The findings observed are possibly mirrored patterns noted in both UCS and CBR values.

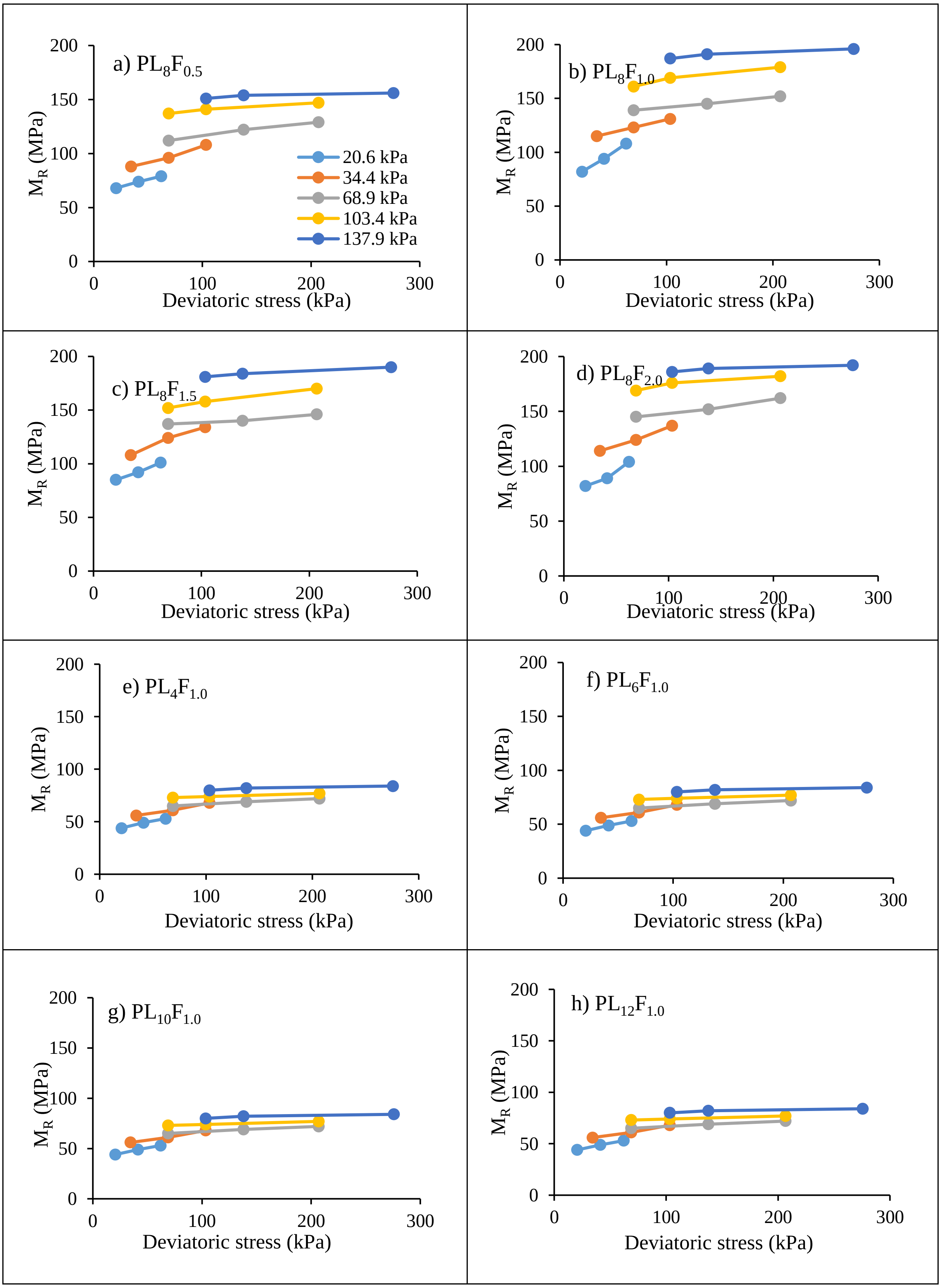

Fig. 5.3(a-h). M_R of lime-fiber treated pond ash at various σ_c and σ_d stress levels (28 days)

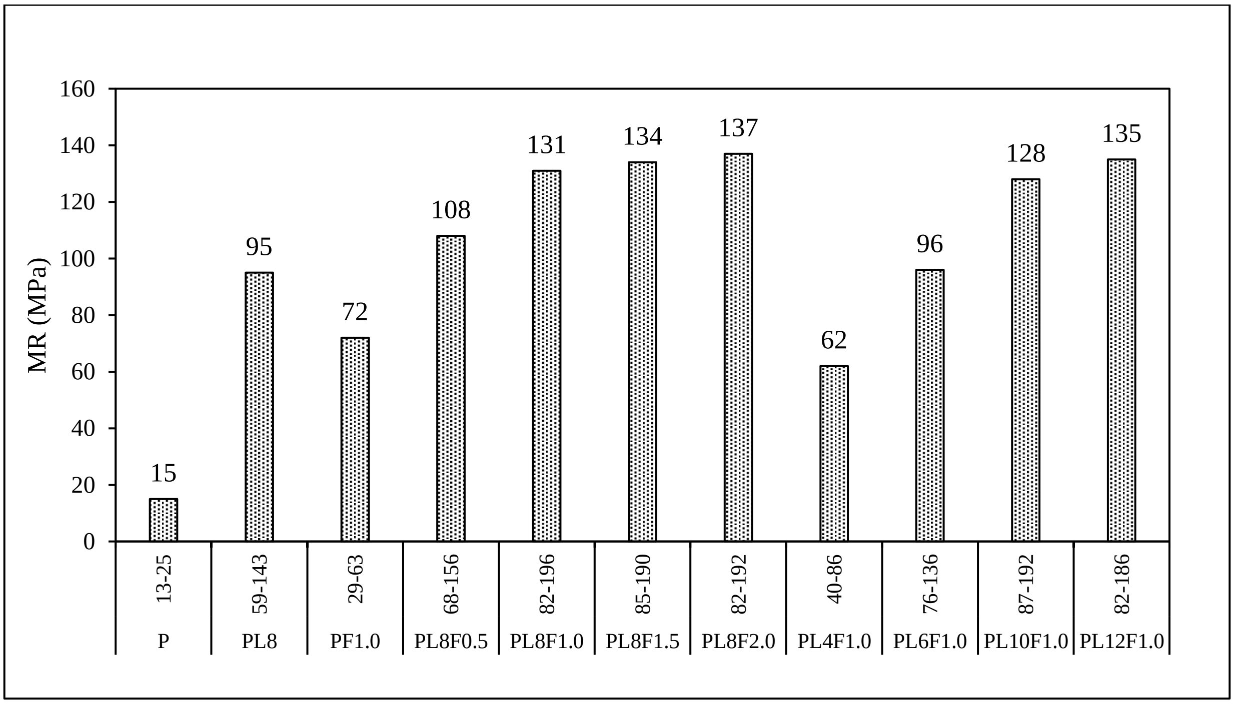

Fig. 5.4. M_R values of pond ash-lime-fiber mixes (MPa)

5.4 Effect of Applied Stresses on M_R

The effect of σ_c and σ_d stresses on the pond ash specimens is Figure 5.5 and Figure 5.6. In general, for a typical flexible pavement, base/subbase layers experience confining and deviatoric stresses of 34.4 kPa and 103.4 kPa, respectively (NCHRP 2004). Therefore, these stresses considered as the reference for comparing M_R in the following section. From the previous section results, it is seen that the M_R values of treated pond ash increased with an increase of deviatoric and confining stresses levels. For instance, from Figure 5.5, at σ_c of 34.4 kPa, the M_R values of PL_8 increased by 30% with an increase of σ_d from 34.4 kPa to103.4 kPa; which is due to the development of strain hardening nature by yielding low axial strains in the specimen. Strain hardening is the phenomenon where the material becomes more robust with each cycle of loading, and this phenomenon is more pronounced in cohesionless materials (Puppala et al. 2011). Similarly from Figure 5.6, at σ_d of 103.4 kPa, with an increase of σ_c from 34.4 kPa to 137.9 kPa, M_R values of specimen increased by 42%. This is because, with an increase in confinement, the specimen tend to get denser and stiffer; thereby exhibiting greater stiffness and hence higher M_R values for given deviatoric stress level (Patel et al. 2019).

In the case of reinforced pond ash, with an increase in σ_d, M_R values increased at particular given confining stress. This behaviour is attributed to the increase in pond ash-fibre interface mechanism, which allows a change from slip-yield phase to a phase where all fibres yield-stretch in a specimen (Consoli et al. 2010). For instance, pond ash with 1 % fibre inclusion, at constant σ_d of 103.4 kPa, M_R values increased by 42% with an increase of σ_c from 34.4 kPa to 137.9 kPa. Similarly, at constant σ_c of 34.4 kPa, the M_R values increased by 18% when σ_d increased from 34.4 kPa to 103.4 kPa. The same kind of M_R improvement was observed in PLF composite specimens, and the improved trend in M_R of treated pond ash is in good comparable to UCS and CBR values. From Figures. 5.5 and 5.6, it is also noted that the rate of increase in M_R with increased deviatoric stress is less at high confining stress than at lower confining stress levels. This indicates the lesser influence of deviatoric stress on M_R at higher confining stresses owing to higher strength and stiffness at this confinement; hence, the specimen did not exhibit any additional stiffening when loaded with higher deviatoric loads (Puppala et al. 2011).

As per IRC-37 (2018), the subbase material should exhibit the modulus values of minimum 100MPa to a maximum of 300MPa-350MPa. However, the obtained M_R values under laboratory conditions (at particular stress level) show M_R of 131MPa marginally satisfying the criteria. Accordingly, its application in unpaved/low volume roads (up to 10msa, based on strength aspects) is preferable rather than in high volume flexible pavements like National Highways and important state highways though the strength criteria like CBR satisfying its minimum requirement at seven days curing)

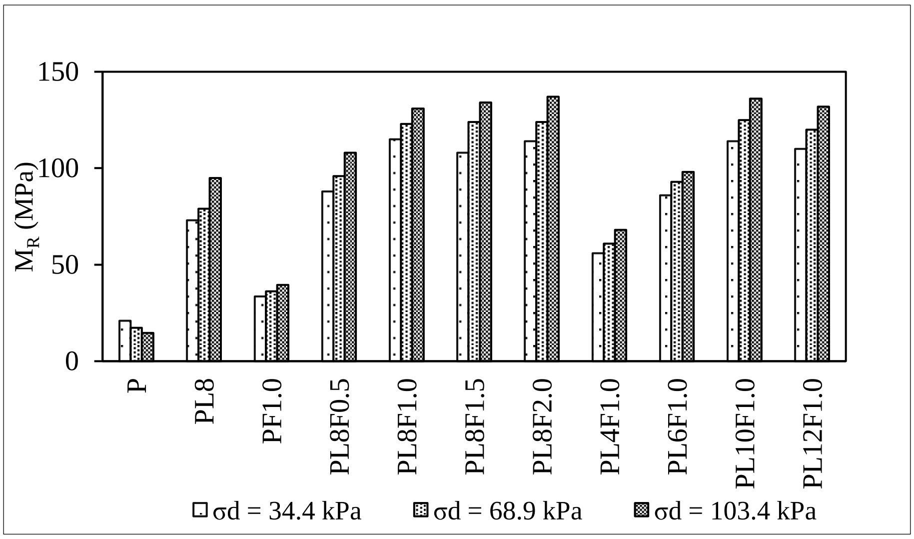

Fig. 5.5. M_R of PLF mixtures for various σ_d stress levels at σ_c of 34.4 kPa

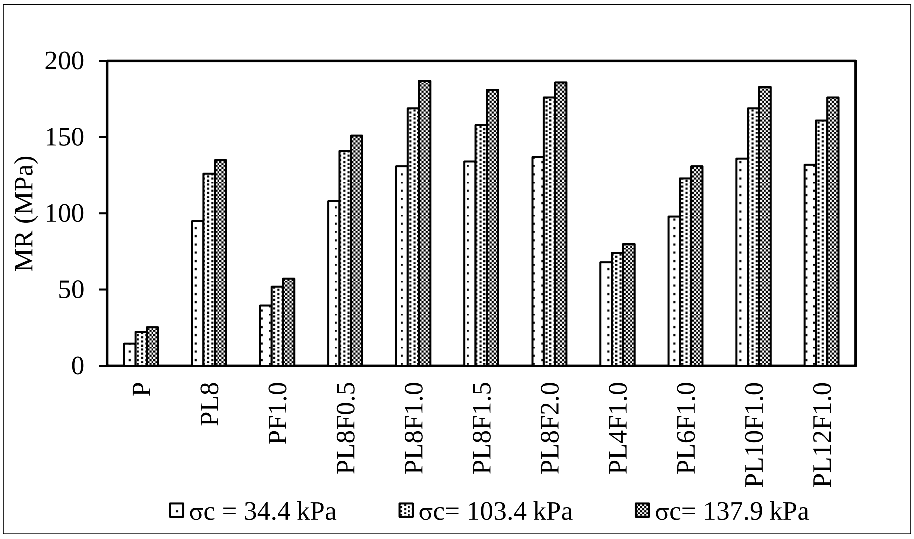

Fig. 5.6. M_R of PLF mixtures for various σ_c stress levels at σ_d of 103.4 kPa

5.5 Modelling Studies on M_R Response

Development of a model to estimate M_R depends on material type, physical condition, and stress state. Therefore, an ideal model that represents the behaviour of the physical system must integrate all such influencing factors (Ikeagwuani and Nwonu 2019, 2020). In practice, the stresses acting around the material at its particle level have a greater significance on M_R response. These stresses are often represented in terms of confinement (confining/bulk/octahedral stresses), and loading (deviatoric/octahedral shear stresses) and also suction (if unsaturated soil state considers) (Noolu et al. 2018; AASTHO 2000). Hence, to simulate such field-stress conditions, previous researchers have developed various stress-based constitutive models and expressed them in mathematical equations for both cohesive and cohesionless soils. These models are powerful tools for the mechanistic and empirical design of pavement layers. As resilient modulus is stress-dependent, the response of fine-grained soils and coarse-grained soils to the stress application vary. AASHTO recommends the use of bulk stress-related models for prediction of resilient modulus for cohesionless soils.

The present work was conducted on pond ash material. Hence, the bulk stress model (Uzan 1985), Power model (Witczak and Uzan 1988) as two-parameter based models, and Pezo model (Uzan 1985), Octahedral shear stress model (Mohammed et al. 1999) as three-parameter based models, are considered to evaluate their suitability in pavement applications. The linear statistical regression analysis was carried out to validate the experimental M_R values; as well as to identify the corresponding regression model constants and correlation coefficients (R^2 values)

5.5.1 Model 1 (M1): Bulk Stress Model (Uzan 1985)

The bulk stress model is based on the relation

$$M_R = k_1 * \theta^{k_2} \tag{5.1}$$

Where, k_1 and k_2 are regression parameters

θ = bulk stress (kPa) = (σ_1+2σ_d)

σ_1 = axial stress (kPa) = (σ_c+σ_d)

σ_d = deviatoric stress (kPa)

σ_c = confining stress (kPa)

The bulk stress model depends only on the confining pressure; it does not consider the effect of deviator stress, shear stresses and strains developed during loading, which is one of the major disadvantages of this model. The regression constants k_1, k_2 and the R^2 values obtained for all the samples based on bulk stress model for pond ash-based specimens are given in Table 5.1. It can be observed that the bulk stress model was not able to model the resilient modulus behaviour as the R^2 values obtained are less than 0.9 for most of the cases.

Table 5.1. Regression analysis for bulk stress model

		P	**PL_8**	**$PF_{1.0}$**	**$PL_8F_{0.5}$**	**$PL_8F_{1.0}$**	**$PL_8F_{1.5}$**	**$PL_8F_{2.0}$**	**$PL_4F_{1.0}$**	**$PL_6F_{1.0}$**	**$PL_{10}F_{1.0}$**	**$PL_{12}F_{1.0}$**
Model 1	k_1	15.824	9.168	4.946	11.202	16.154	17.388	14.658	13.623	18.046	16.706	14.180
	k_2	0.001	0.4195	0.387	0.413	0.386	0.367	0.405	0.282	0.313	0.378	0.401
	R^2	0.415	0.893	0.781	0.882	0.880	0.865	0.870	0.856	0.884	0.861	0.865

5.5.2 Model 2 (M2): Power Model, (Witczak and Uzan 1988)

The power model is based on the relation

$$M_R = k_3 * {\sigma_d}^{k_4} \tag{5.2}$$

Where k_3 and k_4 are regression parameters

σ_d is the deviator stress

Power model is based on the deviator stress acting on the pavement layer. It does not consider the effect of the confining stresses at different layers. The regression constants k_3, k_4 and the R^2 values obtained for the power model of M_R values are given in Table 5.2. It can be observed that the R^2 values obtained are less than 0.9, suggesting that the Resilient Modulus of both compacted and treated pond ash depends on both confining pressure and deviator stresses applied.

Table 5.2. Regression analysis for power model

		P	PL_8	$PF_{1.0}$	$PL_8F_{0.5}$	$PL_8F_{1.0}$	$PL_8F_{1.5}$	$PL_8F_{2.0}$	$PL_4F_{1.0}$	$PL_6F_{1.0}$	$PL_{10}F_{1.0}$	$PL_{12}F_{1.0}$
	k_3	14.728	17.559	10.808	25.540	34.269	35.340	32.290	22.678	33.720	35.968	31.240
Model 2	k_4	0.065	0.380	0.309	0.333	0.315	0.301	0.331	0.239	0.253	0.302	0.325
	R^2	0.056	0.864	0.693	0.704	0.713	0.717	0.713	0.757	0.705	0.677	0.701

5.5.3 Model 3 (M3): Pezo Model, (Pezo 1993)

Pezo model is an advanced model used in predicting the Resilient Modulus behaviour of both cohesionless and cohesive soils as it takes in to account the effect of both confining pressure and deviator stress acting on the sample. The Pezo model is based on the relationship:

$$M_R = k_5 * \left(\frac{\sigma_c}{P_a}\right)^{k_6} * \left(\frac{\sigma_d}{P_a}\right)^{k_7} \qquad (5.3)$$

Where k_5, k_6 and k_7 are regression parameters,

σ_d is the deviator stress

σ_c is the confining stress

The regression parameters and correlation coefficients obtained based on Pezo model are given in Table 5.3. It can be observed that except for compacted pond ash, Pezo model could predict the Resilient Modulus behaviour of al treated pond ash in an efficient way as the R^2 values obtained for all the samples are greater than 0.9.

Table 5.3. Regression analysis for pezo model

		P	PL_8	$PF_{1.0}$	$PL_8F_{0.5}$	$PL_8F_{1.0}$	$PL_8F_{1.5}$	$PL_8F_{2.0}$	$PL_4F_{1.0}$	$PL_6F_{1.0}$	$PL_{10}F_{1.0}$	$PL_{12}F_{1.0}$
	k_5	0.528	1.039	0.743	1.135	1.237	0.909	1.248	0.871	1.075	1.234	1.217
Model 3	k_6	0.371	0.199	0.291	0.303	0.273	0.172	0.286	0.175	0.231	0.286	0.289
	k_7	-0.213	0.231	0.091	0.106	0.110	0.218	0.117	0.107	0.079	0.088	0.108
	R^2	0.928	0.977	0.986	0.982	0.968	0.983	0.966	0.952	0.986	0.965	0.965

5.5.4 Model 4 (M4): Octahedral Shear Stress Model, (Witczak and Uzan 1988)

The Octahedral shear stress model is developed by Witczak and Uzan (1988) and is incorporated in the Mechanistic-Empirical Pavement Design Guide (M-EPDG). This model considers the effect of confining pressure, deviator stress, and the shear stresses developed on the sample during loading. This model is based on the relationship,

$$M_R = k_8 * \left(\frac{\theta}{P_a}\right)^{k_9} * \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_{10}} \qquad (5.5)$$

Where, k_8, k_9 and k_{10} are regression parameters

θ = bulk stress (kPa) = (σ_1+2σ_3)

τ_{oct} = Octahedral shear stress, = 1/3 $\{(\sigma_1-\sigma_2)^2 + (\sigma_2-\sigma_3)^2 + (\sigma_3-\sigma_1)^2\}^{1/2}$;

σ_d = deviatoric stress (kPa)

σ_3 = confining pressure (kPa)

σ_1= axial stress (kPa) = σ_c +σ_d and Pa = Atm. Pressure = 101.4 kPa

The obtained regression parameters and correlation coefficients for pond ash based on the octahedral shear stress model are given in Table 5.4. It can be observed from the table that the model was able to predict the Resilient Modulus behaviour of treated and compacted pond ash efficiently as the R^2 values obtained for all the samples are greater than 0.9.

Table 5.4. Regression analysis octahedral shear stress model

		P	**PL$_8$**	**PF$_{1.0}$**	**PL$_8$F$_{0.5}$**	**PL$_8$F$_{1.0}$**	**PL$_8$F$_{1.5}$**	**PL$_8$F$_{2.0}$**	**PL$_4$F$_{1.0}$**	**PL$_6$F$_{1.0}$**	**PL$_{10}$F$_{1.0}$**	**PL$_{12}$F$_{1.0}$**
Model 4	k_8	0.488	0.795	0.591	0.893	0.989	0.715	0.986	0.738	0.896	0.993	0.964
	k_9	0.448	0.323	0.424	0.466	0.431	0.284	0.457	0.293	0.352	0.441	0.450
	k_{10}	-0.724	0.267	-0.104	-0.148	-0.125	0.262	-0.145	-0.032	-0.108	-0.175	-0.135
	R^2	0.769	0.975	0.985	0.989	0.975	0.980	0.976	0.956	0.990	0.972	0.971

The regression constants k_1, k_3, k_5, and k_8, are related to material properties, proportional to their elastic behaviour. Thus the values are always positive and are observed to be increased with an increase in the additive contents. Likewise, the constants k_2, k_4, k_6, and k_9 represent either bulk/confining or deviatoric stress exponents depending on the model considered. These constants are positive in nature with an increase of stresses, and resulting in increased modulus for all treated specimens. And the constants k_7 and k_{10} are assumed to represent the exponents of shear stress terms, though these values were observed to be both positive and negative with no specific trend in variation.

From the regression analysis results, compared to untreated compacted pond ash the regression constants k_1, k_3, k_5, and k_8 values are observed to be higher for treated specimens; and these values increase with additive contents. The values k_2, k_4, k_6, and k_9, in general, were also found to be higher for treated ash composites compared to untreated ash specimens. These values are positive in nature for all the models. The regression constants k_7 values found to be both positive and negative, and k_{10} values are found to be negative for almost all the cases.

From the statistical multi regression analysis carried out for M_R of treated and compacted pond ash, It can be seen that the R^2 values for two-parameter models (M1 and M2) are less than 0.9 compared to three-parameter models (M3 and M4). This is due to considering the effect of only confining stress (in M1) and deviatoric stress (in M2), and ignoring the combined effect all stresses acting on the specimen during loading. On the other hand, the three-parameter models such as M3 and M4 considered the combined stress effect (bulk/confining, deviatoric and shear) acting on the specimen and have shown good correlation values ($R^2 > 0.9$), particularly for treated specimens, indicating better fitting of the models with experimental results. The advantage of these three-parameter models lies in the separation of individual stress effect on M_R values (Patel et al. 2019; Dev and Robinson 2015). From the regression analysis studies, it can be stated that the three-parameter regression analysis models used for the evaluation of M_R prediction can be used effectively for modified coal ash-based materials.

5.6 Correlation between UCS, CBR and M_R

5.6.1 Correlation between *CBR* and UCS

In the previous chapter, UCS, CBR results obtained for pond ash samples were discussed. It is also mentioned that as per IRC specification the UCS of 750 kPa and CBR of 30 are the minimum required strength criteria for subbase application. Based on these limitations, the following correlation studies are carried out from the test results.

From the test data of

The correlation between CBR (%) values with UCS (kPa) values obtained for clayey soils observed by Black (1979) as

$$\text{UCS} = 17.2 \text{ to } 22 \times \text{CBR} \tag{5.5}$$

In a similar way, Brown et al. (1987), the relation reported as

$$\text{UCS} = 15.6 \times \text{CBR} \tag{5.6}$$

Dev and Rabinson (2019), reported for flowable fills as

$$\text{UCS} = 1.34\ (\text{CBR})^{1.67} \tag{5.7}$$

In the present study, the variation of CBR with UCS is shown in Figure 5.7.

As expected, the values of CBR increase with UCS values. From the limited data, the relation between CBR and UCS can be obtained as,

$$UCS = 0.3514 * (CBR)^{2.134} \tag{5.8}$$

with a correlation coefficient of 0.83.

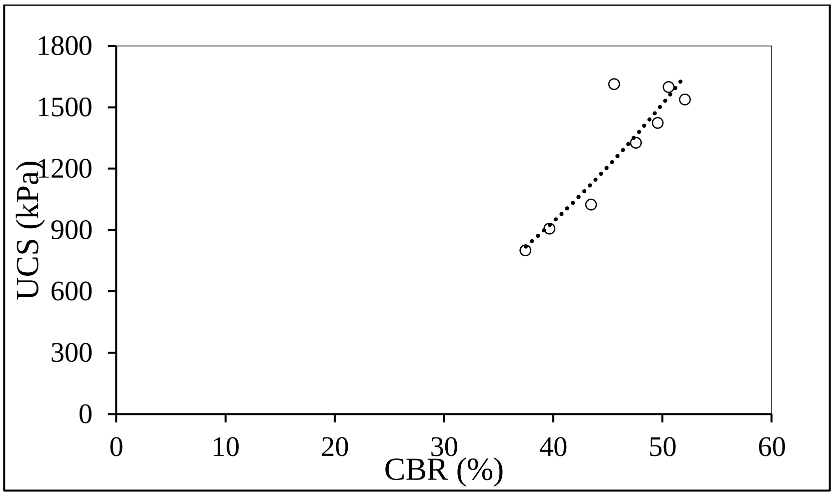

Fig. 5.7. Correlation between UCS and CBR

5.6.2 Correlation between M_R and UCS

M_R values are often correlated with UCS values (Lee et al. 1995; Thompson and Robnett, 1979, Dev and Rabinson (2019). As M_R varies with confining and deviatoric stresses, the values corresponding to 34.5 kPa and 103.4 kPa is generally considered for design for subbase/base (NCHRP 2004). The variation of M_R with UCS values obtained in the present study for the subbase condition is plotted in Figure 5.8. The data corresponding to UCS to greater than or equal to 750 KPa is considered. The correlation between M_R (in MPa) and UCS (in kPa) is obtained as,

$$M_R = 2.38 * (UCS)^{0.549} \tag{5.9}$$

with a correlation coefficient of 0.94.

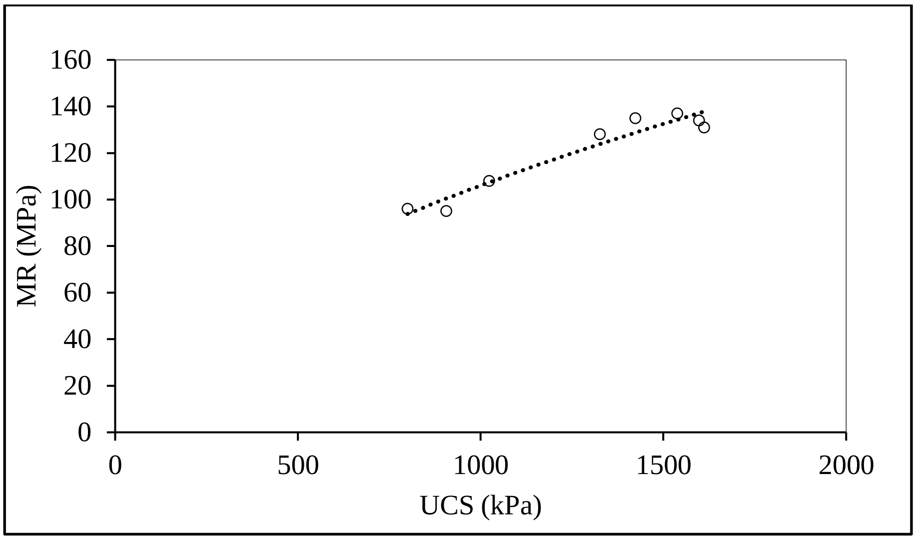

Fig. 5.8. Correlation between UCS and M_R

5.6.3 Correlation between M_R and CBR

Similar to correlation between M_R and UCS, attempts were also made in the literature to correlate M_R with CBR (%). Some of the correlations developed for soils are:

M_R (MPa) = 10.34 CBR (Heukelom and Klomp, 1962) (5.10)

M_R (MPa) = 37.3 $(CBR)^{0.71}$ (Green and Hall, 1975) (5.11)

M_R (MPa) = 17.6 $(CBR)^{0.64}$ (Powell *et al.* 1984) (5.12)

M_R (MPa) = 1.1 to 16.69 (CBR) (Duncan and Buchignani, 1976) (5.13)

MR (MPa) = 1.75 $(CBR)^{1.46}$ (Dev and Rabinson, 2019). (5.14)

The relationship obtained between the Resilient Modulus values and the soaked CBR values is shown in Figure 5.9. The best fit is obtained as,

$$M_R = 0.344 * (CBR)^{1.529} \quad (5.15)$$

with a correlation coefficient of 0.95

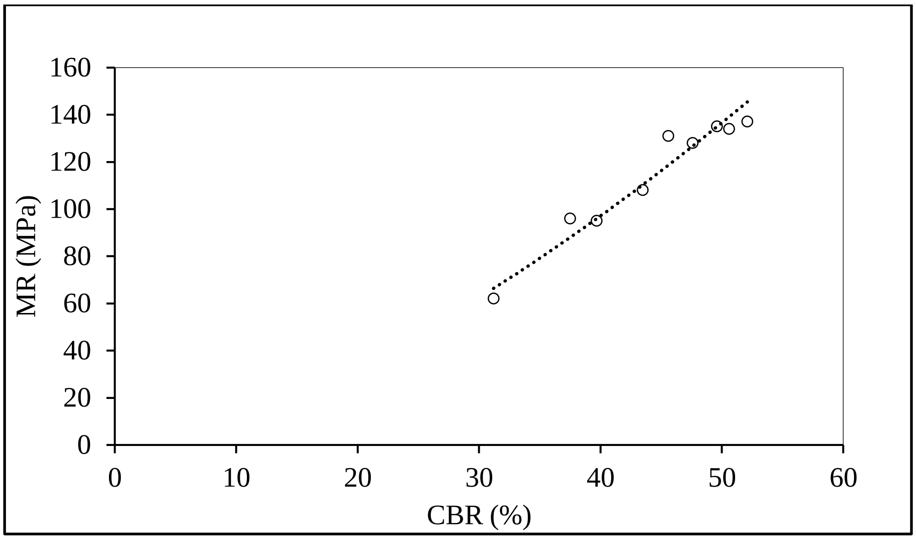

Fig. 5.9 Correlation between M_R and CBR

5.6.4 Correlation between M_R, UCS and CBR

Similar to the above, the relations obtained between the M_R, UCS, and the soaked CBR values is shown in Figure 5.10, and the best fit is obtained as,

$$M_R = 0.03 * (UCS) + 1.59 * (CBR) + 9.25 \tag{5.16}$$

with a correlation coefficient of 0.96

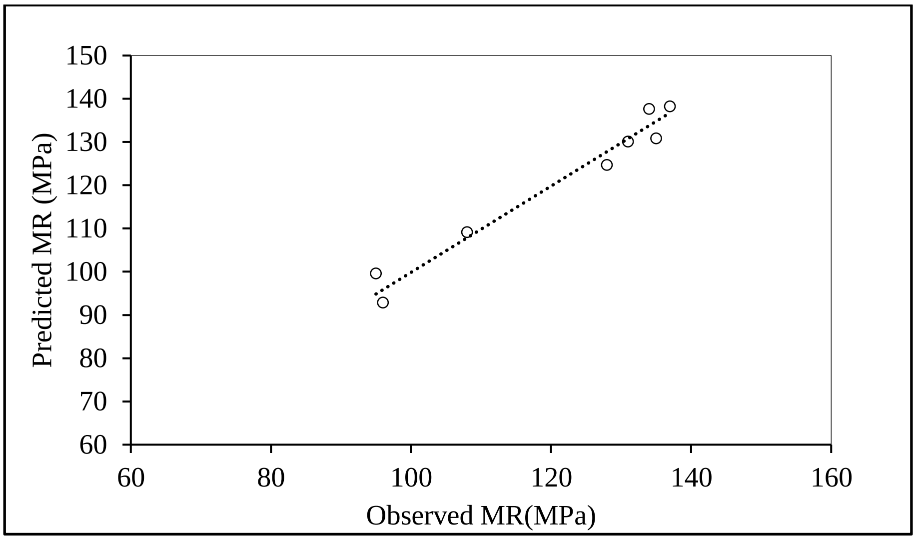

Fig. 5.10 Correlation between M_R, UCS and CBR

5.7 Summary

The current work examined the effectiveness of lime, fiber and both on the enhancement of resilient modulus of pond ash. The previous section results indicated that 8% lime and 1% fiber were found to be optimal based on the strength characteristics of pond ash. With the addition of 8% lime and 1% fiber reinforcement, the resilient modulus values of pond ash exhibited 3.3 to 6.5 times and 1.6 to 2.8 times higher than untreated ash, respectively. The combined effect of lime and fiber leads to increased M_R of pond ash further by 4.6-9.0 times due to increased mobilized frictional bonding area between fiber-ash particles, with significant rate up to their optimum content combination (PL_8F_1). The effect of confining pressure, deviatoric stress levels on the resilient modulus of treated and untreated pond ash samples were examined. The resilient modulus values increased with an increase in confining and deviatoric stresses for treated specimens. The increased stress levels show the insignificant variation in M_R values, whereas, for untreated specimens, M_R values decreased with an increase of deviatoric stresses. Among the four models considered, using multiple regression analysis the three-parameter models such as Model 3 and Model 4 were found to fit the experimental data of M_R values well for treated pond ash samples with the coefficient of determination of $R^2 > 0.9$.

CHAPTER – 6

PERMANENT DEFORMATION CHARACTERISTICS

6.1 General

The pavement distress is closely related to permanent deformations (ϵ_p) due to the accumulation of permanent deformations on the pavement layers due to repeated wheel loads, which can significantly influence the pavement performance and cause distress as rutting, cracking, and potholes. Hence, Mechanistic-Empiristic Pavement Design methods [M-EPD] consider the deformation criteria an important phenomenon for pavements (Puppala et al. 1999). In this chapter, the ϵ_p behaviour of pond ash treated with lime, fiber and both in various proportions subjected to repeated cyclic loading under varying deviator stress levels is evaluated. Effect of various factors such as admixture, applied cyclic deviatoric stress, and the number of load cycles on ϵ_p response of pond ash is studied. The ϵ_p data obtained from the experiments is fitted with four regression models such as Logarithmic model by Barksdale (1972), Power law model by Monismith et al. (1975), Three-parameter models recommended by Ullditz (1993), and Puppala et al. (1999).

6.2 Repeated Load Triaxial Test

The accumulation of permanent deformation mainly depends on the intensity of applied cyclic deviatoric stress and the number of loading cycles; it is generally used to study the deformation characteristics of compacted material (Dawson et al. 2007). In this study, two replicates of each mix proportion cured for 28 days are subjected to stress level (discussed in section 3.2.9.2) through RLT test to determine the ϵp. The average value of accumulated permanent strains was measured at a regular interval after completing 1, 10, 100, 1000, 2000, 3000 and up to 10,000 cycles (Ling X et al. 2017; Rababah et al. 2020).

6.2.1 Effect of Additives on ϵ_p Behaviour

The variation of permanent deformation behaviour with the number of load cycles for pond ash (P), lime treated pond ash (PL_8), and fiber-reinforced pond ash (PF_1) is shown in Figure 6.1. It is observed that with an increase in the number of load cycles, the ϵ_p values increased because each load application contributed to a slight increase in the accumulation of strain in the specimen. However, at the start of the process, the responses were found to be plastic for a finite number of load cycles (up to 200-400 N), i.e., ϵ_p values increased rapidly as load repetitions increased. After the completion of post compaction phase, ϵ_p values remained nearly constant, indicating that the response becomes resilient (Puppala et al. 2009, 2011; Patel et al. 2016, 2018, 2019). Similar responses were observed for lime-fiber treated pond ash blends (PLF), and the permanent strain (ϵ_p) values at selected regular load intervals are shown in Figure 6.2. It is noted from fig 6.1 that with an increase of load cycles from 100 to 10000 number, the ϵ_p values of untreated pond ash increased from 2.313%-3.562%. Whereas for the same load cycle range, the addition of lime and fiber reduced the accumulation of strains in pond ash to a considerable extent, i.e., from 0.44% to 1.52% for PL_8 and 1.42% to 2.196% for $PF_{1.0}$, indicate 57% and 43% reduction in ϵ_p values compared to untreated pond ash, respectively.

The addition of both lime and fiber further decreased the accumulation of ϵ_p compared to untreated pond ash; which is due to increased compressive resistance against external loading leading to reduced deformation (Kumar and Singh 2008; Rababah et al. 2020). However, the reduction in ϵ_p is less at lower lime-fiber mix combinations than lime treated pond ash but higher than reinforced pond ash. The maximum rate of reduction in ϵ_p was found to be at higher PLF mixes up to $PL_8F_{1.0}$, (i.e. 67%). After that, the reduction rate in ϵ_p values was observed to be marginal for remaining higher mix proportions (Figure 6.2). This behaviour of PLF specimens was in good agreement with previous literature (Mo Y X et al. 2019; Patel et al. 2016, 2019; Georgees R et al. 2018; Arulrajah et al. 2013; Oliveira et al. 2018). All these observations are found to be precisely in contrast to the behaviour observed for M_R value. The higher the M_R values, the higher the deformation resistance, which results in lower ϵ_p characteristics under applied loads; indicating that both lime and fiber can be used to further minimize the rut deformation of the pavement layer.

6.2.2 Effect of Applied Stresses on ϵ_p

It can be observed from Figure 6.1 that the ϵ_p values increased with an increase in the applied cyclic deviatoric stress on pond ash for untreated and treated conditions; showing the dependency of these test specimens on the deviatoric stress. The similar behaviour is noticed for all specimens with irrespective of the influence of additives. The higher the applied deviatoric stress, higher is the permanent strain in the specimen for a given confining pressure (Patel et al. 2018; Mohanty et al. 2011, 2013).

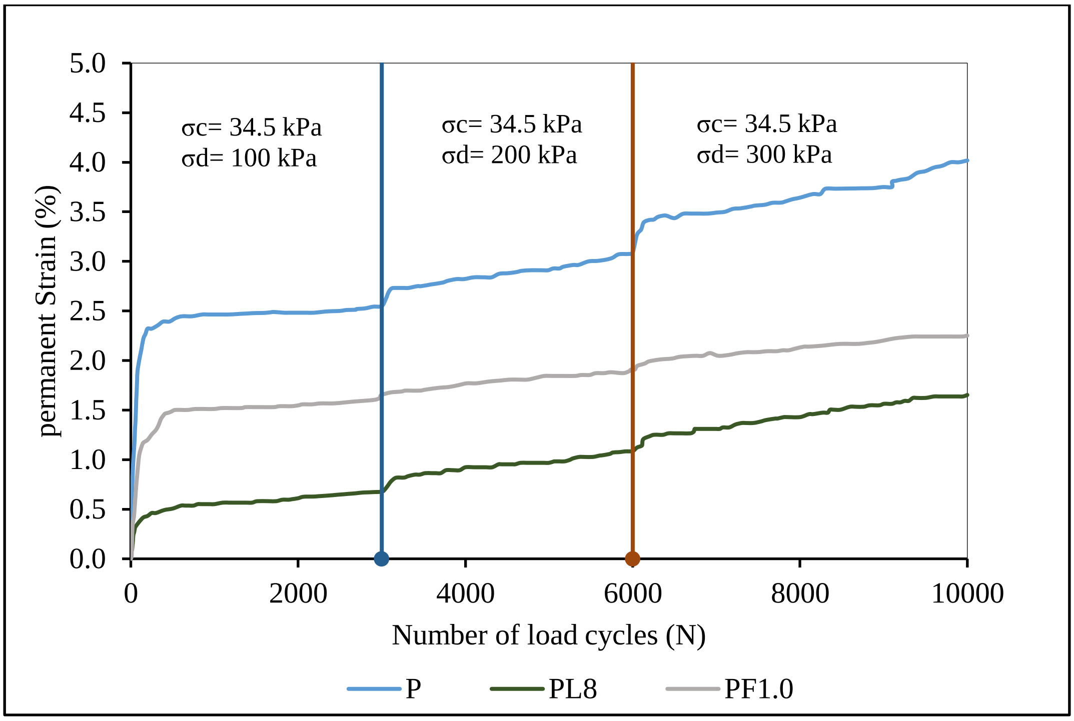

Fig. 6.1. Permanent strain (ϵ_p) of untreated/ lime and fiber treated pond ash

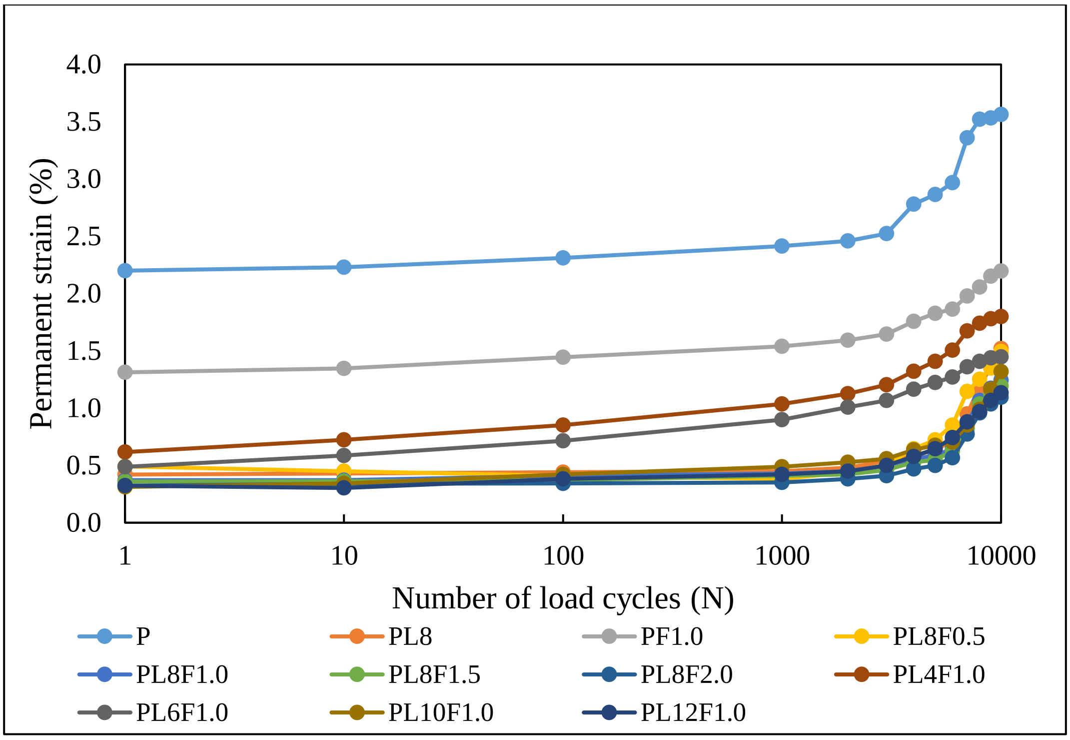

Fig. 6.2. Permanent deformation (ϵ_p) behaviour of PLF compositions

6.3. Modelling Studies on ϵ_p Response

Similar to the M_R model studies, several researchers have proposed mechanistic-empirical permanent deformation prediction models in acceptable accuracy by considering various key factors (Zhao and Dennis 2007; Noolu et al. 2018; Uzan et al. 2004; Rehman et al. 2014; Puppala et al. 2009; Perez et al. 2006; Patel et al. 2019). A few models reported in the literature were developed by considering the effect of stress levels or the effect of the number of load cycles. A few models were developed by combining the influence of stress levels and the number of load cycles. In the present study, few established and widely used models (4-No's) have been chosen to validate experimental deformation characteristics. The multiple linear statistical regression analysis was carried out to identify the corresponding model constants and correlation coefficients (R^2 values).

6.3.1 Model 1 (M1): Logarithmic Model, (Barksdale 1972)

Logarithmic model (semi-log model) is expressed as a linear relationship between permanent axial strain and the logarithm of number of load cycles as follows;

$$\epsilon_p = \alpha_1 + \alpha_2 \log_e (N) \tag{6.1}$$

Where

α_1, α_2 = model constants;

N = Number of load repetitions

The regression constants α_1, α_2 and the R^2 values obtained for all pond ash-based specimens based on the logarithmic model are given in Table 6.1. It can be observed that the logarithmic model was not able to model the ϵ_p behaviour as the R^2 values obtained are less than 0.8 for most of the cases. The logarithmic model considered only the effect of load cycles effect and does not account for stress levels, thus resulting in low correlation values.

Table 6.1. Regression analysis model constants of permanent deformation for Model 1

		P	PL_8	$PF_{1.0}$	$PL_8F_{0.5}$	$PL_8F_{1.0}$	$PL_8F_{1.5}$	$PL_8F_{2.0}$	$PL_4F_{1.0}$	$PL_6F_{1.0}$	$PL_{10}F_{1.0}$	$PL_{12}F_{1.0}$
	α_1	1.885	0.197	1.149	0.210	0.210	0.185	0.176	0.157	0.423	0.347	0.146
Model 1	α_2	0.134	0.079	0.086	0.081	0.081	0.069	0.066	0.061	0.124	0.105	0.078
	R^2	0.566	0.359	0.706	0.355	0.355	0.408	0.394	0.382	0.791	0.877	0.532

6.3.2 Model 2 (M2): Power Law model, Monismith et al. (1975)

Power law model or log-log model is suggested as

$$\epsilon_p = \alpha_3 * N^{\alpha_4} \tag{6.2}$$

Where

α_3, α_4 = model constants;

N = Number of load repetitions

The The regression constants α_3, α_4 and the R^2 values obtained for all pond ash-based specimens based on power law model are given in Table 6.2. It can be observed that the obtained R^2 values from power law model are less than 0.8 for most of the cases, and was not able to model the ϵ_p behaviour due to do not considering the effect of stress levels.

Table 6.2. Regression analysis model constants of permanent deformation for Model 2

		P	**PL_8**	**$PF_{1.0}$**	**$PL_8F_{0.5}$**	**$PL_8F_{1.0}$**	**$PL_8F_{1.5}$**	**$PL_8F_{2.0}$**	**$PL_4F_{1.0}$**	**$PL_6F_{1.0}$**	**$PL_{10}F_{1.0}$**	**$PL_{12}F_{1.0}$**
Model 2	α_3	1.973	0.317	1.200	0.333	0.333	0.285	0.272	0.247	0.548	0.445	0.254
	α_4	0.049	0.107	0.052	0.105	0.105	0.108	0.107	0.108	0.115	0.119	0.130
	R^2	0.615	0.459	0.765	0.386	0.386	0.523	0.498	0.477	0.897	0.955	0.725

6.3.3 Model 3 (M3): A Three-Parameter Model, (Ullditz 1993)

To take into account the stress dependency nature along with load cycles, the prediction model is improved as follows.

$$\epsilon_p = \alpha_5 * \left(\frac{\sigma_d}{P_a}\right)^{\alpha_6} * N^{\alpha_7} \tag{6.3}$$

Where,

α_5, α_6 and α_7= model constants

σ_d = deviatoric stress (kPa)

P_a = Atm. Pressure

N = Number of load repetitions

The regression constants α_5, α_6 and α_7, and the R^2 values obtained for all pond ash-based specimens based on model 3 are given in Table 6.3. It can be observed that the obtained R^2 values from model 3 are greater than 0.8 for all cases, indicating that the ϵ_p behaviour can be predicted well using the model 3.

Table 6.3. Regression analysis model constants of permanent deformation for Model 3

		P	PL8	PF1.0	PL8F0.5	PL8F1.0	PL8F1.5	PL8F2.0	PL4F1.0	PL6F1.0	PL10F1.0	PL12F1.0
Model 3	α_5	1.404	0.684	1.117	0.720	0.720	0.645	0.636	0.614	0.805	0.724	0.597
	α_6	0.293	0.796	0.210	0.995	0.995	0.720	0.754	0.792	0.293	0.195	0.534
	α_7	0.014	0.013	0.027	-0.012	-0.012	0.023	0.018	0.014	0.080	0.096	0.067
	R^2	0.965	0.861	0.967	0.933	0.933	0.889	0.888	0.889	0.990	0.995	0.919

6.3.4 Model 4 (M4): A Three-Parameter Model, (Puppala et al. 1999)

In addition to the growth-type prediction models, a representative model proposed by Puppala et al. (1999) also depicted the effects of both stress levels and number of load cycles on the buildup of permanent strain as

$$\epsilon_p = \alpha_8 * \left(\frac{\sigma_{oct}}{P_a}\right)^{\alpha_9} * N^{\alpha_{10}} \tag{6.4}$$

Where,

α_8, α_9 and α_{10}= model constants

σ_{oct} = Octahedral normal stress = $(\sigma_{d} + 3\sigma_c)/3$

σ_d = deviatoric stress (kPa)

σ_c = confining pressure (kPa)

P_a = Atm. Pressure = 101.4 kPa

Table 6.4. Regression analysis model constants of permanent deformation for Model 4

		P	PL8	PF1.0	PL8F0.5	PL8F1.0	PL8F1.5	PL8F2.0	PL4F1.0	PL6F1.0	PL10F1.0	PL12F1.0
Model 4	α_8	1.383	0.657	1.105	0.685	0.685	0.623	0.613	0.590	0.793	0.717	0.582
	α_9	0.314	0.855	0.226	1.066	1.066	0.773	0.810	0.851	0.314	0.208	0.573
	α_{10}	0.014	0.013	0.027	-0.012	-0.012	0.023	0.018	0.014	0.080	0.096	0.067
	R^2	0.967	0.865	0.968	0.936	0.936	0.893	0.892	0.893	0.991	0.995	0.921

The regression constants α_8, α_9 and α_{10} and corresponding regression coefficient of the above model are calculated from the statistical regression analysis and shown in Table 6.4. It can be observed from the table that the model 4 show a good correspondence with the permanent strain behaviour of treated and compacted pond ash in an efficient way as the R^2 values obtained for all the samples are greater than 0.8.

From the multi regression analysis of permanent deformation, it is observed that the regression coefficients of two-parameter based models were observed to be less (M1 and M2 < 0.8) than the three-parameter models (M3 and M4 > 0.8). This is due to the consideration of only load cycles in the two-parameter model evaluation. While the three-parameter models considered the effect of stress levels along with load cycles as previous researches reported the influence of stresses in developing deformations in pavement structure, which resulted in higher correlation values than M1 and M2. Thus, the present study concludes that the development of permanent strains for pond ash-based mixtures can be predicted well using three-parameter models (Model 3 and Model 4).

6.4 Summary

This chapter, a series of repeated load triaxial tests were carried out on pond ash specimens to study its permanent deformation behaviour. The effect of lime, fiber and lime-fiber modification, number of load cycles and cyclic stress levels on the permanent deformation behaviour of the pond ash was investigated. With an increase in both σ_d and load cycles (N), the permanent strain (ϵ_p) values of both untreated and treated specimens increased. Compared to untreated pond ash, the ϵ_p values were less in treated specimens (57% in $PL_{8\%}$, and 43% in $PF_{1.0\%}$). Modification of pond ash with both admixtures further decreased ϵ_p (65% at both optimum), and this reduced strain rate indicate the increase of life span of the pavement structure. In evaluating the permanent strain values Model 3 followed by Model 4 were found to be useful to fit the experimental data as it considers the effect of both σ_d and load cycles (N) acting on the specimen.

CHAPTER – 7

COST ESTIMATION

7.1 General

Using pond ash in road works as subbase material with improved mechanical behaviour, results in economic construction. Moreover, environmental protection achieved by the effective utilization of pond ash is well beyond the assessment. Accordingly, this study quantifies the cost of road construction comprised of pond ash modified with lime and fiber to their optimum composition.

7.2 Recommended Design Nalues for Pavement Subbase

The use of pavement materials in subbase material should be expressed the modulus values of minimum 100MPa to a maximum of 300MPa-350MPa (IRC: 37-2018). However, under laboratory conditions (at a particular stress level), the obtained M_R value of the optimized mix (PL_8F_1) is shown as 131MPa, which is slightly satisfied with the subbase layer criteria. By considering the safety factors for field variations, these laboratory values may be further decreased during the pavement design. Henceforth, from the field point of view, the investigated pond ash mixture may not be suitable for high volumes flexible pavement roads (National Highways and important state highways). Even though the strength criteria like UCS, CBR met its criteria for subbase applications. However, the same can be used for unpaved/low volume roads with a traffic volume of up to 10msa.

7.3 Analysis for Minimum Required Thickness of Pavement

A pavement design has been taken up based on IRC: 72-2015 “Guidelines for the design of flexible pavements for low-volume roads” to demonstrate whether the proposed mix meets the economic. This code consists of design charts for stabilized base and subbase; and it enables the design of roads with a traffic volume of 0.1 to 2 million standard axels, divided into nine traffic categories (T1 to T9). However, in the current study, the traffic categories T6 to T9 are selected for the design analysis based on minimum traffic of 0.5msa consideration (Table 7.1). The corresponding pavement design catalogues are presented in Figure 7.1. Also, a CBR of subgrade equal to 5 is considered for the study.

Table 7.1. Considered Pavement sections for design analysis (as per IRC 72-2015)

Traffic Category	T_6	T_7	T_8	T_9
Traffic volume (msa)	0.3 to 0.6	0.6 to 1.0	1.0 to 1.5	1.5 to 2.0

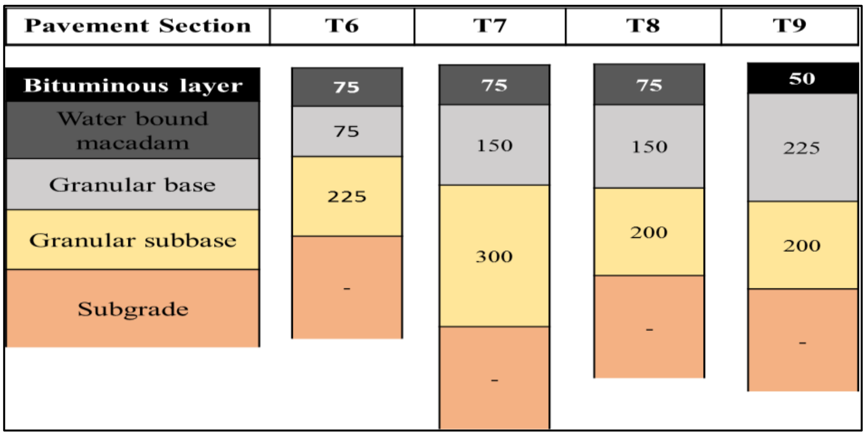

Fig. 7.1. Layer thicknesses of various pavement sections as per IRC 72

Initially, the design thickness of each pavement layer (conventional) was taken from an appropriate CBR Plate presented in IRC: 72-2015. To obtain the vertical strains at the top surface of subgrade, a multi-layered elastic model was designed in KENLAYER software. Since KENLAYER tool is user-friendly and is used to predict the performance of flexible pavements easily and efficiently. Table 7.2 shows the properties of materials and Figure 7.2 shows the schematic view of input details considered for the KENLAYER pavement analysis. During the analysis, the thickness of treated subbase was varied based on the equivalent vertical strain of pavement structure until the optimised thickness determined.

Table 7.2. Properties of materials considered in the study

Layer	**Material**	**M_R (MPa)**	**Poisson's ratio (v)**
Bitumen	BM	500	0.35
Water bound macadam	WBM	Calculate based on thickness	0.35
Granular Base	GB	Calculate based on thickness	0.35
Treated Subbase	PL_8F_1	131	0.25
Subgrade	Soil	50	0.35

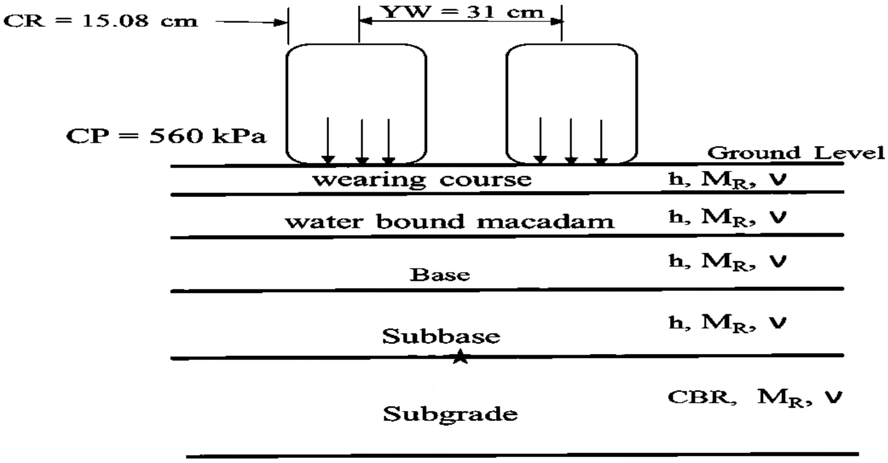

Fig. 7.2. Schematic view of input details considered in KENPAVE analysis

Table 7.3. Optimized thicknesses of T6 Pavement section

T6	**CSB**	**$PL_8F_{1.0}$**
Bitumen	-	-
Water bound macadam	75	75
Granular Base	75	75
Granular subbase/ Treated Subbase	**225**	**120**
Subgrade	-	-
Total Thickness	375	270
Vertical compressive strains (µm)	**1831**	**1806**

Table 7.4. Optimized thicknesses of T7 Pavement section

T7	**CSB**	**$PL_8F_{1.0}$**
Bitumen	-	-
Water bound macadam	75	75
Granular Base	150	150
Granular subbase/ Treated Subbase	**200**	**110**
Subgrade	-	-
Total Thickness	425	315
Vertical compressive strains (µm)	**1453**	**1314**

Table 7.3-7.6 shows the maximum vertical compressive strain at the top of the subgrade vs subbase thickness for the cases when the subbase layer is made up of pond ash mixed with lime and fiber (PL_8F1), respectively, having the properties as given in Table 7.2. As per the specifications, the minimum subbase thickness required to be 100mm. Hence, by considering min thickness of the sub-base and equivalent vertical strains of pavement structures, all the analyses were performed and determined optimum cross-sections for pavement roads of T6, T7, T8 and T9.

Table 7.5. Optimized thicknesses of T8 Pavement section

T8	**CSB**	**$PL_8F_{1.0}$**
Bitumen	-	-
Water bound macadam	75	75
Granular Base	150	150
Granular subbase/ Treated Subbase	**300**	**210**
Subgrade	-	-
Total Thickness	525	435
Vertical compressive strains (µm)	**971**	**952**

Table 7.6. Optimized thicknesses of T9 Pavement section

T9	**CSB**	**$PL_8F_{1.0}$**
Bitumen	50	50
Water bound macadam	-	-
Granular Base	225	225
Granular subbase/ Treated Subbase	200	115
Subgrade	-	-
Total Thickness	475	400
Vertical compressive strains (µm)	**1024**	**1010**

7.4 Economic Assessment

It is essential for professional designers and construction managers to have an estimation of the construction cost (Tavafzadeh Haghi et al. 2019). In the present study, the pavement is designed for a single subgrade soil. The various layers considered are shown in Table 7.3 to Table 7.7.

The unit cost of the various construction tasks as mentioned rates summarises Material/Manpower/Machinery/contractors profit and overhead charges (MoRTH 2018, Sarkar and Dawson 2015). The cost used here is as per Scheduled rates of PWD works, Telangana (also, some information is obtained from field engineers).

For 1 Km road with a single lane of width w = 3.75m, the approximate quantity and its costs were shown in Table. 7.7

<u>Details considered in cost-analysis</u>

Cost of Pond ash transport = 300/-ton (<50 km lead)   Density of fiber = 920 kg/m^3

Cost of fiber = 100/- per Kg   Cost of lime = 15/- per Kg

Density of PLF Mix = 1289 kg/m^3 Density of coarse aggregate = 1545 kg/m^3

Density of fine aggregate = 1650 kg/m^3 Bulk density of mix = 2376 kg/m^3

<u>Cost of Road per KM (width 3.75m)</u>

Table 7.7. Cost per unit

Layer	**Unit**	**Rate (Rs.)**
BC layer	Cum	6402
Water bound macadam	Cum	1726
Granular base	Cum	1424
Granular subbase Pond ash-based subbase	Cum Cum	1424 2510
Subgrade	Cum	157

Table 7.8. Cost analysis of subbase course

Mix	Conventional subbase pavement			PLF Mix-based subbase pavement			% Save in cost
T6	**Depth (m)**	**Quantity**	**Total (Rs)**	**Depth (m)**	**Quantity**	**Total (Rs)**	
WBM	0.075	281.5	486009.8	0.075	281.5	486009.8	
GB	0.075	281.5	400940.5	0.075	281.5	400940.5	
GSB	0.225	843.75	1201753	0.12	450	1129500	
SG	0.5	1875	294843.8	0.5	1875	294843.8	
Total	0.875		23,83,547	0.875		23,11,294	**3.03**
T7	**Depth (m)**	**Quantity**	**Total (Rs)**	**Depth (m)**	**Quantity**	**Total (Rs)**	
WBM	0.075	281.5	486009.8	0.075	281.5	486009.8	
GB	0.15	563	801880.9	0.15	563	801880.9	
GSB	0.2	750	1068225	0.1	375	941250	
SG	0.5	1875	294843.8	0.5	1875	294843.8	
Total	0.875		26,50,959	0.875		25,23,984	**4.79**
T8	**Depth (m)**	**Quantity**	**Total (Rs)**	**Depth (m)**	**Quantity**	**Total (Rs)**	
WBM	0.075	281.5	486009.8	0.075	281.5	486009.8	
GB	0.15	563	801880.9	0.175	656	934340.8	
GSB	0.3	1125	1602338	0.15	562	1410620	
SG	0.5	1875	294843.8	0.5	1875	294843.8	
Total	0.875		31,85,072	0.875		31,25,814	**1.86**
T9	**Depth (m)**	**Quantity**	**Total (Rs)**	**Depth (m)**	**Quantity**	**Total (Rs)**	
BC	0.05	187.5	1200375	0.05	187.5	1200375	
GB	0.225	843.75	1201753	0.225	843.75	1201753	
GSB	0.2	750	1068225	0.105	393.75	988312.5	
SG	0.5	1875	294843.8	0.5	1875	294843.8	
Total	0.875		37,65,197	0.875		36,85,284	**2.12**

From the above calculations, it is estimated that compared to conventional subbase layer pavement, the pond ash-based subbase pavement showed the decreased thickness in all cases, and a saving in cost ranges from 1.86% - 4.79% for 1 Km road with a single lane of width w = 3.75m.

CHAPTER – 8

CONCLUSIONS

8.1 Brief Conclusions from Each Phase

In this study, the experimental program was divided into five phases (Figure 1). The first phase concerns the physical and engineering properties of materials used in the study (pond ash, lime, and fiber). The second phase deals with the compaction and strength characteristics (UCS, CBR) of pond ash (untreated, treated with lime and fiber), studied the effect of additives addition, curing period. The third phase deals with the effect of lime, fiber, lime-fiber, confining and deviatoric stresses on resilient modulus characteristics (M_R) of pond ash, and performed regression model validation of experimental M_R values with existing models. Similarly, the fourth phase deals with the effect of lime, fiber, lime-fiber, deviatoric stresses, and load cycles on the permanent deformation characteristics of pond ash and performed model validation (ϵ_p) with existing models. Phase five deals with pavement design analysis by thickness optimization and its evaluation of its economic assessment.

8.2 Phase-I

This objective deals with strength characteristics of pond ash modified with lime and fiber addition on for pavement subbase applications. From the experimental results, the following conclusions are drawn.

- The modification of pond ash with lime caused an increase of MDD and decrease of OMC. While the inclusion of fibers in both untreated and lime treated pond ash slightly alters compaction parameters.
- UCS, CBR of lime treated pond ash increases linearly with an increase of lime content due to cementitious gel formation (CSH, CASH). The increment rate is significant only at 6% to 8% lime content; after that, it shows only marginal increment. The failure strains of UCS were observed to be low with brittle nature and observed to be in the range of 0.75% to 1.2% for most specimens. Based on IRC specifications, pond ash with 8% lime content satisfies the strength requirements.
- As compared to CBR of compacted pond ash, CBR values of lime treated pond ash samples are 2.60-5.69 and 3.07-6.76 times higher for 7 and 28, respectively. As per IRC

specification, the subbase material should have a minimum of 30% soaked CBR. Pond ash with minimum lime content of 6% satisfies the criteria.

- Reinforced pond ash specimen showed the rate of gain in CBR values maximum up to 1% fiber content; after that, the increment rate has been reduced with a further addition of fiber.
- The UCS and failure strain of $PL_{\%}F_{\%}$ mixtures were more than that of corresponding $PL_{\%}$ specimen and changed failure mode from brittle to ductile, with a significant improvement rate at the $PL_{8\%}F_{1\%}$ combination.
- Addition of lime and fiber in pond ash could improve the CBR values in linear trend with improved bearing capacity and satisfied the CBR criteria as per IRC specifications (except for low lime content).

8.3 Phase-II

Based on the test results (objective 1), the following study was focused on investigating the stiffness characteristics of pond ash by conducting repeated load triaxial tests treated with lime ($PL_{8\%}$) and fiber ($PF_{1\%}$) and in combinations (PLF) at their optimum contents (i.e. $PL_{8\%}F_{X\%}$, $PL_{X\%}F_{1\%}$). From the experimental results, the following conclusions are drawn.

- M_R values of untreated pond ash do not show much variation with stress levels acting on the specimen. Whereas, lime/fiber treated pond ash showed a linearly improved MR behaviour due to strain hardening nature/ development of tensile resistance forces at all increased stress levels. M_R values vary from 34 MPa-163 MPa and 21 MPa -62 MPa for lime and fiber-reinforced pond ash compared to compacted pond ash of 13 MPa-25 MPa respectively.
- The combined effect of lime and fiber on M_R of pond ash leads to further improvement and 43 MPa-196 MPa for lime-fiber treated pond ash due to increased mobilized frictional bonding area between fiber-ash particles with significance up to at both optimum content combination ($PL_{8\%}F_{1\%}$).
- The increment rate in M_R was observed to be decreased with an increase of deviatoric stresses for higher confining stresses for all test specimens due to the reduction in lateral strain deformations.
- Based on the regression model studies of M_R on four stress-dependent models, the three-parameter models provide a good correlation coefficient (R^2 > 0.9) than the two-

parameter models as they do consider the only effect of either confining stress (Model 1) or deviatoric stress (Model 2) and neglect its combined stresses effect acting on the specimen.

8.4 Phase-III

This part of the study presents the permanent deformation behaviour of pond ash modified with lime, fiber and lime-fiber. From the experimental results, the following conclusions are drawn:

- Under repeated loading conditions, with an increase in both σ_d and load cycles (N), the ϵ_p values of untreated samples increased. After modification of pond ash with lime and fiber, the strains values were observed to be less (57% in $PL_{8\%}$, and 43% in $PF_{1\%}$).
- Modification of pond ash with both admixtures further decreased ϵ_p (65% at both optimum), and this reduced strain rate indicates the increase of life span of the pavement structure.
- In evaluating the permanent strain values, both Model 3 and Model 4 were found to be suitable as they consider both σ_d and load cycles acting on the specimen ($R^2 > 0.9$). Whereas, Model 1 and Model 2 consider the only number of load cycles without accounting the effect of σ_d, resulting in lower correlation values ($R^2 < 0.9$).

8.5 Conclusions from project work:

The following conclusions are drawn based on the experimental investigations carried out in the present work.

- The strength and resilient characteristics of coal ash used in pavement applications depend on its source and various factors. Such characteristics of pond ash (Class F) used in the present study were found to be very low.
- Stabilization of pond ash with lime improved the UCS, CBR, and M_R and reduction in ϵ_p of pond ash due to cementitious gels formation. The improved rate was observed to be significant at 6% to 8% lime content; after that, it was a marginal increase only. Hence, 8% lime can be considered as an optimum lime for pond ash stabilization.
- Reinforcing pond ash with fiber inclusions enhanced the performance in terms of increased bearing values, M_R, and lower ϵ_p values compared to compacted pond ash due

to frictional and interlocking forces development between fiber and ash a considerable improvement up to 1% fiber.

- Although appreciable results were observed with fiber reinforcement, CBR and M_R values did not satisfy the criteria as per IRC specifications. Hence its use alone cannot be appropriate for pavements.
- The combined effect of both lime and fibers in pond ash improved the UCS, CBR, M_R and decreased ϵ_p values than their individual; the results are more encouraging, especially at their combined optimum contents (8% L and 1.0% F).
- The improvement in M_R values was observed with lime and fiber, and increased with applied stress levels (σ_c and σ_d). At higher σ_c, the increment rate in M_R was observed to decrease with an increase of σ_d.
- The permanent deformation characteristics of modified pond ash were reverse to the broad trends observed in M_R, UCS, and CBR. The higher the M_R value, the higher the resistance against deformations, leading to lower ϵ_p values.
- The model studies carried out with M_R, and ϵ_p results of both untreated and treated pond ash samples showed that the three-parameter based models (Universal and Octahedral shear for M_R), and (Model 3 and Model 4, for ϵ_p) were able to fit effectively. These models consider the combined effect of σ_c, σ_d, shear stresses, and load cycles and σ_d acting on the specimen.
- Apart from the improved behaviour of pond ash with lime and fiber, the obtained results indicated the minimum requirement range of UCS and M_R values (except CBR values) for its applicability in high volume roads. Hence their use may not be suitable in them; still, the same can be used effectively for low-volume flexible roads.

Compared to conventional subbase layer pavement, the pond ash-based subbase ($PL_{8\%}F_{1.0\%}$) pavement showed a reduced thickness with a saving of around 1.86% - 4.79% of total costs (for laying 1Km road with a single lane of width w = 3.75m).

8.6 Future Scope of the Investigation

- A comparative study of fiber reinforced ashes with different types of fibers can be conducted to find out economical fiber that can be used for reinforcing the pond ashes.

- Combination of various cementitious additives and reinforcing elements in coal fly ash to study its resilient characteristics to use them in high volume roads.

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Publications Related to the Work

Journals

1. **Mogili, S**., Mudavath, H., Gonavaram, K. K., & Paluri, Y. (2020). Strength and resilient behavior of lime modified pond ash as pavement layer. *Materials Today: Proceedings*, *32*, 567-573. https://doi.org/10.1016/j.matpr.2020.02.168 (SCOPUS)
2. **Mogili, S.,** Mohammed, A., Mudavath, H., & Gonavaram, K. K. (2020). Mechanical strength characteristics of fiber-reinforced pond ash for pavement application. *Innovative Infrastructure Solutions*, *5*(3), 1-12. https://doi.org/10.1007/s41062-020-00313-y (SCOPUS)
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4. **Mogili, S.,** Mudavath, H., & Gonavaram, K. K. (2021). Role of Lime-Fiber on Mechanical Strength and Resilient Characteristics of Pond ash. *Transportation Infrastructure Geotechnology*. https://doi.org/10.1007/s40515-021-00149-8 (SCOPUS)

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1. **Mogili, S.,** Mudavath, H., & Gonavaram, K. K. (2020). Mechanical Strength Behaviour of Lime treated Pond Ash for Pavement Applications. *3rd International Conference on Innovative Technologies for Clean and Sustainable Development* ***(ITCSD-2020)***, Chandigarh, India.
2. **Mogili, S.,** Mudavath, H., & Gonavaram, K. K. (2020). Resilient behavior of coal pond ash. *Fifth International Online Conference on Reuse and Recycling of Materials* ***(ICRM-2020).*** Mahatma Gandhi University, Kerala, India.
3. **Mogili, S.,** Mudavath, H., Gonavaram, K. K. (2020). Effect of Stabilization on Geomechanical Properties of Pond ash for Pavement Subbase application. *Indian Geotechnical Conference* ***(IGC-2020),*** Vishakapatnam, India.
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