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Soil Behaviour and Characterization: A Study on Improvement of CBR Characteristics of Soil Subgrade

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Abstract. Soil characterization with the help of interpretation of in-situ and laboratory test data have been evolving from basic empirical recommendations to a more advanced area demanding a thorough knowledge of material behaviour. With the advent of modern testing techniques and more rigorous methods of analysis, site characterization in natural soils is gaining momentum. Since the in-situ behaviour of natural soils is complex, a general recommendation is to cross-correlate measurements from different tests. When data are combined there is more scope for rational interpretation and, for this reason, emphasis has been placed on correlations with mechanical properties that are based on the combination of independent measurements. In the present study, an attempt has been made to study the soil behaviour in terms of CBR characteristics for two methods of soil character enhancement, viz. addition of lime and rice husk ash (RHA) to soil and the method of improvement of soil subgrade overlain by compacted fly ash and geotextile at interface. It has been found that the latter method proves to be a better means of enhancing the soil character at high water contents. But the method of soil improvement by addition of lime and RHA, somewhat yields a better result at OMC than the soil – geotextile – fly ash matrix method. A design of bituminous pavement with granular base and sub-base layers has also been attempted using the guidelines laid by IRC 37:2018 and IITPAVE software to find out the adequacy of the two subgrade improvement methods with respect to pavement design.

Keywords: Soil behaviour, Subgrade improvement, CBR, Lime, RHA, Flyash, Geotextile interface, IITPAVE

1 Introduction

Soil behaviour is complex due to the fact that it is a three phase system containing soil, water and air. It depends upon various factors like the age of deposition, the geological history of the deposit and the stress history, which affects the size, shape, mineral composition and packing of the particles etc. In wide variety of applications in geotechnical engineering, such as, foundation of structures, embankment design, improvement of weak soil, etc., understanding of soil behaviour (under seismic and non-seismic conditions) with respect to soil characterization and strength has become very important to a geotechnical engineer. The properties related to soil characterization, strength, compressibility and compaction are obtained by different parameters. Grain size, shape and Atterberg's limits for characterization; shear strength parameters and shear modulus for soil strength; coefficient of consolidation and compression index for soil compressibility; and optimum moisture content along with maximum dry density for compaction characteristics etc. influence soil behaviour pertinent to geotechnical applications. But soil character may themselves vary significantly even within a zone, both laterally and vertically. Because of the variation and uncertainty in soil properties, judicious interpretation of soil test data has to be done. Inadequate soil investigation may lead to project delays, failures and cost over-run. Thus, the planning of soil investigation and its proper execution including proper testing must be a part of the entire design process. Acquisition of topographical, hydro-geological, geotechnical and geoenvironmental data along with appropriate feasibility study play an important role in success of a project. With respect to geotechnical engineering, the primary purposes of site investigation involve (a) evaluation of the general suitability of the site for the proposed project, (b) adequate and economical planning, (c) physical and mechanical properties of soils for design and construction, (d) groundwater scenario, and (e) suitability of materials for construction. Both in–situ (Standard Penetration and Cone Penetration Tests) and laboratory (Grain size analysis and Atterberg's limits) tests are conducted for geotechnical site characterization. Since each testing technique responds to different physical properties, a successful and cost-effective site characterization programme should consist of an appropriate combination of field and laboratory tests, so that the relevant information regarding a project site may be obtained. This results in establishing suitable constitutive relationships to obtain appropriate material behaviour leading to an optimized engineering design with respect to geotechnical aspects. Figure 1 illustrates the various stages associated with site characterization in form of a flowchart.

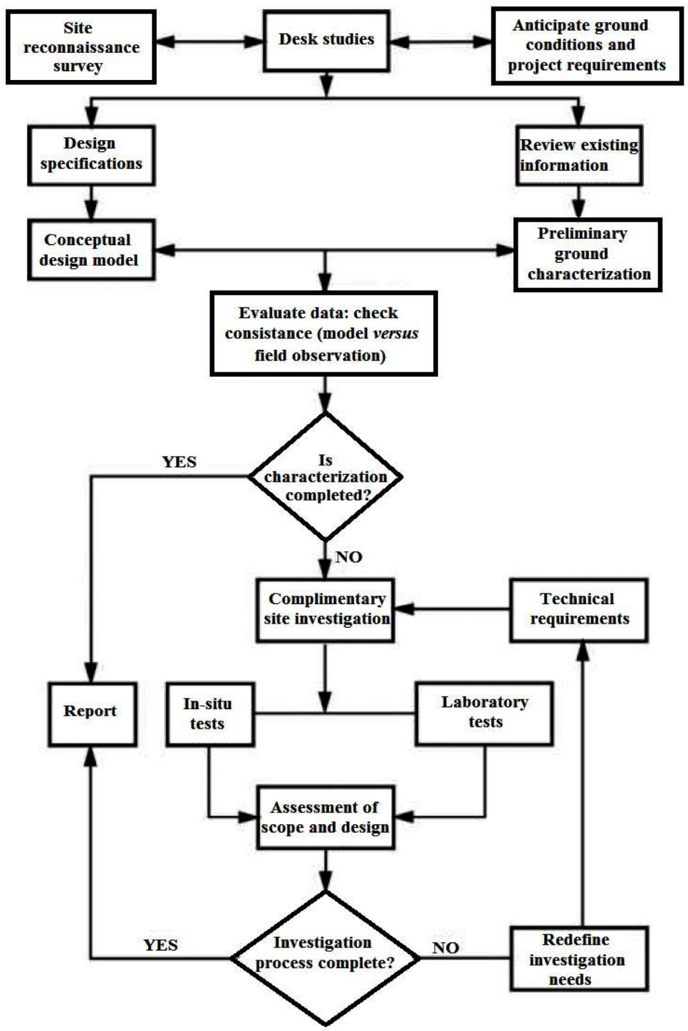

Fig. 1. Site Characterization Flowchart

In the present study, an attempt has been made to study the soil behaviour in terms of CBR characteristics for two methods of soil character enhancement, viz. addition of lime and rice husk ash (RHA) to soil and the method of improvement of soil subgrade overlain by compacted fly ash and geotextile at interface. Further, design of a bituminous pavement with granular base and sub-base layers has also been attempted using the guidelines laid by IRC 37:2018 and IITPAVE software to find out the adequacy of the two subgrade improvement methods with respect to pavement design.

2 Background

Laboratory tests help in understanding the basic soil behaviour. The shear strength parameters and stress-strain relationship obtained from triaxial tests are incorporated to develop constitutive models, in particular for the development of Critical State Soil Mechanics [1] and the family of Cam-Clay models [2]. The recent developments in laboratory testing for experimentally determining the stress-strain-strength and time dependent properties of geomaterials have ushered in a new era with (a) growth of a new generation of devices such as hollow cylinder, resonant column and torsional shear apparatus, (b) extensive use of stress path with computer based systems and (c) development of new techniques for more accurate measurements of local strains and imposed loads [3–6]. All these help in solving various geotechnical problems in a more complex environment, including the effects of microstructure (fabric and bonding), stiffness non-linearity, small and large strain anisotropy, partial saturation and viscosity. The ground behaviour is understood on the basis of laboratory tests conducted on reconstituted soils and natural clays and sands (e.g. [7–13]). In order to predict laboratory test data, attempts have also been made to obtain a large set of in–situ test results which subsequently can be used to establish some correlation between the two. According to Schnaid et al [14], there are three challenges with in situ field tests: (a) extending the modelling of standard clay and sand behaviour to other geomaterials based on existing theoretical and empirical concepts, (b) development of appropriate innovative constitutive model, and (c) validation of proposed models on the basis of large number of experimental data.

Existing methodologies of field testing can be broadly divided into two major classes: (a) non-destructive tests done with least possible disturbance of soil structure and allowing very little modification of the initial mean effective stress of soil during the installation process, for example, seismic techniques, pressuremeter probes, plate load tests, etc., and (b) destructive tests carried out with noticeable disturbance caused by the penetration or installation of the probe into the ground, for SPT, CPT, etc. Due to complexity in behaviour of geomaterials, ongoing research is encouraging prediction of mechanical properties based on the combination of different sensors in a single test device, with feasibility of conducting tests by combinations of a non-destructive and an invasive technique, such as, the seismic cone and cone pressuremeter. A summary of the commonly used in-situ tests is given in Table 1.

Table 1. Commercial in situ testing techniques (modified from Schnaid et al [13])

Category	Test	Measurements	Common Applications
Non-destructive tests	Geophysical tests: Seismic refraction (SR) Surface waves (SASW) Crosshole test (CHT) Downhole test (DHT)	 P-waves from surface R-waves from surface P & S waves in boreholes P & S waves with depth	Ground characterization Small strain stiffness, G_o
	Pressuremeter tests: Pre-bored (PMT) Self-boring (SBPM)	Shear modulus, shear stress vs. shear strain data	Shear modulus, G Shear strength In-situ horizontal stress Consolidation properties
	Plate loading test (PLT)	Load-Deformation data	Stiffness and strength
Invasive penetration tests	Cone penetration tests: Electric (CPT) Piezocone (CPTU)	Cone resistance, sleeve friction, pore pressure	Soil profiling Shear strength Relative density Consolidation properties
	Standard penetration test (SPT)	Penetration (N value) per certain number of blows of a standard hammer	Soil profiling Internal friction angle, φ’
	Flat dilatometer test (DMT)	Corrected first and second readings p_o, p_1; p_o corresponds to membrane movement against the soil (lift off), p_1 corresponds to membrane movement by 1.1 mm against the soil	Stiffness Shear strength
	Vane shear test (VST)	Torque	Undrained shear strength, s_u
Combined tests (Invasive + Non-destructive)	Cone pressuremeter test (CPMT)	Cone resistance, sleeve friction, shear modulus, shear stress vs. shear strain data	Soil profiling Shear modulus, G Shear strength Consolidation properties
	Seismic cone penetration test (SCPT)	Cone resistance, sleeve friction, compressional and shear wave velocities, V_p, V_s	Soil profiling Shear strength Small strain stiffness, G_o Consolidation properties
	Resistivity cone penetration test (RCPT)	Cone resistance, sleeve friction, soil resistivity	Soil profiling Shear strength Soil porosity
	Seismic dilatometer test (SDMT)	Corrected first and second readings p_o, p_1; p_o corresponds to membrane movement against the soil (lift off), p_1 corresponds to membrane movement by 1.1 mm against the soil, compressional and shear wave velocities V_p, V_s	Stiffness (G and G_o) Shear strength

3 Characterization of Soil Properties

3.1 The purpose of constitutive modelling

Modelling of Engineering problems. The constitutive modelling is done to use it as a tool for solving real life engineering problems. In order to examine the soil behaviour, very complex models may be entertained if a high degree of accuracy is required. But, for engineering purposes simplicity will be of overriding importance. Even though the penalty of simplicity is restricted behaviour, it is still important for an engineering application because of two reasons:

(i) The relevant soil properties must be obtained from a relatively small number of simple tests (carried out in a laboratory) or from in–situ measurements. These tests are unlikely to provide sufficient data for a highly complex model because: (a) the intrinsic accuracy of the tests may not be high, due to problems such as sample disturbance; (b) a considerable scatter may exist between test results due to geological variations of the samples; and (c) the boundary conditions on the tests may not be sufficiently well controlled to allow complete and unequivocal interpretation of the tests.

(ii) The procedure for analysis which must be used are fully developed for only a few simple types of constitutive model. Attempting to carry out analyses using complex models in elaborate numerical procedures is fraught with dangers of hidden inaccuracies, numerical instabilities, lack of unique solutions and straightforward errors.

The second major requirement is that the model should in some way reflect the underlying physical processes of the mechanics of soils. In particular, the parameters describing the soil should have a readily identifiable physical significance (e.g. shear modulus or angle of internal friction) and not be mere curve-fitting constants.

Properties required for Engineering calculations

Conventional calculations fall into two main categories: (i) limit analysis, and (ii) deformation analysis. Limit analysis is concerned with equilibrium of soil masses; deals only with the weight and strength of the soil and takes no account of deformations. In contrast, deformation analysis disregards soil strength and deals only with deformation and consolidation properties. The properties required for limit analysis are: (a) soil strength, s_u, either expressed in terms of total or effective stresses, or shear parameters (c, φ), and (b) for some cases, the bulk unit weight of soil (γ) and ground water conditions. On the other hand, the properties required for deformation analysis are: Young's modulus (E), Poisson's ratio (ν), shear modulus (G) and bulk modulus (K). In many engineering applications, the first pair (E, ν) is adopted, whereas the second pair (G, K) is more fundamental mathematically because it separates pure shear from bulk behaviour. It is also essential to distinguish between undrained and drained behaviour of soils, and whether the analysis is in terms of total or effective stresses. In case of perfectly elastic model of soil, the value of the shear modulus remains the same irrespective of the drainage conditions (the water within the soil skeleton has zero shear stiffness); therefore, it is preferable to use the shear modulus rather than Young's modulus. For conditions of no volume change in most soils, it is reasonable to take ν as approximately constant, whereas both G and K are functions of the mean effective stress, and also of the overconsolidation ratio. For settlement problems in which consolidation occurs, it is usual to represent the compressibility of the soil skeleton by the coefficient of volume change, m_v, or the compression index, c_c and the rate of consolidation by the coefficient of consolidation, c_v.

In spite of the relatively simple concepts behind the above choice of properties, precise definition and accurate measurement of them prove not to be straightforward. The consequence of developing complete constitutive relationships for describing soil behaviour is to encompass the two discrete types of analysis into one single framework in which soil is modelled as deforming monotonically (in a non-linear manner) until it reaches some limit state. This may entail the simple linking of the parameters separately used in limit and deformation analyses to ensure that consistent values are adopted (e.g. by means of the rigidity index, $I_r = G/s_u$). Alternatively, it may mean the creation of new and more complex models which will inevitably introduce new and unfamiliar parameters. These parameters should be expressible in physical terms, and be related to the above simple properties.

3.2 Definition of parameters

Material constants and state variables. In modelling the behaviour of soils, a distinction must be made between various types of algebraic quantities which appear in the equations: (i) material constants which, as their name implies, are fundamental constants which appear in the models and define the type of material, and (ii) state variables, which are quantities that vary as the soil deforms and are required within the model to define the current state of the material. Examples of material constants are shear modulus for an elastic material or the shear parameters for a plastic material. Although they must be treated as constant in any one step in a numerical calculation, this approach may not apply to a real soil (e.g. the real soil may not behave as an elastic material). Particular difficulties arise when, in studying the behaviour of a soil, the variation of a "constant" such as shear modulus is examined. If moduli are treated, for instance, as functions of stress then they become variable quantities when in another context they are fundamental constants. Some state variables (e.g. void ratio) may be easily defined and measured, whilst others (e.g. preconsolidation pressure) may prove more difficult both to define and to measure. The effective stresses are the most important set of state variables.

Choice of stress and strain variables. The geometry of the problem being solved usually determines which is the most convenient co-ordinate system to use – usually Cartesian or cylindrical, occasionally spherical. Also, in the analysis of problems, it is convenient to use certain derived quantities. Triaxial stress parameters, for instance, are best expressed in terms of the effective stress variables as defined by Roscoe et al [15]:

$$p' = (\sigma_1' + 2\sigma_3')/3 \quad (1)$$

$$q = \sigma_1' - \sigma_3' \quad (2)$$

where, p′ = mean effective stress, q = deviator stress, σ_1' = effective vertical stress in triaxial test, and σ_3' = the effective lateral pressure in triaxial test.

Corresponding to these stress variables, the strain variables are:

$$\nu = \varepsilon_1 + 2\varepsilon_3 \quad (3)$$

$$\varepsilon = 2(\varepsilon_1 - \varepsilon_3)/3 \quad (4)$$

where, ν = volumetric strain, ε = distortional strain, ε_1 = major principal strain, and ε_3 = minor principal strain.

These variables are carefully chosen so that the input incremental work (dW) can be expressed as:

$$dW = p'd\nu + qd\varepsilon \quad (5)$$

These variables are closely related to the stress and strain invariants, and also have a simple physical meaning. Wood [16] has given a useful discussion of the application of different stress and strain variables and also suggested a consistent notation.

3.3 Measurement of soil properties

Ideal testing conditions. Before examining the realities of soil testing, it is worth considering what should be regarded as ideal soil testing conditions. Firstly, for an ideal test, the soil sample should be removed from the ground, transported, stored and then set up in the laboratory, all with no disturbance or changes in stress acting on the sample. Clearly this is never achieved, although modern sampling techniques attempt to minimize disturbance. In view of the importance of the problem, astonishingly little work has been done on the problem of transferring a sample to a testing apparatus without relieving the total stresses. The sample should then be subjected in the test to a uniform stress, and when it deforms should do so as a single homogeneous element. Stresses, strains and pore pressure (which should also be uniform throughout the sample) should be measured accurately. The rate of testing is therefore important in drained testing of fine grained materials in order to give adequate equalization of pore pressures. Carter [17] presents calculations which demonstrate how the limits of very fast and very slow testing approach ideal undrained and drained conditions, respectively.

Departures from the ideal in laboratory tests . In practice, the above ideals are never achieved and, in addition to the problems of instrumentation and accuracy of measurement common to all scientific investigations, the following problems apply particularly to soil testing. In some soil tests, for instance the simple shear apparatus [18], an attempt is made to obtain uniform deformation, but the boundary conditions (rough horizontal faces and smooth vertical faces in the apparatus) ensure that non-uniform stresses must occur within the sample [19]. A second source of non-homogeneity is the consolidation history of the sample. In order to cause consolidation there must be hydraulic gradients, however small, within the sample and these can give rise to non-homogeneity. Even if a sample is subjected to ideal test conditions then it may not deform uniformly. Especially during post-peak conditions slip surfaces or intense shear bands may form and measurements of strain become meaningless [20–22]. The subsequent analysis of materials in post-peak and softening conditions also poses special difficulties.

Departures from the ideal in field tests. In order to avoid problems of sample disturbance and stress changes, in-situ tests may be carried out for measuring soil properties, but again problems are encountered. The first problem with the interpretation of a field test is that the initial stress conditions are not definitely known. Usually it is possible to estimate the vertical stress, but the horizontal stress may be more difficult to estimate. More complex stress states are rarely even considered. Secondly, the boundary conditions are uncontrolled, with no fixed outer boundary existing around the testing device. The tests inevitably take place under conditions in which the soil is subjected to fields of varying stress. The gradients of stresses give rise to gradients of pore pressure, with the result that in-situ tests take place under conditions, to a greater or lesser extent, of partial drainage and consolidation. Since the soil is not subjected to uniform stress in in-situ tests, such tests are not well suited to establishing stress-strain models for soils. Each must be interpreted as a boundary value problem in its own right, using a suitable model for the soil. Very few in situ tests (with the pressuremeter being a notable exception) give any measure of the strain in a soil, with most (e.g. the vane and cone penetration tests) being limited to measurements that are converted to stresses.

Relevance of tests to field problems. Even supposing that a soil test can be carried out under ideal conditions, the question should be posed as to whether the test is applicable to a field problem. For instance, triaxial tests may provide excellent stress-strain data for a limited range of stress paths in which the principal stress directions are not allowed to rotate. These data may not be relevant to a field problem like excavation, where rotation of major principal stress occurs. Within these constraints, however, a sensible choice of test from the many that are now available should be possible in order to apply a stress path which is relevant to the field problem.

Since the beginning of geotechnical engineering, it has been found that different soil properties influence the soil behaviour in different ways. Particle size distribution gives an idea about predominance of grain size that indicates whether it is a fine grained or coarse grained soil. In case of fine grained soil, Atterberg's limits show its plasticity, activity and proneness to swelling, thus indicating its important behaviour. The density and specific gravity indicates whether it contains any organic matter like decomposed wood and also indirectly relates to its shear characteristics. The natural moisture content is very important for fine grained soil. If it is nearer to plastic limit, the soil is stiff and vice versa, when it is nearer to liquid limit. This is represented by consistency index. Density index, for coarse grained soil indicates the compactness. It is high when void ratio is close to minimum void ratio. The shear strength parameters are important for obtaining bearing capacity of a foundation, slope stability analysis and determining lateral earth pressure on a flexible or rigid retaining wall. Permeability has a different significance because it helps to find quantity of seepage and seepage force, which affects stability of a hydraulic structure like dam or weir. On the basis of such interpretation of soil test results, the soil can be recommended for use as construction material for embankment or structures of appropriate loading may be allowed to be built on the existing soil. If required, the properties may be improved to some extent by adopting any suitable method.

4 Current Study

Many research works on sub-soil characterization and improving soil conditions have been undertaken in the Geotechnical laboratory of Jadavpur University in the past few years. One such method involves improvement of soft subgrade soil by stabilization with cheap and locally available materials like lime and rice husk ash (RHA). Another method has been envisaged with fly ash and geotextile. These two researches focusing on improvement of soil subgrade has been discussed here.

4.1 Improvement of soil subgrade by addition of RHA and Lime (Chakraborty, S., [23])

This study has been directed towards the strength improvement of soft soil for the flexible pavement construction. An attempt has been made to mix the local soil, considered as subgrade material, with easily available materials like rice husk ash (RHA) and lime. CBR of soil is a major parameter for strength improvement of subgrade. Therefore, the soil has been compacted at optimum moisture content and its CBR has been determined under both soaked and unsoaked conditions for different proportions of admixtures to find the optimum one. An attempt has also been made to generate some correlation to find the influence of different soil properties like Atterberg's limits, OMC and MDD by adopting multiple linear regression analysis.

Materials used. The materials used in this study are locally available clayey soil, lime and rice husk ash. The physical properties of these materials are summarized one by one as follows:

a) Soil. The soil has been collected from some locality within Kolkata Municipal Corporation area in West Bengal, India. It has been taken from a depth of 2.5 to 3.5 m below the ground level as disturbed sample. The engineering properties of the soil used in this study are summarized in Table 2.

Table 2. Engineering properties of soil

Basic Properties of Soil	Value
Sand (%)	5
Silt (%)	68
Clay (%)	27
Liquid Limit (%)	51
Plastic Limit (%)	28
Plasticity index (%)	23
IS Classification	CH
Specific Gravity	2.65
Maximum Dry Density (gm/cc)	1.630
Optimum Moisture Content (%)	15.92
$CBR_{Unsoaked}$ at OMC (%) at OMC	4.25
CBR_{Soaked} at OMC (%) at OMC	3.50

b) *Lime.* Lime is a very good stabilizing material. Lime makes the soil less permeable and improves its strength. Therefore, in this study hydrated lime has been procured from the local market for using it as an admixture.

c) *Rice husk ash.* Rice husk is obtained from rice milling as a byproduct. During milling of paddy, about 78 % of weight is recovered as rice, broken rice and bran, and rest 22 % of the weight of paddy remains as husk. Rice husk ash (RHA) is obtained by burning rice husk in open fire or boiler. It is predominantly a siliceous material annually generated in large volumes. The Physical properties of rice husk ash (RHA) are given in Table 3.

Table 3. Physical properties of RHA

Basic Properties of RHA	Value
Liquid Limit (%)	NP
Plastic Limit (%)	NP
Plasticity index (%)	NP
Specific Gravity	1.96

Maximum Dry Density(gm/cc)	0.85
Optimum Moisture Content (%)	32
Angle of internal friction(°)	38
CBR $_{Unsoaked}$ at OMC (%) at OMC	8.75
CBR $_{Soaked}$ at OMC (%) at OMC	8.15

Methodology and test programme. In order to obtain soil properties with and without admixtures, relevant tests (Atterberg's limits and Standard Proctor test) have been carried out as per IS: 2720: (Part 3–16). In case of conducting tests with admixtures, requisite quantities of soil and stabilizers have been thoroughly mixed in pre–selected proportions in dry state. Required quantity of water has then been added and mixed thoroughly to prepare a homogeneous and uniform mixture. Lastly, California Bearing Ratio (CBR) tests have been performed at OMC under unsoaked and soaked conditions for the original soil as well as for amended soil mixes, as per Table 4. The test program is presented in Table 5

Table 4. Soil-Lime-RHA mixes

Sl. No.	Soil (%)	Lime (%)	RHA (%)	Remarks
1	100	0	0	Only Soil
2	98	2	0	Soil-Lime Mixes
3	96	4	0	
4	94	6	0	
5	92	8	0	
6	90	10	0	
7	97	0	3	Soil-RHA Mixes
8	94	0	6	
9	91	0	9	
10	88	0	12	
11	95	2	3	Soil-Lime-RHA Mixes
12	92	2	6	
13	89	2	9	
14	86	2	12	
15	93	4	3	
16	90	4	6	
17	87	4	9	
18	84	4	12	
19	91	6	3	
20	88	6	6	
21	85	6	9	
22	82	6	12	
23	89	8	3	

24	86	8	6
25	83	8	9
26	80	8	12
27	87	10	3
28	84	10	6
29	81	10	9
30	78	10	12

Table 5. Test programme

Sl. No.	Test	No of Tests
1.	Atterberg's Limits (Liquid Limit, LL and Plastic Limit, PL)	30+30
2.	Standard Proctor Compaction Test (OMC, MDD)	30
3.	CBR Test at OMC (Unsoaked and soaked)	30+30

Results and discussion

a) Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI). When only lime is added, liquid limit decreases with increasing lime percentage and plastic limit increases, thereby decreasing the plasticity index. When only RHA is added, liquid limit and plastic limit both increase but not appreciably and plasticity index almost remains in the range of that of original soil although effect of plasticity index is much pronounced when RHA content is as high as 12 %.When lime and RHA are added in combination, their combined effect decreases the plasticity index. Combined effect of chemical action of lime and the pozzolanic action of RHA is responsible for the occurrence of this phenomenon. Figure 2, Figure 3 and Figure 4 show the plot of liquid limit, plastic limit and plasticity index respectively with varying percentages of lime and RHA contents.

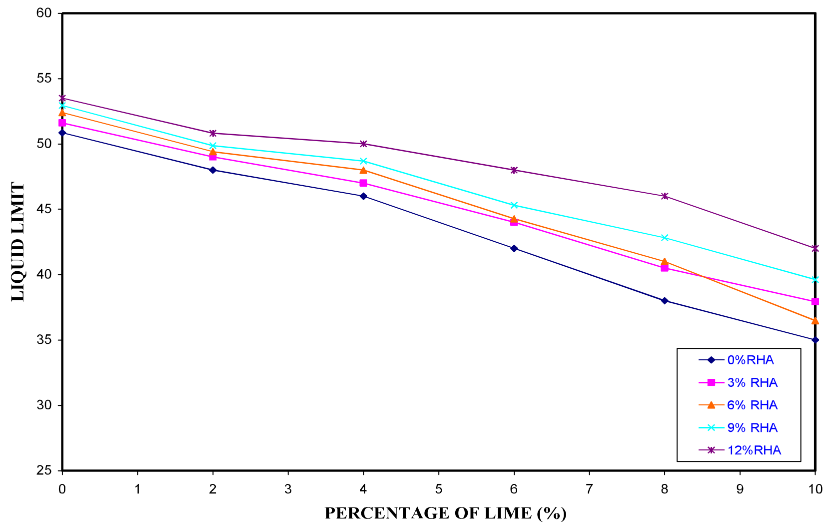

Fig. 2. Variation of liquid limit with lime content for different RHA contents

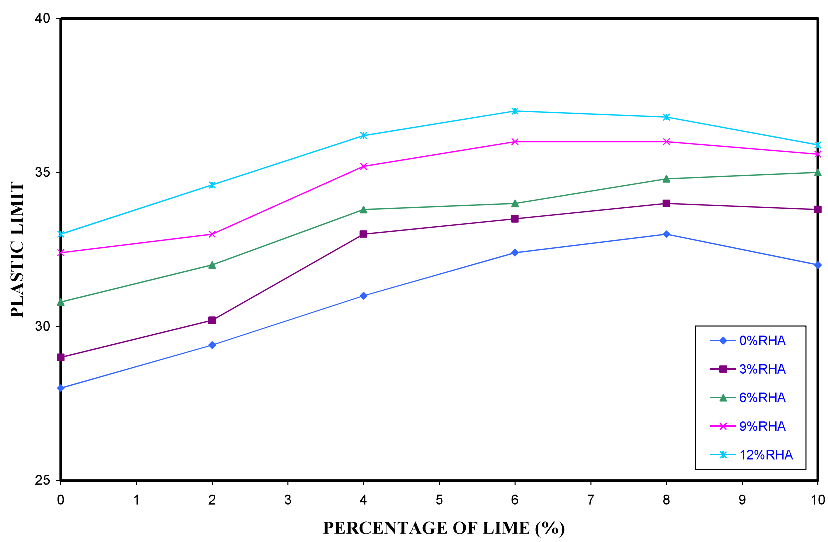

Fig. 3. Variation of plastic limit with lime content for different RHA contents

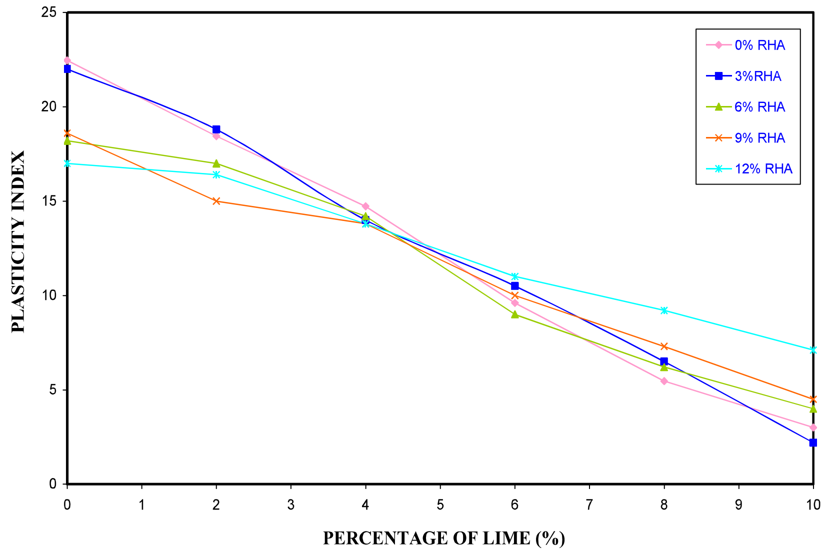

Fig. 4. Variation of plasticity index with lime content for different RHA contents

b) Compaction properties

Maximum Dry Density (MDD). The plot of maximum dry density (MDD) against different percentages of lime and rice husk ash (RHA) combinations is presented in Figure 5. The maximum dry density (MDD) generally decreases with increasing lime content. From Figure 5, it can be seen that maximum dry density (MDD) continually decreases with increase in lime content for a particular percentage of rice husk ash (RHA) admixture. Flocculation and agglomeration of clay particles caused by cation exchange reaction become the cause of decrease in dry density. Further, in case of lime treated soil, MDD decreases as resistance against compactive effort increases due to flocculated soil structure.

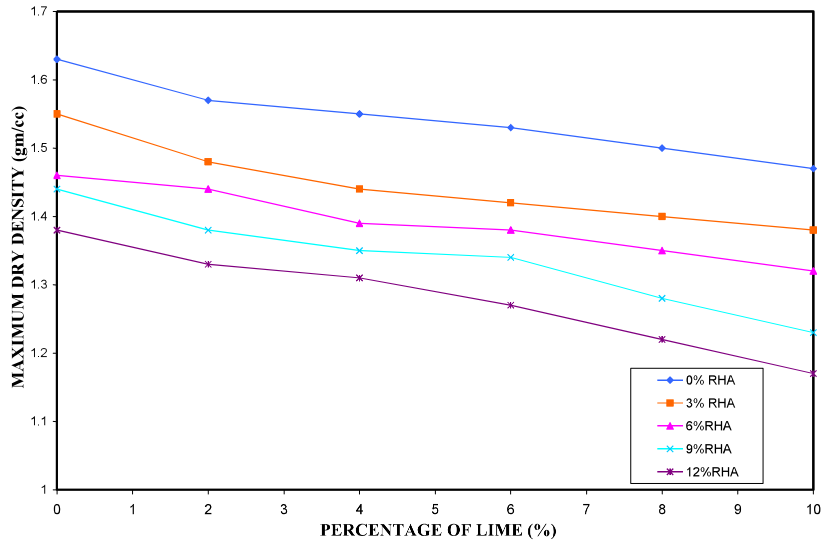

Fig. 5. Variation of maximum dry density with lime content for different RHA contents

Optimum Moisture Content (OMC). The variation of the optimum moisture content (OMC) with varying lime and RHA contents is presented in the Figure 6. Generally, the optimum moisture content (OMC) is observed to increase with increase of lime content up to 6 % and then it decreases. OMC increases due to further addition of fine contents even with reduced surface area as free lime needs more water for pozzolanic reaction. Moreover addition of RHA causes increased coarse fraction with greater surface area. This leads to increase of OMC due to addition of RHA.

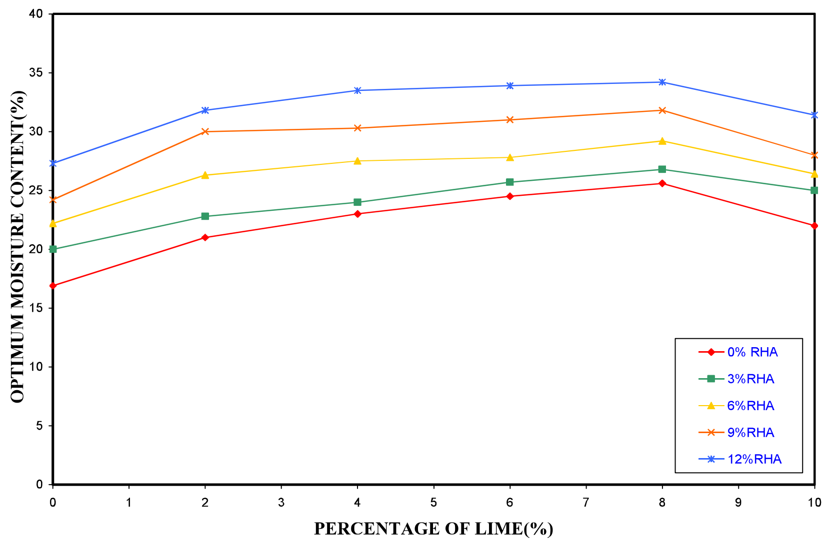

Fig. 6. Variation of optimum moisture content with lime content for different RHA contents

c) California Bearing Ratio (CBR) at OMC. The plot of unsoaked and soaked CBR for different lime and RHA contents are shown in Figure 7 and Figure 8 respectively. Both the figures show that CBR value increases with increase of percent of lime and RHA, when each is used as a single admixture and also when both are used in combination. With 6 % lime and 9 % RHA contents under unsoaked condition a maximum CBR value of 28.25 % has been found. On the other hand a maximum CBR value of 29.82 % is obtained for 6 % lime and 6 % RHA combination under soaked condition. Increase of CBR with addition of RHA occurs at low lime content due to chemical action of lime. The increase in CBR value with the addition of lime is due to the formation of various cementing agents due to pozzolanic reaction between the amorphous silica and alumina present in natural soil and lime. This reaction produces stable calcium silicate hydrates and calcium aluminates hydrates as the calcium from the lime reacts with the aluminates and silicates of the soil. It is also observed that when RHA is added to the original soil, the strength characteristics also goes on increasing but at a slower rate than that for lime mixed soil. The soaked CBR value increases with the increase in RHA content at a higher rate than unsoaked CBR. The decrease in the rate of increase of soaked CBR after 9% of RHA content at OMC may be due to the excess RHA which is not mobilized in the reaction as sufficient quantity of naturally occurring CaOH may not be present in soil. The excess RHA occupies space within the specimen and reduces the clay and silt content in soil which reduces the cohesion in the soil – RHA mixture.

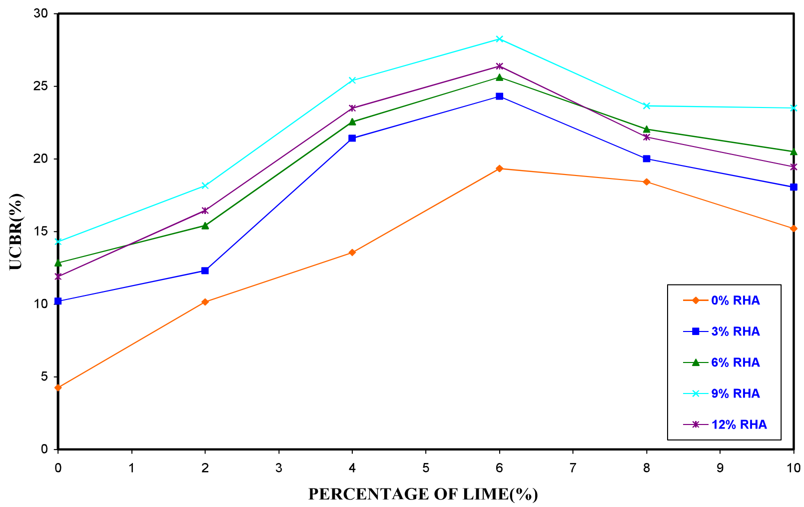

Fig. 7. Variation of unsoaked CBR with lime and RHA contents at OMC

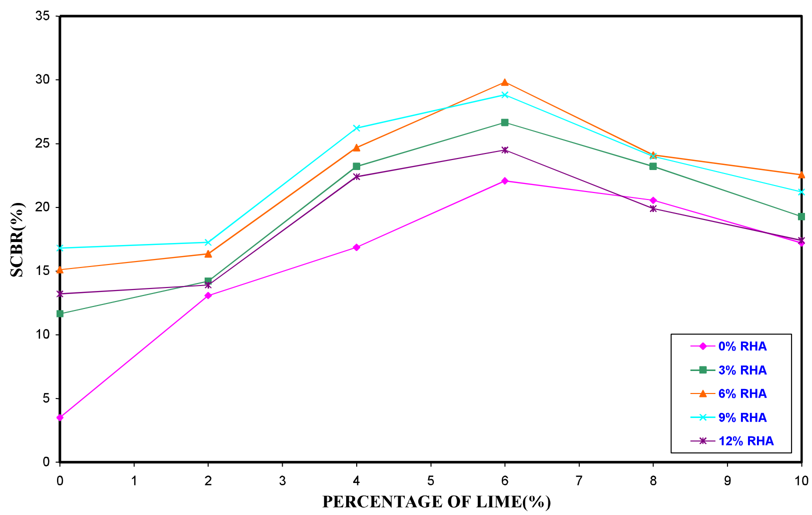

Fig. 8. Variation of soaked CBR with lime and RHA contents at OMC

d) Statistical analysis. A statistical analysis has been done with the help of Multiple Regression Analysis to obtain the following correlations in order to predict CBR value in terms of index properties and compaction characteristics of soil at the Optimum Moisture Content (OMC) under both unsoaked and soaked conditions. The validity of the correlations has been established with R^2 values equal to 0.83 and 0.71 respectively for unsoaked and soaked CBR value.

$$UCBR_{(OMC)} = 6.09LL - 2.92PL - 6.18PI + 19.16MDD - 0.29OMC - 105.52 \quad (6)$$

$$SCBR_{(OMC)} = 7.12LL - 3.97PL - 7.38PI - 22.65MDD - 0.62OMC - 97.24 \quad (7)$$

where,
UCBR $_{(OMC)}$ = CBR value at the optimum moisture content in unsoaked conditions (%)
SCBR $_{(OMC)}$ = CBR value at the optimum moisture content in soaked conditions (%)
LL = Plastic Limit (%)
PL = Plastic Limit (%)
PI = Plasticity Index (%)
MDD = Maximum Dry Density (gm/cc)
OMC = Optimum Moisture Content (%)

4.2 Improvement of soil subgrade overlaid by compacted Fly Ash and Geotextile at interface (Sengupta et al., [24])

This study examines the effect of, compaction energy and moulding water content of clay on CBR of soil – fly ash composite matrix with geotextile at the interface.

Materials used

a) Soil. The soil (silty clay) used in this study has been collected locally from a marshy land situated at Barrackpore, West Bengal, India. The geotechnical properties of soil are shown in Table 6.

Table 6. Geotechnical properties of soil

Basic Properties of Soil		Value
Sand (%)		8.34
Silt (%)		67.33
Clay (%)		24.33
Liquid Limit (%)		41.2
Plastic Limit (%)		24.16
Shrinkage Limit (%)		18.45
IS Classification		CI
Specific Gravity		2.54
Standard Proctor Compaction	Maximum Dry Density (kN/m^2)	16.88

	Optimum Moisture Content (%)	16
Modified Proctor Compaction	Maximum Dry Density (kN/m^2)	18.21
	Optimum Moisture Content (%)	12

b) Fly ash. Flyash sample used in this study has been collected from Titagarh Thermal Power Plant situated in West Bengal, India. The composition and properties of fly ash have been shown in Table 7 and Table 8 respectively.

Table 7. Composition of Fly ash

Composition	Percentage
SiO_2	61.8
Al_2O_3	22.82
Fe_2O_3	8.4
TiO_2	1.6
CaO_2	1.48
MgO	0.9
Mn_3O_4	0.156
P_2O_5	0.657
SO_3	0.357
Na_2O	0.245
K_2O	1.355

Table 8. Properties of Fly ash

Basic Properties of Fly ash			Value
Sand (%)			82.17
Silt (%)			16.83
Clay (%)			1
C_U			2.22
C_C			4.54
Specific Gravity			2.11
Standard Proctor Compaction		Maximum Dry Density (kN/m^2)	10.24
		Optimum Moisture Content (%)	41
Modified Proctor Compaction		Maximum Dry Density (kN/m^2)	11.4
		Optimum Moisture Content (%)	28
Direct Shear Test	Standard Proctor Compaction	Cohesion (kPa)	-
		φ (deg)	39
	Modified Proctor Compaction	Cohesion (kPa)	-
		φ (deg)	41

c) Geotextile. 100 % polypropylene high strength fibre woven geotextile, which is available commercially, has been collected to be used as reinforcement material in these experiments. The properties of geotextile have been summarized in Table 9.

Table 9. Properties of Geotextile

Basic Properties of Geotextile	Value
Thickness (mm)	1.5
Mass per unit area (gsm)	450
Apparent opening size (mm)	0.35
Tensile strength at 5 % strain (kN/m)	35
Tensile strength at 10 % strain (kN/m)	75
CBR Puncture Strength (kN)	10
CBR Push through displacement (mm)	25

Methodology and test programme. Various CBR tests have been conducted in the laboratory according to the test programme presented in Table 10 to find the effects on bearing ratio of compacted fly ash – soil matrix (of thickness ratio 1:1) with geotextile at interface. The thickness ratio has been so chosen considering the limited dimensions of CBR mould. The tests have been conducted on fly ash – soil – geotextile matrix by maintaining moisture content of fly ash at its optimum (obtained from relevant Proctor compaction tests) and by increasing moisture content of clay from OMC towards its liquid limit. To observe repeatability, each test has been repeated three times until the results obtained varied within ± 0.5 %.

Table 10. CBR test programme of fly ash – soil composite matrix with geotextile at interface using standard Proctor and modified Proctor energy

Sl. No.	Ht. of Flyash (h_f) (mm)	Ht. of Soil (h_s) (mm)	Thickness Ratio, Flyash: Soil (h_f/h_s)	*Moulding Water Content of Soil (%)	
				Standard Proctor energy	Modified Proctor energy
1	63.5	63.5	1:1	16	12
2	63.5	63.5	1:1	22	18
3	63.5	63.5	1:1	28	24
4	63.5	63.5	1:1	34	30
5	63.5	63.5	1:1	40	36

*Moulding water content (MWC) of fly ash has been kept same (41 %).

Results and discussion. The CBR values of soil and fly ash, when tested separately, at different moulding water contents for standard and modified Proctor compaction energy are presented in Table 11 and Table 12 respectively.

Table 11. Soaked CBR values of soil and fly ash for standard Proctor compaction energy

Type	Depth (mm)	Moulding water content (MWC)				
		(16 %)	(22 %)	(28 %)	(34 %)	(41 %)
Clay	127	3.24	1.32	0.39	0.24	-
Fly ash	127	-	-	-	-	20.10

Table 12. Soaked CBR values of soil and fly ash for modified Proctor compaction energy

Type	Depth (mm)	Moulding water content (MWC)				
		(12 %)	(18 %)	(24 %)	(28 %)	(30 %)
Clay	127	3.87	2.16	0.51	-	0.22
Fly ash	127	-	-	-	25.93	-

The CBR values of fly ash – soil composite matrix with geotextile at interface at different moulding water contents for standard and modified Proctor compaction energy have been presented in Table 13 and Table 14 respectively.

Table 13. Soaked CBR values of fly ash – soil composite matrix with geotextile at interface (for standard Proctor compaction energy)

Type	Depth of Fly ash (mm)	Depth of Soil (mm)	Thickness ratio (Fly ash: Soil)	Moulding water content (MWC)			
				(16 %)	(22 %)	(28 %)	(34 %)
Fly ash – soil composite matrix with Geotextile at interface	63.5	63.5	1:1	14.09	14.21	6.20	6.20

Table 14. Soaked CBR values of fly ash – soil composite matrix with geotextile at interface (for modified Proctor compaction energy)

Type	Depth of Fly ash (mm)	Depth of Soil (mm)	Thickness ratio (Fly ash: Soil)	Moulding water content (MWC)				
				(12 %)	(18 %)	(24 %)	(30 %)	(36 %)
Fly ash – soil composite matrix with Geotextile at interface	63.5	63.5	1:1	25.90	12.56	6.13	4.61	2.84

Based on the experimental results, the effects on bearing ratio of reinforced composite fly ash – soil system due to (i) the moulding moisture content of soil and (ii) compaction energy have been discussed. Further, a term namely "improvement factor" in terms of increase of CBR with respect to that of original soil has been introduced to study the improvement in CBR characteristics of soil – geotextile – fly ash matrix.

a) Effect of moulding water content of soil. The bearing ratio of reinforced composite fly ash – soil system decreases as the moulding water content increases. This is revealed from Table 13 and Table 14, which shows the variation of CBR with moulding water content for a thickness ratio of (1:1). It has been observed that the bearing ratio is maximum when moulding water content is in the neighbourhood of OMC. The CBR value decreases from 14.09 % to 6.20 %, for increase in moulding water content from 16 % to 34 %, for thickness ratio of 1:1 and standard compaction energy. In case of modified compaction energy, the CBR value decreases from 25.90 % to 2.84 % for the same thickness ratio when the moulding water content increased from 12 % to 36 %. Therefore, it can be implied that at higher moulding water content also, the soil – geotextile – fly ash composite may prove to be economical in pavement construction.

b) Effect of compaction energy . It is observed from Table 13 and Table 14 that CBR value of soil – geotextile – fly ash matrix increases with increase in compaction energy. The CBR value at OMC increases from 14.09 % to 25.90 % when compaction energy changes from standard to modified. This can be attributed to increase of dry density and soil strength occurring due to application of more compaction energy. Hence, it appears that higher subgrade strength is likely to be achieved with modified compaction energy for the composite matrix.

c) Improvement factor. An improvement factor (I_F), defined by equation 8, has been introduced to study the improvement of CBR of soil – geotextile – fly ash matrix (CBR_{sgm}) with respect to that of original soil (CBR_s).

$$I_F = CBR_{sgm} / CBR_s \tag{8}$$

The variation of improvement factor with different moulding water contents for thickness ratio (1:1) has been presented in Figure 9. It can be observed that there is considerable improvement in bearing ratio values for all the cases. The improvement factor is found to be maximum for moulding water content of 34 %, which indicates the effectiveness of soil – geotextile – fly ash matrix under worst case scenario of high water content in field.

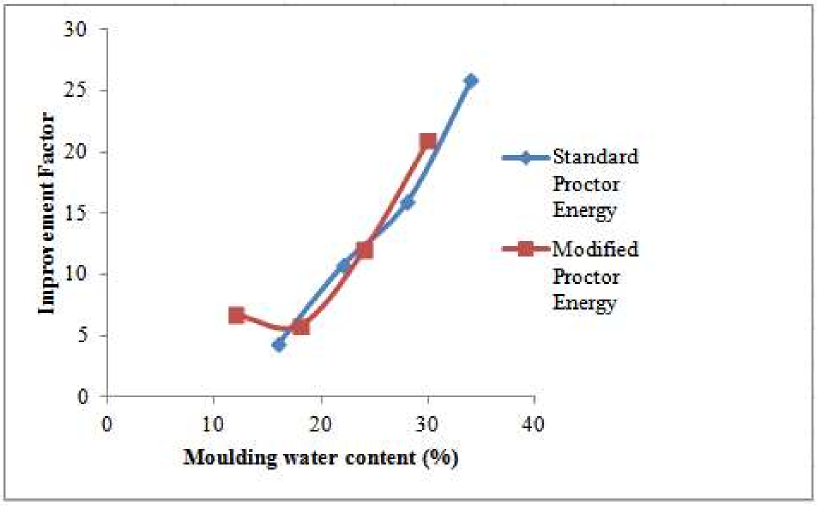

Fig. 9. Variation of improvement factor with thickness ratio (1:1) for different moulding water contents

4.3 Comparison of CBR characteristics of soil subgrade from the Two Studies

In case of first soil improvement study presented in the section 4.1, it has been observed that under unsoaked condition, a maximum improvement in CBR value by a factor of 6.65 occurs when soil is mixed with 6 % lime and 9 % RHA contents at OMC. Whereas under soaked condition, an improvement factor of 8.52 is found to occur when soil is mixed with 6 % lime and 6 % RHA contents at OMC. From the second soil improvement study presented in the section 4.2, the maximum value of improvement factor under soaked condition is found to be 25.83 (Figure 9) which occurs for highest moulding water content of 34 %. But at a moulding water content of 16 % (near to OMC), the improvement factor is observed to be 4.77.

Therefore, it can be said that the method of improvement of soil subgrade overlaid by compacted fly ash and geotextile at interface proves to be a better means of enhancing the soil character at high water contents. But the first method of soil improvement by addition of lime and RHA, somewhat yields a better result at OMC than the soil – geotextile – fly ash matrix method.

4.4 Design of a Flexible Pavement using IRC 37:2018 [25] and IITPAVE Software with Input of CBR value of soil subgrade from the Two Studies

In this section, an attempt has been made to design a bituminous pavement with granular base and sub-base layers using the guidelines laid by IRC 37:2018 and IITPAVE software with the following input data (assumed for a typical case):

(a) Four lane divided carriageway
(b) Initial traffic in the year of completion of construction = 5000 cvpd (two-way)
(c) Traffic growth rate per annum = 6.0 %
(d) Design life period = 20 years
(e) Vehicle damage factor = 5.2 (taken to be the same for both directions)

As per IRC 37:2018, for Lateral Distribution factor = 0.75 (for each direction), Initial directional traffic = 2500 cvpd (assuming 50 per cent in each direction), and Vehicle Damage Factor (VDF) = 5.2, the design cumulative number of standard axles is calculated to be

$$N = \frac{2500 \times 365 \times \left((1 + 0.06)^{20} - 1 \right)}{0.06} \times 0.75 \times 5.2 = 131 msa \quad (9)$$

Since the design traffic is more than 50 msa, a SMA/GGRB or BC with modified bitumen surface course and DBM binder/base layer with VG40 has been considered as per IRC 37:2018 guidelines. A trial section with 190 mm total bituminous layer (40 mm thick surface layer, 70 mm thick DBM-II, 80 mm thick bottom rich DBM-I) and 480 mm total granular layer (250 mm thick WMM and 230 mm thick GSB) has been considered.

Considering 90 % reliability performance models for subgrade rutting and bituminous layer cracking (as design traffic > 20 msa), the allowable vertical compressive strain on subgrade is calculated to be (using equation 3.2, IRC 37:2018) = 0.000301. Also, the allowable horizontal tensile strain at the bottom of bituminous layer assuming a air void content of 3 % and effective binder volume of 11.5 %, and a resilient modulus of 3000 MPa for bottom DBM layer (DBM-I) is calculated to be (using Equation 3.4, IRC 37:2018) = 0.000150.

Now, Table 15 presents the details of four cases that have been tested for their adequacy against the above trial pavement section. Case 1 and case 2 correspond respectively to virgin soil and treated soil (with maximum improvement factor) by soil – lime – RHA matrix method as described in section 4.1. Similarly, case 3 and case 4 correspond respectively to virgin soil and treated soil (with maximum improvement factor) by soil – geotextile – fly ash matrix method as described in section 4.2.

Table 15. Cases analysed in IITPAVE

Sl. No.	Type of soil modification	CBR (soaked) (%)	Resilient modulus of subgrade (MPa) *(Section 6.3, IRC 37:2018)*	Resilient modulus of granular layer (MPa) *(Equation 7.1, IRC 37:2018)*
1.	Soil 1	3.5*	35	113
2.	Soil 1+ 6 % Lime + 6 % RHA	29.82*	100	322
3.	Soil 2	0.24#	2.4	8
4.	Soil 2 overlaid by compacted fly ash (thickness ratio of 1:1) and geotextile at interface	6.2#	57	183

*Refer section 4.1
#Refer section 4.2

The trial pavement section has then been analysed using IITPAVE software for the four cases described in Table 15 with the following inputs (Poisson's ratio values of 0.35 for all the three layers, layer thicknesses of 190 mm and 480 mm, third layer being semi-infinite, a single wheel load of 20000 N, and tyre pressure of 0.56 MPa) to obtain the values of vertical compressive strain on top of subgrade, and horizontal tensile strain at the bottom of bituminous layer. The obtained values of critical strains are then compared with the theoretically calculated values (i.e. 0.000301and 0.000150 respectively) to assess the adequacy of the trial pavement sections. The values of input parameters are chosen in accordance with IRC 37:2018 guidelines. The typical input and output windows of IITPAVE analysis for Case 2 have been shown in Figure 10 and Figure 11 respectively.

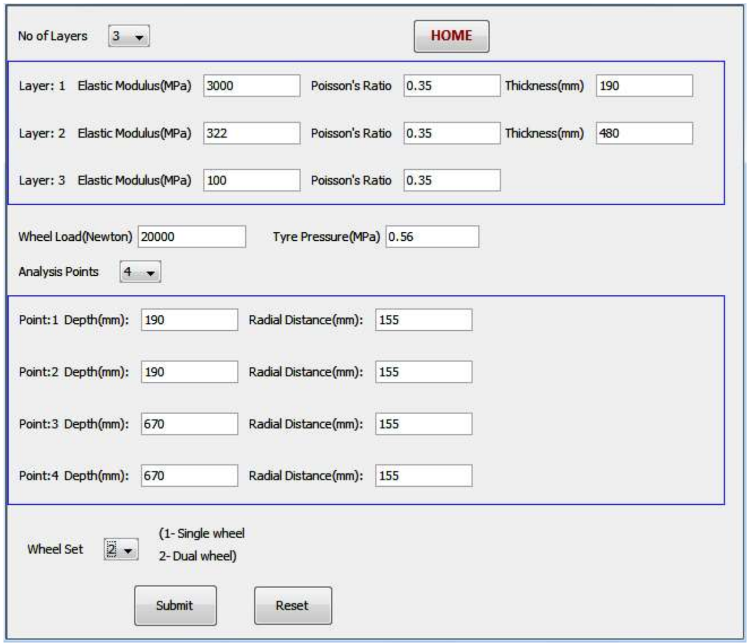

Fig. 10. Typical input window of IITPAVE analysis for Case 2

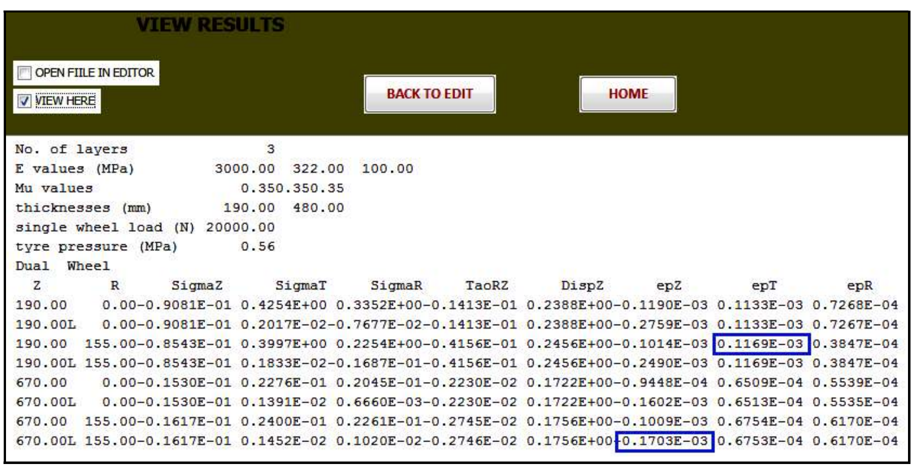

Fig. 11. Typical output window of IITPAVE analysis for Case 2

The results of IITPAVE analyses have been summarized in Table 16. It can be observed from the results that both type of virgin soil (Case 1, 3) fails for the trial section as the obtained values of critical strains are greater than the permissible values. For case 2, where soil is mixed with 6 % lime and 6 % RHA contents, the obtained values of critical strains are well within the limit of permissible values, hence, the trial section proves to be adequate for the input design parameters. Subsequently, it can be inferred that addition of lime and RHA yields quite a good degree of subgrade improvement at OMC. For case 4 which is the worst case scenario for soil – geotextile – fly ash matrix method (i.e. CBR is measured at highest moulding water content), the trial section marginally fails for the input parameters. Therefore, it can be said that for soil – geotextile – fly ash matrix method of subgrade improvement, the trial section might have been adequate for CBR values measured at lower moulding water content. The above observations also reiterates with that made in section 4.3.

Table 16. Results of IITPAVE analyses

Sl. No.	Type of soil modification	Vertical compressive strain on subgrade		Horizontal tensile strain at the bottom of bituminous layer	
		Allowable value *(Equation 3.2, IRC 37:2018)*	Obtained value *(IITPAVE Analysis)*	Allowable value *(Equation 3.4, IRC 37:2018)*	Obtained value *(IITPAVE Analysis)*
1.	Soil 1	0.000301	0.0003666	0.000150	0.0001824
2.	Soil 1+ 6 % Lime + 6 % RHA	0.000301	0.0001703	0.000150	0.0001169
3.	Soil 2	0.000301	0.0009065	0.000150	0.0003593
4.	Soil 2 overlaid by compacted fly ash (thickness ratio of 1:1) and geotextile at interface	0.000301	0.000260	0.000150	0.0001513

5 Conclusions

The following conclusions may be drawn from the present study.

1. Soil characterization and in-situ test interpretation have been evolving from basic empirical recommendations to a more advanced area which requires a thorough knowledge of material behaviour.
2. In general, cross-correlation of measurements from different tests is recommended to understand the complex in-situ soil behaviour.
3. Upon comparing the CBR characteristics of soil – lime – RHA matrix method and soil – geotextile – fly ash matrix method of subgrade improvement, it has

been observed that under unsoaked condition, a maximum improvement in CBR value by a factor of 6.65 occurs when soil is mixed with 6 % lime and 9 % RHA contents at OMC. Whereas under soaked condition, an improvement factor of 8.52 is found to occur when soil is mixed with 6 % lime and 6 % RHA contents at OMC. In case of soil – geotextile – fly ash matrix method of subgrade improvement, the maximum value of improvement factor under soaked condition is found to be 25.83 which occurs for highest moulding water content of 34 % irrespective of compaction energy. But at a moulding water content of 16 % (near to OMC), the improvement factor is observed to be 4.77.

4. The method of improvement of soil subgrade overlaid by compacted fly ash and geotextile at interface proves to be a better means of enhancing the soil character at high water contents. But the method of soil improvement by addition of lime and RHA, somewhat yields a better result at OMC than the former.
5. From IITPAVE analyses, it can be observed that addition of 6 % lime and 6 % RHA contents to soil at OMC under soaked conditions yields an effective level of subgrade improvement, as obtained critical strains are well within the permissible limit for the trial pavement section.

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References

1. Schofield, A., and Wroth, P.: Critical State Soil Mechanics. McGraw-Hill, London, pp. 310 (1968).
2. Roscoe, K.H., and Burland, J.B.: On the generalized stress-strain behaviour of wet clay. Proc. Symp. on Plasticity, Cambridge, pp. 535– 610 (1968).
3. Shibuya, S., Mitashi, T., Yamashita, S., and Tanaka, H.: Recent Japanese practice for investigating elastic stiffness of ground. Advances in Site Investigation Practice, Thomas Telford, pp. 875–886 (1996).
4. Stokoe, K.H. II, Hwang, S.K., Lee, J.N.K., and Andrus, R.D.: Effects of various parameters on the stiffness and damping of soils at small to medium strains. Proc. 1st Int. Conf. on Prefailure Deformation Characterization of Geomaterials, Sapporo, Vol. 2, pp. 785–816 (1995).
5. Tatsuoka, F., Jardine, R.J., Lo Presti, D., Di Benedetto, H., and Kodaka, T.: Theme Lecture: Characterizing the prefailure deformation properties of geomaterials. 14th Int. Conf. Soil Mech. Found. Engg, Hamburg, Vol. 4, pp. 2129–2164 (1997).
6. Lo Presti, D.C.F., Pallara, O., Jamiolkowski, M., and Cavallaro, A.: Anisotropy of small strain stiffness of undisturbed and reconstituted clays. Proc. 2nd Int. Symp. On Pre-Failure Deformation Characterization of Geomaterials, IS-Torino 99, Torino, Vol. 1, pp. 11-18 (1999).

7. Almeida, M.S.S., and Marques, M.E.S.: The behaviour of Sarapui soft clay. Characterization and Engineering Properties of Natural Soils, Balkema Publishers, Vol. 1, pp. 477–504 (2003).
8. Hight, D.W., and Leroueil, S.: Characterization of soils for engineering purposes. Characterization and Engineering Properties of Natural Soils, Balkema Publishers, Vol. 1, pp. 255–362 (2003).
9. Hight, D.W., Paul, M. A., Barras, B. F., Powell, J. M., Nash, D. F. T., Smith, P. R., Jardine, R.J., and Edwards, D. H.: The characterization of the Bothkennar Clay. Characterization and Engineering Properties of Natural Soils, Balkema Publishers, Vol. 1, pp. 543–598 (2003).
10. Diaz-Rodrigues, J.A.: Characterization and Engineering Properties of Mexico City Lacustrine Soils. Characterization and Engineering Properties of Natural Soils, Balkema Publishers, Vol. 1, pp. 725-756 (2003).
11. Lo Presti, D.C.F, Jamiolkowski, M., and Pepe, M.: Geotechnical characterization of the subsoil of Pisa Tower. Characterization and Engineering Properties of Natural Soils, Balkema Publishers, Vol. 2, pp. 909–946 (2003).
12. Coop, M.R., and Airey, D.W.: Carbonate sands. Characterization and Engineering Properties of Natural Soils, Balkema Publishers, Vol. 2, pp. 1049–1086 (2003).
13. Jamiolkowski, M., and Lo Presti, D.C.F.: Geotechnical characterization of Holocene and Pleistocene Messina sand and gravel deposits. Characterization and Engineering Properties of Natural Soils, Balkema Publishers, Vol. 2, pp. 1087–1120 (2003).
14. Schnaid, F., Lehane, B.M., and Fahey, M.: In situ test characterization of unusual geomaterials. 2nd Int. Conf. on Site Characterization, Milpress, Porto, Vol.1, pp. 49–74 (2004).
15. Roscoe, K.H., Schofield, A.N., and Wroth, C.P.: On the yielding of soils. Géotechnique (8), pp. 22-53 (1958).
16. Wood, D.M.: On stress parameters. Géotechnique (34), pp. 282-287(1984).
17. Carter, J.P.: Predictions of the non-homogeneous behaviour of clay in the triaxial test. Géotechnique (32), pp. 55-58 (1982).
18. Roscoe, K.H.: An apparatus for the application of simple shear to soil samples. Proc. 3rd Int. Conf. Soil Mechanics and Foundation Engineering, pp. 186-191, Zurich. (1953).
19. Wood, D.M., Drescher, A., and Budhu, M.: On the determination of the stress state in the simple shear apparatus. Geotechnical Testing J. ASTM (2), Vol. 4, pp. 211-222 (1979).
20. Lade, P.V.: Localization effects in triaxial tests on sand. Proc. IUTAM Symp. Deformation and Failure of Granular Materials, Delft, pp. 461-472 (1982).
21. Drescher, A., and Vardoulakis, I.: Geometric softening in triaxial tests on granular material. Géotechnique (32), pp. 291-304 (1982).
22. Vermeer, P.A.: A simple shear-band -analysis using compliances. Proc. IUTAM Symp. Deformation and Failure of Granular Materials, Delft, pp. 493-502 (1982).
23. Chakraborty, S.: Some studies on improvement of soft soil by stabilization with different materials for use in sub-grade of flexible pavement, PhD Thesis, Jadavpur University, Kolkata (2014).
24. Sengupta, A., Mukherjee, S., and Ghosh, A.: Improvement of Bearing Ratio of Clayey Subgrade using Compacted Flyash Layer. Geotechnical and Geological Engineering, 35(4), pp. 1885–1894 (2017).
25. IRC: 37-2018: Guidelines for the design of flexible pavements, (Fourth revision).