



# Soil Behavior and Characterization: Effect of Improvement in CBR Characteristics of Soil Subgrade on Design of Bituminous Pavements

Sibapriya Mukherjee<sup>1</sup> · Poulami Ghosh<sup>1</sup>

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**Abstract** The California bearing ratio (CBR) value greatly influences the strength characteristics of soil subgrade, hence, is an important parameter to understand the soil behavior pertinent to pavement design. The CBR value, along with the empirical curves, is used to determine the thickness of a pavement and its component layers, thus aiding in the design of flexible pavements. If the naturally occurring local soil subgrade has low strength in terms of CBR and poor engineering properties, improved soil subgrade has to be provided by way of lime/cement treatment or by mixing locally available cheap alternative materials or by mechanical stabilization and other similar techniques. In the present study, an attempt has been made to review the soil behavior in terms of CBR characteristics for two methods of soil properties enhancement, viz. addition of lime and rice husk ash to soil and the method of improvement of soil subgrade overlain by compacted fly ash with geotextile at soil–fly ash interface. Using the data obtained from the review study, design of a bituminous pavement with granular base and sub-base layers and an alternative pavement with RAP layer with CTSB sub-base have also been attempted using the IRC 37:2018 guidelines and IITPAVE software to find out the adequacy of the two subgrade improvement methods with respect to reduction in pavement thicknesses. Further, some design charts have also been developed for the stabilized soil subgrades for

traffic intensities of 5, 15, 30, 50, 80 and 100 msa for both pavement types.

**Keywords** Soil behavior · Subgrade improvement · CBR · Lime · RHA · Fly ash · IITPAVE

## Introduction

Soil behavior 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, flexible pavement design, improvement of weak soil, understanding of soil behavior (under seismic and non-seismic conditions) with respect to soil characterization and strength has become very important to a geotechnical engineer. 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 (OMC) along with maximum dry density (MDD) for compaction characteristics, etc., influence soil behavior pertinent to geotechnical applications. For example, the California bearing ratio (CBR) test is widely used to evaluate the strength characteristics of soil subgrade (both unmodified and modified). The results so obtained along with the empirical curves are then used to determine the thickness of a pavement and its component layers, thus, aiding in the design of flexible pavements. If the naturally occurring local soil subgrade has low strength in terms of CBR and poor engineering properties, improved

✉ Sibapriya Mukherjee  
sibapriya.mukherjee@gmail.com

Poulami Ghosh  
poulamig04@gmail.com

<sup>1</sup> Department of Civil Engineering, Jadavpur University, Kolkata 700032, India

soil subgrade has to be provided by way of lime/cement treatment or by mixing locally available cheap alternative materials or by mechanical stabilization and other similar techniques. 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) [1]. Another method has been envisaged with placing of a compacted fly ash layer over soil subgrade with geotextile at the soil–fly ash interface [2]. Generally, pavements are so designed as to meet with its satisfactory functional and structural performance during the intended service life period. Roughness caused by variation in surface profile, cracking of layers bound by bituminous or cementitious materials, rutting of subgrade, granular layers and bituminous layers are the primary indicators of the functional and structural performance of pavements. However, it is not easy to perform an accurate structural analysis of a flexible pavement structure because the materials forming its component layers exhibit non-uniform properties. Therefore, a mechanistic-empirical approach is followed for the design of bituminous pavements, nowadays. In this approach, stresses or strains in the pavement layers are calculated at critical locations using a linear elastic layered analytical model (e.g., IITPAVE software). These calculated stresses or strains are then compared with the permissible or limiting values. Many researchers have previously carried out the design and analysis of flexible pavements using various approaches. Vorobieff and Murphy [3] have discussed various design procedures for lime stabilized subgrades. Siddique and Rajbongshi [4] have modeled cement and lime stabilized rural roads using CIRCLY program and found that at a certain modulus and CBR, thickness of soil–cement base and soil–lime base increases with allowable number of load repetitions. Ziari and Khabiri [5] have analyzed a previously derived interface constitutive model using KENLAYER to compute the stresses and strains in typical flexible road structures. Ekwulo and Eme [6] have attempted to design a flexible pavement using the layered elastic analysis software EVERSTRESS. Bagui et al. [7] have proposed a C++ computer program to design a flexible pavement considering design traffic and CBR value. However, only a few studies refer to mechanistic principles and have attempted to develop design charts for stabilized subgrades [8].

In the present study, an attempt has been made to review the soil behavior in terms of CBR characteristics for two methods of soil properties 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 with geotextile at soil–fly ash interface. Using the data obtained from the review study, design of a bituminous pavement with granular base and sub-base layers and an alternative pavement have also been attempted using the guidelines laid by IRC 37:2018 [9] and IITPAVE software to find out the adequacy of the two subgrade improvement methods with respect to pavement design (i.e., reduction in pavement thickness). Further, some design charts have also been developed for the stabilized soil subgrades.

## Soil Improvement Studies Carried out at Jadavpur University

Research work carried out at Jadavpur University to study the soil behavior involving the following two methods of sub–soil modification has been reviewed in this section.

### Improvement of Soil Subgrade by Addition of RHA and Lime

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 [1]. 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 study the influence of different RHA and lime contents on soil properties like Atterberg's limits (LL, PL), OMC and MDD.

### 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:

#### *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 1.

**Table 1** Engineering properties of soil

Specific gravity	Grain size			Atterberg’s limits		Standard Proctor compaction		CBR at OMC	
	Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	MDD (kN/m <sup>3</sup> )	OMC (%)	CBR <sub>Unsoaked</sub> (%)	CBR <sub>Soaked</sub> (%)
2.65	5	68	27	51	28	16.30	15.92	4.25	3.50

*Lime*

Lime is a very good stabilizing material, which 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.

*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 2.

**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 3. The test program is presented in Table 4.

**Table 2** Physical properties of RHA

Specific gravity	Atterberg’s limits		Standard Proctor compaction		Angle of internal friction φ (°)	CBR at OMC	
	LL (%)	PL (%)	MDD (kN/m <sup>3</sup> )	OMC (%)		CBR <sub>Unsoaked</sub> (%)	CBR <sub>Soaked</sub> (%)
1.96	NP	NP	8.5	32	38	8.75	8.15

**Results and Discussion**

**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. The variation of liquid limit, plastic limit and plasticity index with varying percentages of lime and RHA contents is shown in Figs. 1, 2 and 3, respectively.

**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 Fig. 4. The maximum dry density (MDD) generally decreases with increasing lime content. From Fig. 4, 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

**Table 3** 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 4** Test program

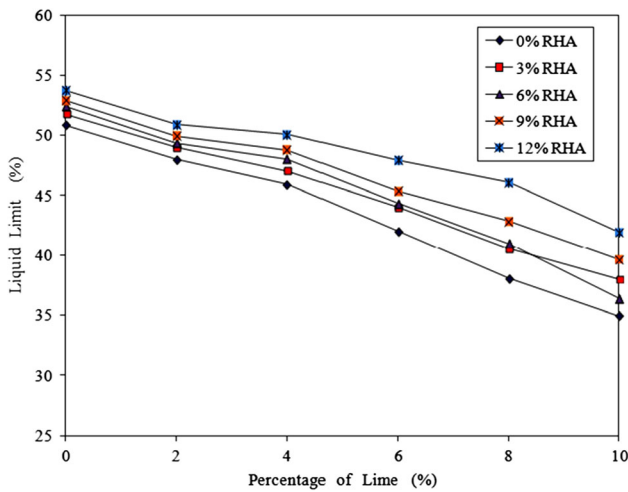
Sl. No	Test	No. of Tests
1	Atterberg's limits (Liquid Limit, LL and Plastic Limit, PL)	30 + 30
2	Standard Proctor compaction test ( OMC, MDD)	30
3	CBR test at OMC (Unsoaked and soaked)	30 + 30

decreases as resistance against compactive effort increases due to flocculated soil structure.

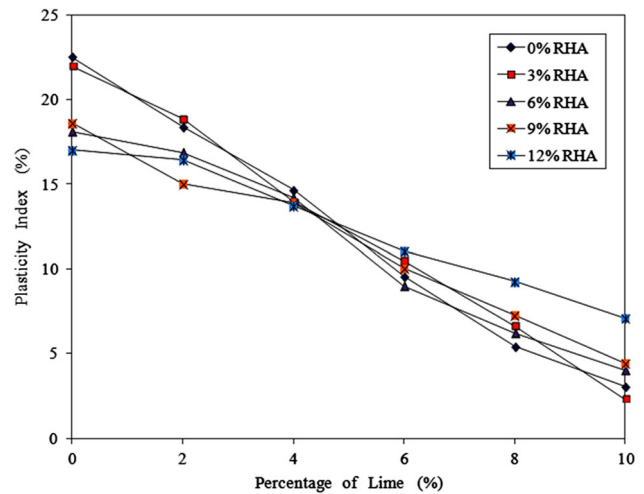
#### *Optimum Moisture Content (OMC)*

The variation of the optimum moisture content (OMC) with varying lime and RHA contents is presented in Fig. 5. Generally, the optimum moisture content (OMC) is

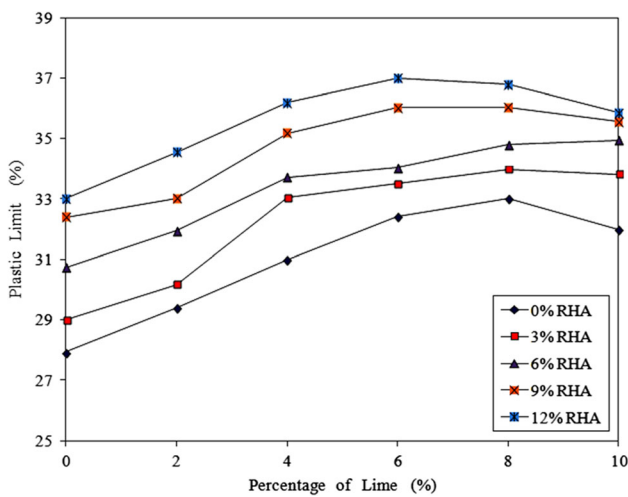
observed to increase with increase in lime content up to 8%, and then, it decreases. OMC increases due to further addition of fine contents even with reduced surface area as free lime needs more water for pozzolanic reaction. Moreover, addition of RHA causes increased coarse fraction with greater surface area. This leads to increase in OMC due to addition of RHA.



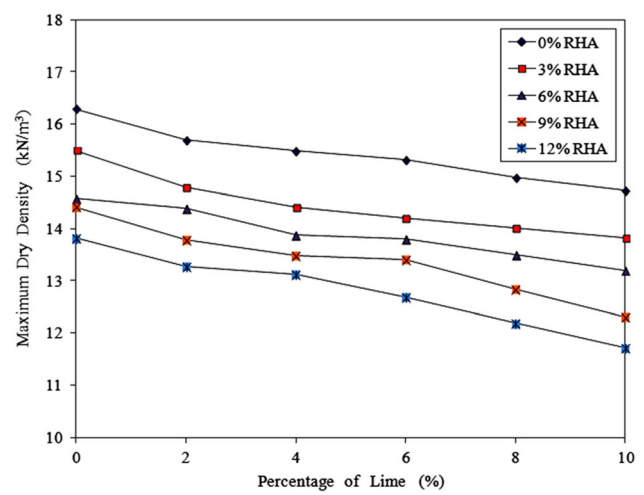
**Fig. 1** Variation of liquid limit with lime content for different RHA contents



**Fig. 3** Variation of plasticity index with lime content for different RHA contents



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

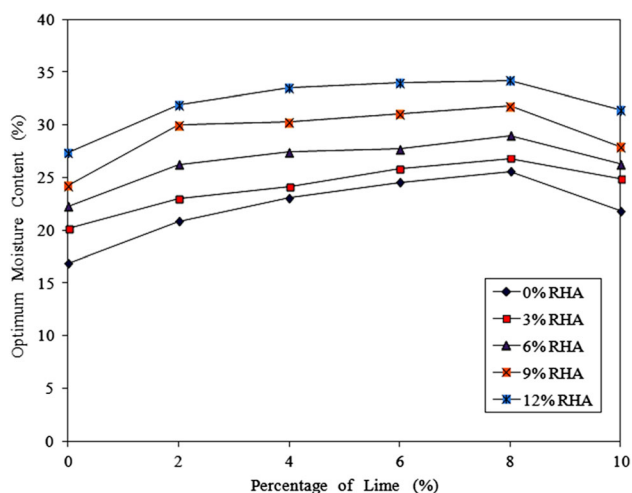


**Fig. 4** Variation of maximum dry density with lime content for different RHA contents

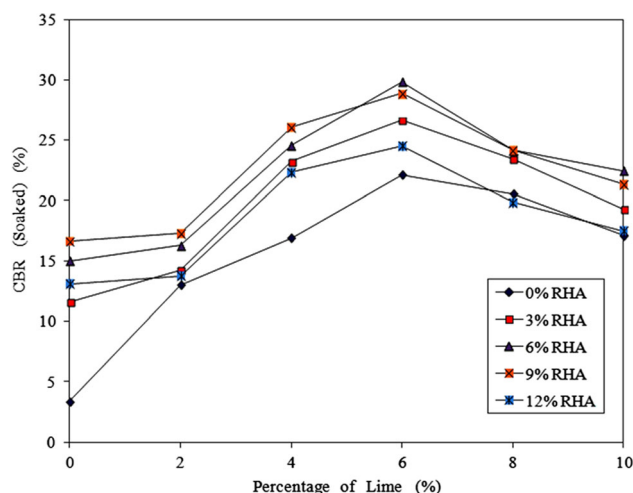
**California Bearing Ratio (CBR) at OMC**

The plot of unsoaked and soaked CBR for different lime and RHA contents is shown in Figs. 6 and 7, respectively. Both the figures show that CBR value increases with increase in 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 in 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

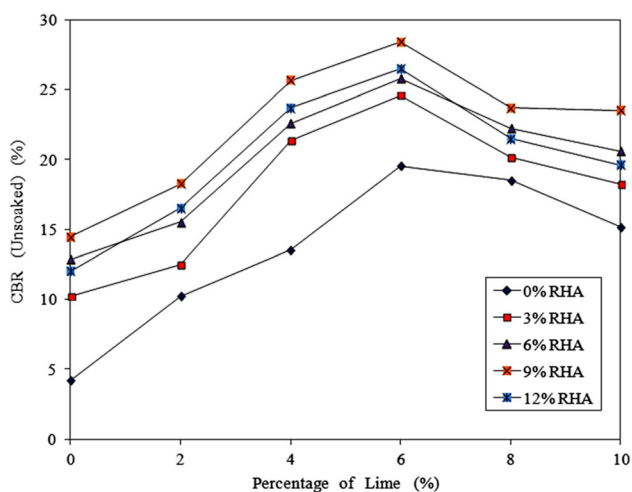
alumina present in natural soil and lime. This reaction produces stable calcium silicate hydrates and calcium aluminates hydrates as the calcium from the lime reacts with the aluminates and silicates of the soil. It is also observed that when RHA is added to the original soil, the strength characteristics also go 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 in 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. 5** Variation of optimum moisture content with lime content for different RHA contents



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



**Fig. 6** Variation of unsoaked CBR with lime and RHA contents at OMC

### Improvement of Soil Subgrade Overlayd by Compacted Fly Ash with Geotextile at Soil–Fly Ash Interface

This study examines the effect of compaction energy and molding water content of clay on CBR of soil–fly ash composite matrix with geotextile at the soil–fly ash interface [2].

#### Materials Used

##### 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 5.

##### Fly Ash

Fly ash 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 are shown in Tables 6 and 7, respectively.

##### Geotextile

100% polypropylene high strength fiber woven geotextile, which is available commercially, has been collected to be used as reinforcement material in these experiments. The properties of geotextile are shown in Table 8.

### Methodology

Various CBR tests have been conducted on compacted fly ash–soil matrix (of thickness ratio 1:2, 1:1 and 2:1) with geotextile at soil–fly ash interface, under soaked conditions as per the test program presented in Table 9. The thickness ratio has been so chosen considering the limited dimensions of CBR mold. For preparing the test molds, the moisture content of fly ash has been maintained at its optimum (obtained from relevant Proctor compaction tests), whereas the moisture content of clay has been increased from OMC toward its liquid limit. To observe repeatability, each test has been repeated three times until the results obtained varied within  $\pm 0.5\%$  in the laboratory.

**Table 5** Geotechnical properties of soil

Specific gravity	Grain size			Atterberg's limits		Standard Proctor compaction		Modified Proctor compaction	
	Sand (%)	Silt (%)	Clay (%)	LL (%)	PL (%)	MDD (kN/m <sup>3</sup> )	OMC (%)	MDD (kN/m <sup>3</sup> )	OMC (%)
2.54	8.34	67.33	24.33	41.2	24.16	16.88	16	18.21	12

**Table 6** Composition of fly ash

Composition	SiO <sub>2</sub>	Al <sub>2</sub> O <sub>3</sub>	Fe <sub>2</sub> O <sub>3</sub>	TiO <sub>2</sub>	CaO <sub>2</sub>	MgO	Mn <sub>3</sub> O <sub>4</sub>	P <sub>2</sub> O <sub>5</sub>	SO <sub>3</sub>	Na <sub>2</sub> O	K <sub>2</sub> O
Percentage	61.8	22.82	8.4	1.6	1.48	0.9	0.156	0.657	0.357	0.245	1.355

**Table 7** Properties of Fly ash

Specific gravity	Grain size					Standard Proctor compaction		Modified Proctor compaction		φ (From direct shear test) (°)	
	Sand (%)	Silt (%)	Clay (%)	C <sub>U</sub>	C <sub>C</sub>	MDD (kN/m <sup>3</sup> )	OMC (%)	MDD (kN/m <sup>3</sup> )	OMC (%)	Standard Proctor compaction	Modified Proctor compaction
2.11	82.17	16.83	1	2.22	4.54	10.24	41	11.4	28	39	41

**Table 8** Properties of geotextile

Thickness (mm)	Mass per unit area (gsm)	Apparent opening size (mm)	Tensile strength at 5% strain (kN/m)	Tensile strength at 10% strain (kN/m)	CBR Puncture Strength (kN)	CBR Push through displacement (mm)
1.5	450	0.35	2.22	4.54	16.88	16

**Table 9** CBR test program of fly ash – soil composite matrix with geotextile at interface using standard Proctor and modified Proctor energy

Sl No	Thickness ratio, Ht. of Flyash: Ht. of Soil (h <sub>f</sub> / h <sub>s</sub> )	*Molding water content of soil (%)	Total no. of tests
1	1:2, 1:1, 2:1	16, 22, 28, 34 (Standard Proctor energy)	27
2		12, 18, 24, 30, 36 (Modified Proctor energy)	

\*Molding water content (MWC) of fly ash has been kept at 41% for standard Proctor compaction energy and 28% for modified Proctor compaction energy

### Results and Discussion

The CBR values of soil and fly ash, when tested separately, at different molding water contents for standard and modified Proctor compaction energy are presented in Tables 10 and 11, respectively.

The CBR values of fly ash–soil composite matrix with geotextile at interface at different molding water contents for standard and modified Proctor compaction energy are presented in Table 12.

Based on the experimental results (Table 12), the effects on CBR characteristics of reinforced composite fly ash–soil

**Table 10** Soaked CBR values of soil and fly ash for standard Proctor compaction energy

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

**Table 11** Soaked CBR values of soil and fly ash for modified Proctor compaction energy

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

**Table 12** Soaked CBR values of fly ash – soil composite matrix with geotextile at interface (for standard Proctor compaction energy)

Type	Thickness ratio (Fly ash: Soil)	Molding water content of clay (MWC)									
		For Standard Proctor Compaction Energy				For Modified Proctor Compaction Energy					
		(16%)	(22%)	(28%)	(34%)	(12%)	(18%)	(24%)	(30%)	(36%)	
Fly ash – soil composite matrix with geotextile at interface	1:2	12.00	5.10	4.00	2.65	23.48	5.90	5.10	3.00	1.46	
	1:1	14.09	13.40	5.70	6.20	27.20	12.56	6.13	6.00	2.84	
	2:1	18.50	15.23	9.43	8.12	35.00	23.58	13.11	9.60	8.25	

system due to (1) thickness ratio, (2) the molding moisture content of soil and (3) compaction energy, have been discussed as follows:

### Effect of Thickness Ratio

Thickness ratio, defined by ratio of thickness of fly ash to that of soil, greatly influences the CBR characteristics of reinforced composite fly ash–soil system. From Table 12, it can be observed that the CBR value increases with increase in thickness ratio and the maximum CBR value is obtained at a thickness ratio of 2:1 for both the cases of applying standard and modified Proctor compaction energy. This might be occurring due to the fact that fly ash is imparting more stiffness to the composite system.

### Effect of Molding Water Content of Soil

The CBR value of reinforced composite fly ash–soil system decreases as the molding water content increases. This is revealed from Table 12, which shows the variation of CBR with molding water content for various thickness ratios of (1:2, 1:1 and 2:1) and compaction energy. It has been

observed that the CBR value is maximum when molding water content of clay is in the neighborhood of OMC. The CBR value decreases from 18.5 to 8.12%, for increase in molding water content from 16 to 34%, for thickness ratio of 2:1 and standard compaction energy. In case of modified compaction energy, the CBR value decreases from 35 to 8.25% for the same thickness ratio when the molding water content increased from 12 to 36%. Therefore, it can be implied that at higher molding water content also, the soil–geotextile–fly ash composite may prove to be economical in pavement construction.

### Effect of Compaction Energy

It is also observed from Table 12 that CBR value of soil–geotextile–fly ash matrix increases with increase in compaction energy. The CBR value at OMC increases from 18.5 to 35% for thickness ratio of 2:1 when compaction energy changes from standard to modified. This can be attributed to increase in 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.

### Comparison of CBR Characteristics of Modified Soil Subgrade from the Two Studies

An improvement factor ( $I_F$ ), defined by Eq. 1, has been introduced in this section to observe the extent of improvement in CBR characteristics of modified soil subgrade ( $CBR_m$ ) with respect to that of original soil ( $CBR_s$ ) at same water content. The CBR values, determined under soaked conditions by applying standard Proctor compaction energy, have only been considered here, in order to draw a comparison between the two soil improvement methods as described in Sect. 2.

$$I_F = CBR_m / CBR_s \tag{1}$$

The variation of improvement factor with different percentages of lime and rice husk ash (RHA) combinations (first soil improvement method) and with different molding water contents for various thickness ratios (second soil improvement method) is presented in Figs. 8 and 9, respectively.

In case of first soil improvement study presented in Sect. 2.1, it has been observed from Fig. 8 that under soaked condition, the maximum value of improvement factor is found to be 8.98 which occurs when soil is mixed with 6% lime and 6% RHA contents at OMC.

From the second soil improvement study presented in Sect. 2.2, the maximum value of improvement factor under soaked condition is found to be 33.83 (Fig. 9) which occurs for thickness ratio of 2:1 at highest molding water content of 34%. But at a molding water content of 16% (near to

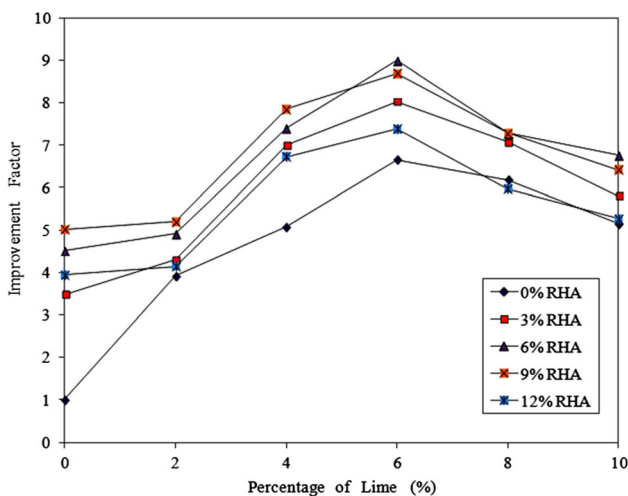


Fig. 8 Variation of improvement factor with lime and RHA contents at OMC

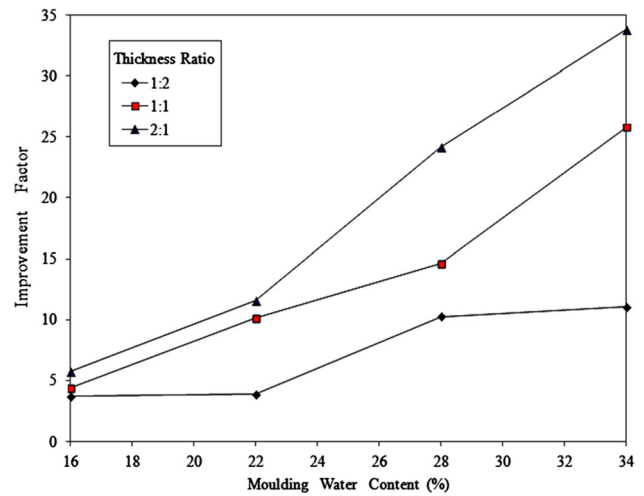


Fig. 9 Variation of improvement factor with molding water contents for different thickness ratios

OMC), the improvement factor is observed to be 5.71 for same thickness ratio.

Therefore, it can be said that the method of improvement of soil subgrade overlaid by compacted fly ash with geotextile at soil–fly ash interface proves to be a better means of enhancing the soil characteristics under worst-case scenario of high water content in field. 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.

### Current Study—Design of a Flexible Pavement Using Guidelines of IRC 37: 2018 [9] and IITPAVE Software

Flexible pavements are best modeled as linear elastic-layered systems. In order to ensure satisfactory performance of flexible pavements in terms of subgrade rutting and bottom-up cracking of bituminous layers, the vertical compressive strain at the top of subgrade and the horizontal tensile strain at the bottom of the bituminous layer are considered to be the critical mechanistic parameters as per IRC 37:2018 guidelines. Therefore, the basic principle for the design procedure is to select a trial pavement section so as to limit the critical strains induced by traffic loads.

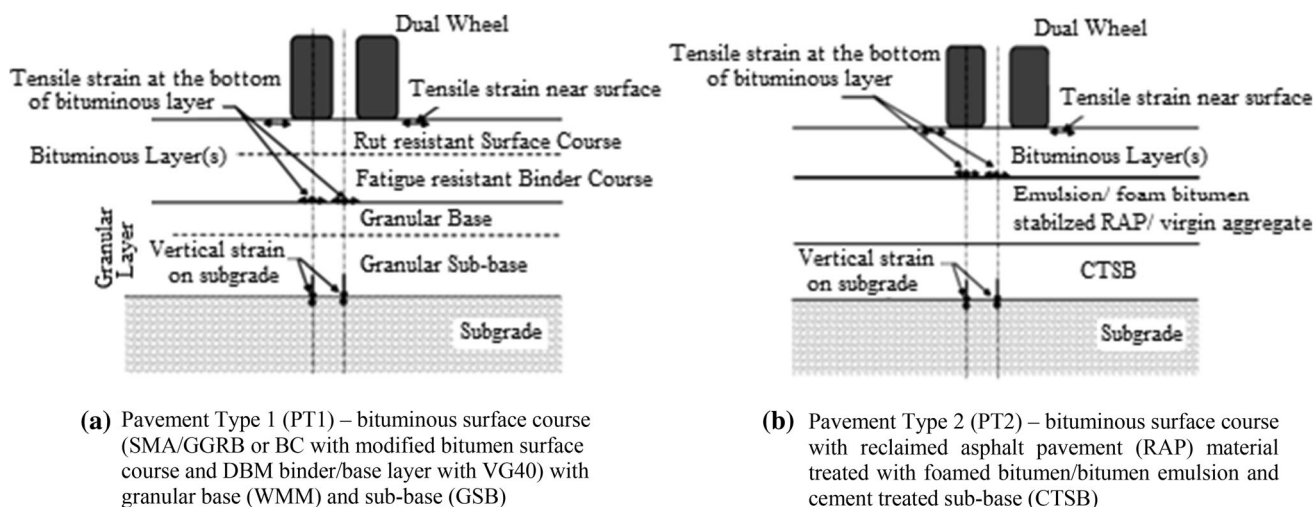
### Methodology

In the current study, IITPAVE analysis has been carried out for two categories of pavements: (a) Pavement Type 1 (PT1)—bituminous surface course (SMA/GGRB or BC with modified bitumen surface course and DBM binder/base layer with VG40) with granular base (WMM) and

sub-base (GSB) and (b) Pavement Type 2 (PT2)—bituminous surface course with reclaimed asphalt pavement (RAP) material treated with foamed bitumen/bitumen emulsion and cement treated sub-base (CTSBS) underlain by various soil subgrades. The two pavement types are chosen so as to compare the pavement thicknesses for a conventional bituminous pavement (PT1) with an alternative and relatively economical pavement type (PT2). The different flexible pavement compositions and the locations at which different critical strains are to be calculated are shown Fig. 10. Additionally, a total of six soil subgrade types have been analyzed in this study: (a) S1—local soil subgrade at OMC with engineering properties as described in Table 1, (b) S2—modified subgrade obtained by mixing S1 with 6% lime and 6% RHA at OMC (since soaked CBR value improved by a factor of 8.98 with this mix proportion), (c) S3—local soil subgrade with engineering properties as described in Table 5 at a molding water content of 16%, (d) S4—modified subgrade obtained by overlaying compacted fly ash over S3 with thickness ratio of 1:1 with geotextile at soil–fly ash interface at a molding water content of 16%, i.e., nearer to OMC of S3 (since soaked CBR value improved by a factor of 4.48), (e) S5—local soil subgrade with engineering properties as described in Table 5 at a molding water content of 34% and (f) S6—modified subgrade obtained by overlaying compacted fly ash over S5 with thickness ratio of 1:1 with geotextile at soil–fly ash interface at highest molding water content of 34% (since soaked CBR value improved by a factor of 20.5). The details of soil subgrade types and the values of relevant input parameters in the IITPAVE software are summarized in Tables 13 and 14 for the two pavement types, PT1 and PT2, respectively.

Then, the trial thicknesses of the constituting pavement layers have been assumed for each of the cases described in

Tables 14 and 15, considering functional and constructability criteria subject to the minimum thicknesses recommended in IRC 37:2018 guidelines. Structural analysis of the above pavement sections has then been carried out using IITPAVE software under a standard dual wheel load of 20 kN on each wheel spaced at 310 mm center to center and tyre pressure of 0.56 MPa to obtain the values of critical strains, which are subsequently compared with the respective allowable values to assess the adequacy of the trial pavement thicknesses. Several iterations have been attempted by changing the layer thicknesses until the strains computed by IITPAVE are found to be less than the allowable strains derived from performance models, in order to obtain the optimum pavement thicknesses for each case. The allowable values of the critical mechanistic parameters have been calculated considering 90% reliability performance models (expressways, national highways, state highways and urban roads) for subgrade rutting and bituminous layer cracking for design traffic of 5, 15, 30, 50, 80 and 100 msa, covering a wide range of traffic volume to account for varying traffic on different types of roads. The allowable vertical compressive strain on subgrade has been calculated using Eq. 3.2, IRC 37:2018. Whereas the allowable horizontal tensile strain at the bottom of bituminous layer has been calculated using Eq. 3.4, IRC 37:2018 assuming an air void content of 3%, effective binder volume of 11.5% and a resilient modulus of 3000 MPa. The values of all input parameters are chosen in accordance with IRC 37:2018 guidelines. The typical input and output windows of IITPAVE analysis for soil subgrade S2, pavement type PT1 under design traffic of 5 msa are shown in Figs. 11 and 12, respectively.



**Fig. 10** Pavement sections showing the locations of critical strains

**Table 13** Cases analyzed in IITPAVE for pavement type PT1

Soil subgrade type	CBR (soaked) (%)	Resilient modulus of subgrade (MPa) (Sect. 6.3, IRC 37:2018)	Poisson's ratio of subgrade (Table 11.1, IRC 37:2018)	Resilient modulus of granular layer (MPa) (Eq. 7.1, IRC 37:2018)	Poisson's ratio of granular layer (Table 11.1, IRC 37:2018)	Resilient modulus of bituminous layer (MPa) (Table 11.1, IRC 37:2018)	Poisson's ratio of bituminous layer (Table 11.1, IRC 37:2018)
S1	3.5	35	0.35	Varies with trial thickness of granular layer and resilient modulus of supporting subgrade	0.35	3000	0.35
S2	29.82	100	0.35		0.35	3000	0.35
S3	3.24	32.4	0.35		0.35	3000	0.35
S4	14.09	95.68	0.35		0.35	3000	0.35
S5	0.24	2.4	0.35		0.35	3000	0.35
S6	6.2	56.58	0.35		0.35	3000	0.35

**Table 14** Cases analyzed in IITPAVE for pavement type PT2

Soil subgrade type	CBR (soaked) (%)	Resilient modulus of subgrade (MPa) (Sect. 6.3, IRC 37:2018)	Poisson's ratio of subgrade (Table 11.1, IRC 37:2018)	Resilient modulus of CTSB layer (MPa) (Table 11.1, IRC 37:2018)	Poisson's ratio of CTSB layer (Table 11.1, IRC 37:2018)	Resilient modulus of stabilized RAP layer (MPa) (Sect. 8.4, IRC 37:2018)	Poisson's ratio of stabilized RAP layer (Sect. 8.4, IRC 37:2018)	Resilient modulus of bituminous layer (MPa) (Table 11.1, IRC 37:2018)	Poisson's ratio of bituminous layer (Table 11.1, IRC 37:2018)
S1	3.5	35	0.35	600	0.25	800	0.35	3000	0.35
S2	29.82	100	0.35	600	0.25	800	0.35	3000	0.35
S3	3.24	32.4	0.35	600	0.25	800	0.35	3000	0.35
S4	14.09	95.68	0.35	600	0.25	800	0.35	3000	0.35
S5	0.24	2.4	0.35	600	0.25	800	0.35	3000	0.35
S6	6.2	56.58	0.35	600	0.25	800	0.35	3000	0.35

## Results and Discussion

The allowable values of vertical compressive strain at the top of soil subgrade and the horizontal tensile strain at the bottom of the bituminous layer derived from performance models along with the corresponding values of critical strains and optimum pavement thickness obtained from IITPAVE analyses for all the case are summarized in Table 15. It can be observed from the results that maximum reduction in pavement thickness has been achieved for traffic intensity of 80 msa and 15 msa in case of PT1 and PT2, respectively. On the other hand, minimum thickness reduction has been observed for 5 msa in case of both PT1 and PT2.

Based on the results of IITPAVE analyses, the effects of improvement in CBR characteristics of soil subgrade on pavement design in terms of pavement thickness have been discussed as follows:

### Effect on Pavement Design due to Improvement of Soil Subgrade By addition of RHA and Lime

The soil subgrade types S1 and S2 represent the local soil subgrade and local soil mixed with 6% lime and 6% RHA, respectively, with water content maintained at 16%, i.e., OMC. Therefore, the results of IITPAVE analyses for these two types of subgrades have been compared to study the effect of the method of soil subgrade improvement by addition of lime and RHA on pavement thicknesses. The variations of thicknesses of constituent pavement layers for both PT1 and PT2, corresponding to the critical strains, with respect to design traffic, for S1 and S2 soil subgrade types are depicted in Fig. 13. It can be observed from the results that in case of local subgrade (S1), an average reduction in pavement thickness of about 26.88% is obtained for alternative pavement section (PT2) when compared to corresponding values for conventional

**Table 15** Results of IITPAVE analyses

Soil subgrade type	Design Traffic (Msa)	Vertical compressive strain on soil subgrade				Horizontal tensile strain at the bottom of bituminous layer				Total pavement thickness (mm)	
		Allowable value (Eq. 3.2, IRC 37:2018)	Obtained value (IITPAVE Analysis)		Allowable value (Eq. 3.4, IRC 37:2018)	Obtained value (IITPAVE Analysis)					
			PT1	PT2		PT1	PT2	PT1	PT2		
S1	5	0.000618	0.000598	0.000533	0.000347	0.000301	0.000437	570	410		
	15	0.000485	0.000478	0.000399	0.000262	0.000234	0.0001040	605	470		
	30	0.000416	0.000411	0.000399	0.000219	0.000198	0.0001040	630	470		
	50	0.000372	0.000360	0.000365	0.000192	0.000182	0.0001026	680	495		
	80	0.000335	0.000307	0.000321	0.000171	0.000166	0.0000974	750	530		
	100	0.000319	0.000292	0.000311	0.000161	0.000156	0.0000968	760	540		
S2	5	0.000618	0.000431	0.000447	0.000347	0.000252	0.0000582	430	340		
	15	0.000485	0.000431	0.000447	0.000262	0.000252	0.0000582	430	340		
	30	0.000416	0.000361	0.000413	0.000219	0.000208	0.0000588	460	360		
	50	0.000372	0.000331	0.000309	0.000192	0.000189	0.0001097	475	420		
	80	0.000335	0.000304	0.000309	0.000171	0.000171	0.0001097	490	420		
	100	0.000319	0.000287	0.000309	0.000161	0.000160	0.0001097	500	420		
S3	5	0.000618	0.000593	0.000548	0.000347	0.000288	0.0000423	580	410		
	15	0.000485	0.000473	0.000411	0.000262	0.000225	0.0001046	615	470		
	30	0.000416	0.000408	0.000411	0.000219	0.000191	0.0001046	640	470		
	50	0.000372	0.000359	0.000351	0.000192	0.000175	0.0000982	690	510		
	80	0.000335	0.000323	0.000330	0.000171	0.000171	0.0000972	750	530		
	100	0.000319	0.000307	0.000313	0.000161	0.000161	0.0000950	760	545		
S4	5	0.000618	0.000448	0.000457	0.000347	0.000259	0.0000574	430	340		
	15	0.000485	0.000448	0.000457	0.000262	0.000259	0.0000574	430	340		
	30	0.000416	0.000375	0.000392	0.000219	0.000213	0.0000589	460	380		
	50	0.000372	0.000333	0.000315	0.000192	0.000187	0.0001097	480	420		
	80	0.000335	0.000306	0.000315	0.000171	0.000169	0.0001097	495	420		
	100	0.000319	0.000289	0.000315	0.000161	0.000159	0.0001097	505	420		
S5	5	0.000618	0.000612	0.000617	0.000347	0.000276	0.0000888	920	560		
	15	0.000485	Trial pavement thickness fails in IITPAVE analysis	0.000456	0.000262	Trial pavement thickness fails in IITPAVE analysis	0.0000808	-	650		
	30	0.000416	Trial pavement thickness fails in IITPAVE analysis	0.000377	0.000219	Trial pavement thickness fails in IITPAVE analysis	0.0000795	-	720		
	50	0.000372	Trial pavement thickness fails in IITPAVE analysis	0.000331	0.000192	Trial pavement thickness fails in IITPAVE analysis	0.0000671	-	760		
	80	0.000335	Trial pavement thickness fails in IITPAVE analysis	0.000331	0.000171	Trial pavement thickness fails in IITPAVE analysis	0.0000671	-	760		
	100	0.000319	Trial pavement thickness fails in IITPAVE analysis	0.000314	0.000161	Trial pavement thickness fails in IITPAVE analysis	0.0000665	-	780		
S6	5	0.000618	0.000612	0.000581	0.000347	0.000301	0.0000473	450	340		
	15	0.000485	0.000482	0.000426	0.000262	0.000232	0.0001109	485	400		
	30	0.000416	0.000410	0.000395	0.000219	0.000197	0.0001086	510	420		
	50	0.000372	0.000363	0.000368	0.000192	0.000174	0.0001069	530	440		
	80	0.000335	0.000326	0.000324	0.000171	0.000169	0.0001004	580	470		
	100	0.000319	0.000309	0.000296	0.000161	0.000159	0.0000928	590	490		

pavement section (PT1). Whereas with subgrade modification by addition of lime and RHA (S2), an average reduction in pavement thickness of about 29.92% and 21.04% has been observed, respectively, for PT1 and PT2

with respect to the corresponding values obtained for local subgrade. Also, a further reduction of 29.68% has been seen if PT2 is preferred over PT1.

**Fig. 11** Typical input window of IITPAVE analysis for soil subgrade S2, pavement type PT1 under 5 msa

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No. of layers	3		
E values (MPa)	3000.00	279.16	100.00
Mu values	0.350.350.35		
thicknesses (mm)	80.00	350.00	
single wheel load (N)	20000.00		
tyre pressure (MPa)	0.56		
Dual Wheel			
Z	R	SigmaZ	SigmaT
80.00	0.00-0.2437E+00	0.9380E+00	0.7679E+00-0.1995E-01
80.00L	0.00-0.2437E+00-0.3172E-01	-0.4755E-01-0.1995E-01	0.4082E+00-0.7735E-03
80.00	155.00-0.1589E+00	0.5748E+00-0.1339E+00-0.1091E+00	0.4073E+00-0.1044E-03
80.00L	155.00-0.1589E+00-0.2411E-01	-0.9005E-01-0.1091E+00	0.4073E+00-0.4260E-03
430.00	0.00-0.3772E-01	0.4650E-01	0.3644E-01-0.7187E-02
430.00L	0.00-0.3772E-01	0.3609E-02	0.1283E-04-0.7187E-02
430.00	155.00-0.4141E-01	0.5092E-01	0.4255E-01-0.1170E-01
430.00L	155.00-0.4138E-01	0.3951E-02	0.9498E-03-0.1169E-01
			TaoRZ
			DispZ
			epZ
			epT
			epR
			0.2515E-03
			0.1750E-03
			0.9315E-04
			0.9315E-04
			0.1195E-03
			0.1196E-03
			0.1405E-03
			0.1405E-03
			0.4310E-03
			0.1810E-03
			0.1405E-03

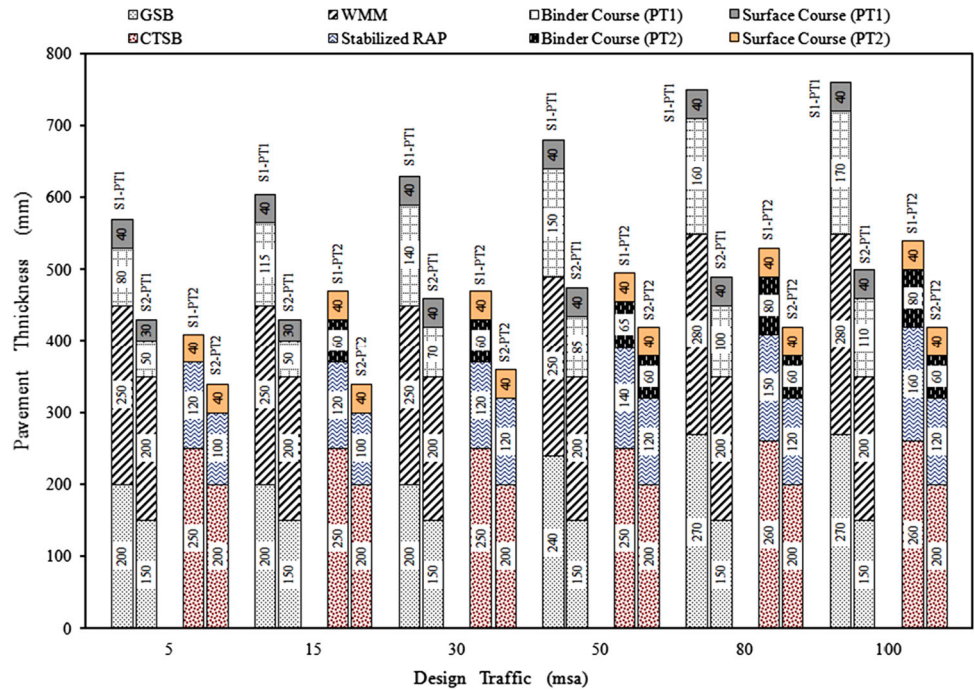
**Fig. 12** Typical output window of IITPAVE analysis for soil subgrade S2, pavement type PT1 under 5 msa

**Effect on Pavement Design due to Improvement of Soil Subgrade Overlaid by Compacted Fly Ash with Geotextile at Soil–Fly Ash Interface**

The soil subgrade types S3 and S4, respectively, represents the local soil subgrade and modified subgrade obtained by overlaying compacted fly ash over S3 with thickness ratio

of 1:1 with geotextile at soil–fly ash interface, at a molding water content of 16%, i.e., nearer to OMC of S3. Similarly, the soil subgrade types S5 and S6, respectively, represents the local soil subgrade and modified subgrade obtained by overlaying compacted fly ash over S5 with thickness ratio of 1:1 with geotextile at soil–fly ash interface, at a molding water content of 34%. Therefore, the results of IITPAVE

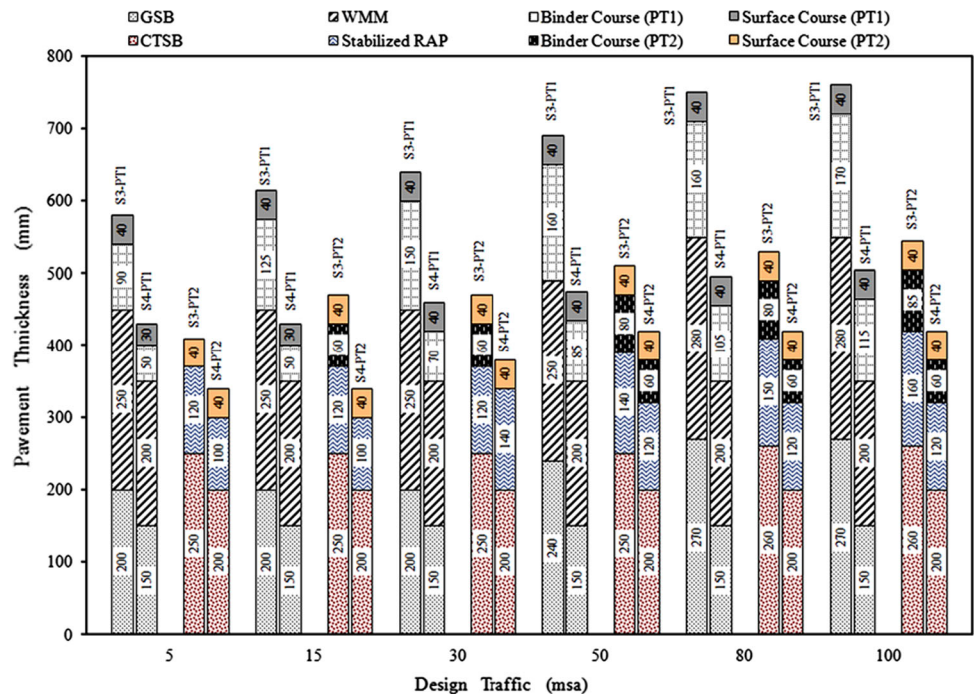
**Fig. 13** Design chart for both PT1 and PT2 corresponding to S1 and S2 soil subgrade types



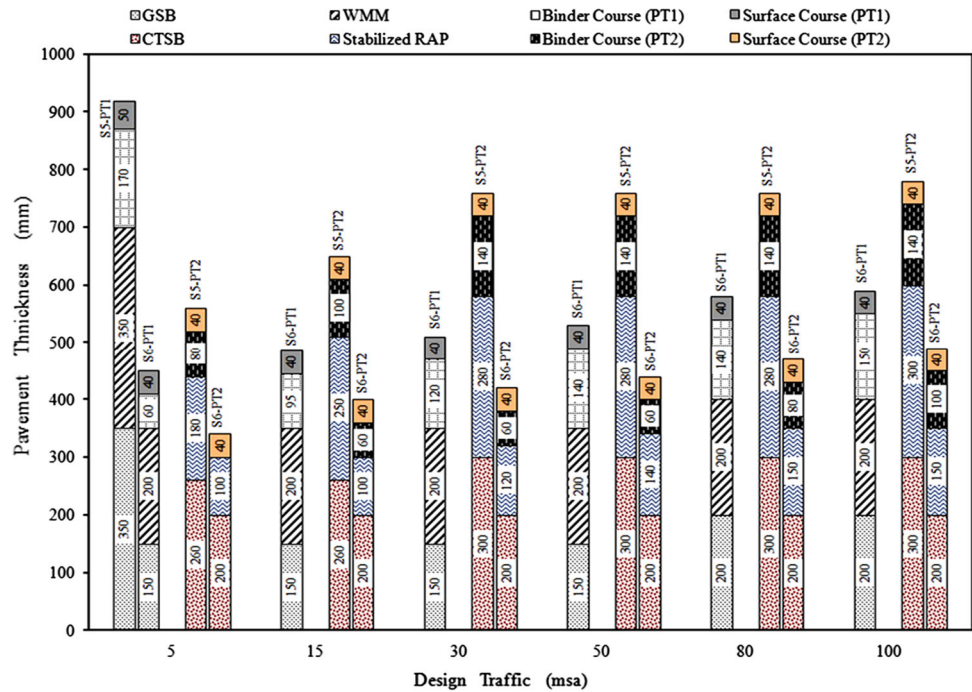
analyses for these four types of subgrades have been compared to study the effect of soil–geotextile–fly ash matrix method of subgrade improvement on pavement thicknesses. The variations of thicknesses of constituent pavement layers for both PT1 and PT2, corresponding to the critical strains, with respect to design traffic, are presented in Figs. 14 and 15, for S3, S4 soil subgrade types and for S5, S6 soil subgrade types, respectively. It can be

observed from the results that in case of local subgrade (S3) at low molding water content of 16%, an average reduction in pavement thickness of about 27.19% is obtained for alternative pavement section (PT2) when compared to corresponding values for conventional pavement section (PT1). While with subgrade modification by soil–geotextile–fly ash matrix method (S4), an average reduction in pavement thickness of about 30.34% and

**Fig. 14** Design chart for both PT1 and PT2 corresponding to S3 and S4 soil subgrade types



**Fig. 15** Design chart for both PT1 and PT2 corresponding to S5 and S6 soil subgrade types



20.87% has been observed, respectively, for PT1 and PT2 with respect to the corresponding values obtained for local subgrade. Also, a further reduction of 31.21% has been seen if PT2 is preferred over PT1.

However, at high molding water content of 34%, the design of conventional pavement section (PT1) could not be achieved for local subgrade (S5), as trial pavement thickness of value as high as 1500 mm has been found to fail in IITPAVE analysis. But alternative pavement section (PT2) could be easily designed resulting in economy in pavement construction (Table 15). Whereas with subgrade modification by soil–geotextile–fly ash matrix method (S6), the design of conventional pavement section (PT1) could be attempted yielding comparative values similar to those for S4. Also, an average reduction in pavement thickness of about 39.48% has been observed for PT2 with subgrade modification (S6) when compared to the corresponding values obtained for local subgrade (S5).

Therefore, it can be inferred from the above observations that the method of improvement of soil subgrade overlaid by compacted fly ash with geotextile at soil–fly ash interface proves to be a better means of enhancing the soil characteristics under worst-case scenario of high water content in field. Whereas the first method of soil improvement by addition of lime and RHA yields somewhat similar results as compared to that of the soil–geotextile–fly ash matrix method at low water content in the neighborhood of OMC. Also PT2, i.e., bituminous surface course with reclaimed asphalt pavement (RAP) material treated with foamed bitumen/bitumen emulsion and cement

treated sub-base (CTSB) proves to be a economical pavement section and better alternative to conventional bituminous pavement (PT1).

### Conclusions

The following conclusions may be drawn from the present study.

- A design chart has been developed for local and stabilized subgrades for traffic intensities of 5, 15, 30, 50, 80 and 100 msa for both conventional bituminous pavement and alternative pavement with RAP layer with CTSB sub-base.
- It has been observed that maximum reduction in pavement thickness is achieved for traffic intensity of 80 msa and 15 msa in case of conventional bituminous pavement and alternative pavement, respectively. On the other hand, minimum thickness reduction is observed for 5 msa in case of both pavement types.
- 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 soaked condition, the maximum value of improvement factor is found to be 8.98 which occurs 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 33.83 which occurs for thickness ratio of 2:1 at

highest molding water content of 34%. But at a molding water content of 16% (near to OMC), the improvement factor is observed to be 5.71 for same thickness ratio.

- With subgrade modification by addition of lime and RHA (S2), an average reduction in pavement thickness of about 29.92% and 21.04% has been observed, respectively, for conventional bituminous pavement and alternative pavement with respect to the corresponding values obtained for local subgrade. Whereas with subgrade modification by soil–geotextile–fly ash matrix method (S4) at low molding water content of 16%, the average reduction in pavement thickness of about 30.34% and 20.87% has been observed, respectively, for conventional bituminous pavement and alternative pavement with respect to the corresponding values obtained for local subgrade. Therefore, the method of soil improvement by addition of lime and RHA yields somewhat similar result at low water content (near to OMC) when compared to the method of improvement of soil subgrade overlaid by compacted fly ash with geotextile at soil–fly ash interface.
- At high molding water content of 34%, the design of conventional pavement section could only be made possible with subgrade modification by soil–geotextile–fly ash matrix method. Also, an average reduction in pavement thickness of more than 50% and about 39.48% has been observed, respectively, for conventional pavement section and alternative pavement section, with subgrade modification (S6) when compared to the corresponding values obtained for local subgrade (S5). Therefore, the method of improvement of soil subgrade overlaid by compacted fly ash with geotextile at soil–fly ash interface proves to be a better means of enhancing the soil character at high water contents.
- From IITPAVE analyses, it can be observed that pavement type 2—bituminous surface course with reclaimed asphalt pavement (RAP) material treated with foamed bitumen/bitumen emulsion and cement treated sub-base (CTSB) proves to be better alternative

and economical pavement section over conventional bituminous pavement. The reduction in pavement thickness for PT2 has been found to be of about 29.68% and 31.21% when compared to corresponding values of PT1 in case of soil subgrades modified by soil–lime–RHA matrix method and soil–geotextile–fly ash matrix method, respectively.

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## References

1. Chakraborty S (2014) 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
2. Sengupta A, Mukherjee S, Ghosh A (2017) Improvement of bearing ratio of clayey subgrade using compacted flyash layer. *Geotech Geol Eng* 35(4):1885–1894
3. Vorobieff G, Murphy G (2003) A new approach to pavement design using lime stabilized subgrades. In: *Proceedings of 21st Australian road research board conference*, pp 1–10
4. Siddique A, Rajbongshi B (2005) An analytical study on design and analysis of stabilized rural roads. *Proc Eastern Asia Soc Transport Studies* 5(8):13–28
5. Ziari H, Khabiri MM (2007) Interface condition influence on prediction of flexible pavement life. *J Civil Eng Manage* 1(7):1–6
6. Ekwulo EO, Eme DB (2009) Fatigue and rutting strain analysis of flexible pavements designed using CBR methods. *African J Environ Sci Technol* 3(4):12–21
7. Bagui SK (2012) Pavement design for low volume roads using cement and lime treated base. *Jordan J Civil Eng* 6(3):293–303
8. Selvi P (2015) Fatigue and rutting strain analysis on lime stabilized subgrades to develop a pavement design chart. *Transp Geotech* 2:86–98
9. IRC: 37–2018: Guidelines for the design of flexible pavements, (Fourth revision)

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