CHAPTER-X

PERFORMANCE OF HOMOGENEOUS AND STRATIFIED SOIL-ASH DEPOSIT AS A PAVEMENT MATERIAL

10.1 INTRODUCTION

The pavement subgrade acts as a foundation layer for the sub-base, base, and surface course in the flexible and rigid pavement. According to IRC-37 (2018), there are some particle size gradation guidelines for the sub-base, base, and other layers that limits the scope of the coal ash utilization. Among pavement layers, the subgrade layer possesses a high potential of mass utilization of material because the minimum thickness of subgrade layer is 500 mm. High quantity of coal ash can be utilized when it is used for subgrade as well as an embankment fill material. These coal ash has also been introduced in the highway construction in India such as Nizamuddin Bridge Project (New Delhi), National highway development programme (NH-6) from Dankuni to Kolaghat (West Bengal), NH-2 Okhla fly over project (New Delhi), Sarita Vihar fly over (New Delhi) etc. (RDSO- GE:0-S005). Considering the minimum required California bearing ratio (CBR) criteria, the homogeneous and stratified soil-ash deposit has been incorporated over the existing soil by verifying the effective modulus of layered soil arrangements. The subgrade should be compacted with the density greater than or equal to 97% of the maximum dry density with corresponding moisture content from the density-water content plot. It is essential to design the pavement based on 4 days soaked CBR, but the strength of the subgrade will be highly underestimated for the area having low rainfall intensity. Therefore, the area which experiences annual rainfall less than 1000 mm can omit the soaked CBR condition.

In this section, an attempt has been made to analyze the thickness of the flexible pavement of both the homogeneous and stratified soil-ash arrangements. The whole analysis was done using IIT-Pave software performing multiple iterations. Also, the stratified arrangements were checked with different forms of reinforcement placed in between fly ash and local soil.

10.2 MECHANISTIC-EMPIRICAL DESIGN METHODOLOGY

The design of the flexible pavement is usually accomplished by satisfying the two general performance criteria of strain at subgrade and at bituminous layer, as discussed in IRC-37 (2018). The methodology is based on the mechanistic empirical approach that is derived considering the linear elastic layered theory principle. Based on this theory, the critical strains were considered at the bottom of the bituminous layer (tensile strain) and at the top of the soil subgrade (vertical strain). The brief details regarding the performance criteria, i.e., (a) subgrade rutting criteria, and (b) fatigue cracking criteria, has been explained below and also presented in pictorial form in Fig. 10.1.

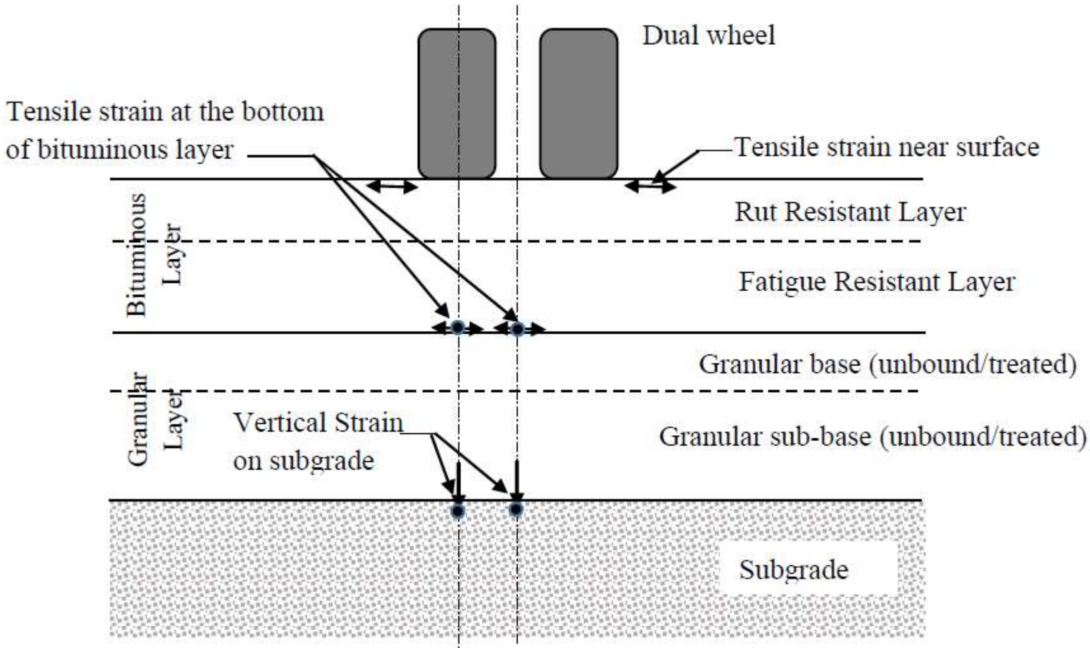


Fig. 10.1. The schematic diagram of flexible pavement with various layers illustrating critical strains (IRC-37-2018).

10.2.1 Criteria for Rutting of Subgrade

This criterion is to evaluate the depth of the ruts formation due to the application of standard axle load (80 kN). The pavement is assumed to be failed when the critical depth of ruts is 20 mm or more. Therefore, the permissible vertical compressive strain is calculated from the mechanistic model expressed as in the equation 10.1 & 10.2. The rutting performance model, which was initially establish using performance data from the MoRTH R-6 Research scheme (1995), was later divided into two independent models for two different reliability levels using the extra performance data obtained for the MoRTH R-56 Research scheme (1999). These equations will be used to determine the permissible vertical strain that is allowed during the entire loading span.

$$N_R = 4.1656 \times 10^{-08} \left[\frac{1}{\varepsilon_v}\right]^{4.5337} \qquad \text{(for 80\% reliability)} \qquad (10.1)$$

$$N_R = 1.4100 \times 10^{-08} \left[\frac{1}{\varepsilon_v}\right]^{4.5337} \qquad \text{(for 90\% reliability)} \qquad (10.2)$$

Where, N_R is the rutting life of the subgrade (cumulative equivalent number of 80 kN standard axle loads that can be served by the pavement before the critical rut depth of 20 mm or more occurs), and ε_v is the vertical compressive strain that develop at the top of the subgrade (estimated with the use of linear elastic layer theory when subjected with standard axle load on the pavement surface)

10.2.2 Criteria for Fatigue Cracking of Bituminous Layer

Similar to the rutting criteria, the fatigue criteria are implemented to incorporate the development of percentage of tensile strain at the bottom of top wearing course (bituminous layer). This strain should be restricted and not allowed to exceed beyond 20% of the cracked surface area when subjected with an equivalent number of standard axle load. The fatigue performance models provided by equations 10.3 and 10.4 were created under the MoRTH R-56 scheme (1999) using performance data principally from the R-6 scheme (Benkelman Beam Studies) (1995) with the addition of data from the R-19 (Pavement Performance Studies) (1994) and R-56 schemes (1999). The expression for the maximum permissible tensile strain that needs to be satisfied while designing are mentioned below.

$$N_f = 1.6064 \times C \times 10^{-04} \left[\frac{1}{\varepsilon_t}\right]^{3.89} \times \left[\frac{1}{M_{Rm}}\right]^{0.854} \quad \text{(for 80\% reliability)} \qquad (10.3)$$

$$N_f = 0.5161 \times C \times 10^{-04} \left[\frac{1}{\varepsilon_t}\right]^{3.89} \times \left[\frac{1}{M_{Rm}}\right]^{0.854} \quad \text{(for 90\% reliability)} \qquad (10.4)$$

$$C = 10^M \qquad (10.5)$$

$$M = 4.84\left(\frac{V_{be}}{V_a+V_{be}} - 0.69\right) \qquad (10.6)$$

Where, N_f is the fatigue life of the bituminous layer (cumulative equivalent number of 80 kN standard axle loads that can be served by the pavement before the critical cracked area of 20 % or more of paved surface area occurs), ε_t is the horizontal tensile strain produces at the bottom of the bituminous layer (estimated with the use of linear elastic layer theory when subjected with standard axle load on the pavement surface), M_{Rm} is the bituminous layer's resilient modulus, V_a is the volume of air void in percent, and V_{be} is the volume of the effective bitumen in percent.

10.3 DESIGN OF FLEXIBLE PAVEMENT

10.3.1 Design of Unreinforced Flexible Pavement

The mechanistic empirical design has been adopted for the design of the flexible pavement. The procedure includes the determination of permissible above-mentioned strains for a particular considered traffic loading. After that, based on the CBR of the subgrade, the resilient modulus (M_{RS}) has been estimated that helped in the estimation of the resilient modulus of the above layers as per the assumed thickness. The equations required for the evaluation of resilient modulus of subsequent layers are listed below in equation between 10.7-10.10 (Shell pavement design manual 1978 & Powell et al. 1984).

$$M_{RS} = 10.0 \times CBR \qquad \text{for CBR} \leq 5\% \qquad (10.7)$$

$$M_{RS} = 17.6 \times (CBR)^{0.64} \qquad \text{for CBR} \geq 5\% \qquad (10.8)$$

$$M_{R\_gsb} = 0.2 \times h^{0.45} \times M_{R\_sg} \qquad (10.9)$$

$$M_{R\_gb} = 0.2 \times h^{0.45} \times M_{R\_gsb} \qquad (10.10)$$

Where, M_{Rs} is the resilient modulus of the subgrade material calculated employing the CBR of the subgrade material, M_{R_gsb} is the resilient modulus of the granular sub-base, M_{R_gb} is the resilient modulus of the granular base.

Once the resilient modulus of the layers has been achieved, then these resilient moduli for a specific assumed thickness was analyzed using IIT-PAVE pavement analysis software. This analysis was executed considering one set of dual wheels carrying 20 kN (each) load with centre to centre spacing of 310 mm between the wheels. The Poisson's ratio of 0.35 and four analysis point [(0,0), (t_B, 155), (0,0) & (t_T, 155)] at the bottom and top of the bituminous and subgrade layer respectively (where t_B is the thickness of the bituminous layer and t_T is the total thickness of the pavement layer) were considered. The design outcome of the IIT-PAVE software is in the form of stress, strain, and deflection at the analysis locations. The maximum strain at the critical strain location was identified and compared with the permissible strain for a particular traffic condition. If the critical strain evaluated from the software is lower than the permissible, then the assumed pavement layer thickness can be considered as safe. Otherwise, the analysis should be repeated for the different thickness of the pavement layers.

10.3.2 Design of Geosynthetic-Reinforced Flexible Pavement

The design of geosynthetic-reinforced flexible should be done in two steps, a) first design the pavement section treated as unreinforced as per IRC-37 (2018), and b) then determine the improved layer parameters due to the inclusion of geosynthetics which is used for the thickness evaluation of reinforced pavement. There are four approaches developed for the reinforced pavement design such as modified AASHTO method, Traffic benefit ratio (TBR), Layer coefficient ratio (LCR), and Modulus improvement factor as per IRC-SP-59 (2019). The LCR approach has been implemented in the present study considering the convenient and simple design procedure. LCR is basically represents the enhancement provided by the geosynthetics to the layer coefficient of the layer in which the geosynthetics is placed. The prime objective is to determine the modified resilient modulus of the section in which the inclusion of geosynthetics has been done. In order to design reinforced section, initially the pavement must be design as unreinforced section and then the improved modulus will be determined using the following equations mentioned in IRC-SP-59 (2019) and AASHTO (1993).

$$a_2 = 0.249(\log_{10} M_{R\_GB}) - 0.977 \quad (10.11)$$

$$a_3 = 0.227(\log_{10} M_{R\_GSB}) - 0.839 \quad (10.12)$$

$$LCR_2 a_2 = 0.249(\log_{10} M^1{}_{R\_GB}) - 0.977 \quad (10.13)$$

$$LCR_3 a_3 = 0.227(\log_{10} M^1{}_{R\_GSB}) - 0.839 \quad (10.13)$$

Where a_2 and a_3 are the structural layer coefficients for base and subbase, M_{R_GB} and M_{R_GSB} are the resilient modulus of base and subbase layers. Similarly, $M^1{}_{R\_GB}$ and $M^1{}_{R\_GSB}$ are the modified resilient modulus of base and subbase layers.

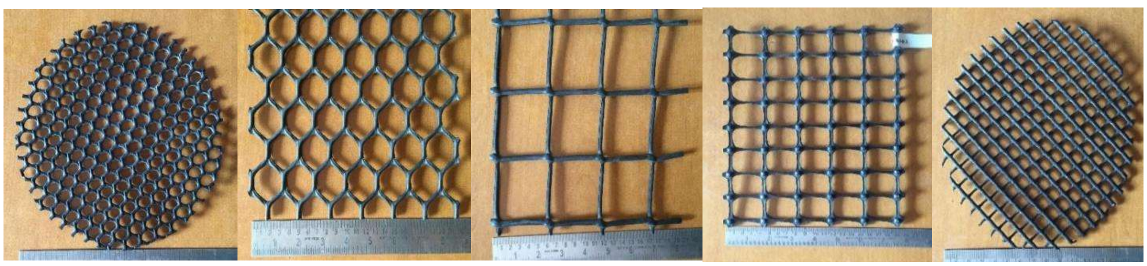
First, the layer coefficients will be determined (a_2 & a_3) with the conventionally determined modulus. After this, the layer coefficient will be modified by multiplying with LCR (LCR = 1.2 for the present study) from which the updated modulus can be back calculated. The updated modulus along with the rest input parameters will be implemented in IITPAVE for the revised thickness of pavement.

10.4 RESULTS AND DISCUSSION

The present investigation deals with the evaluation of the suitability of considered materials under different traffic conditions and experimental conditions. The outcomes of all the experimental and thickness evaluation of the flexible pavement were presented in the subsequent sections.

10.4.1 CBR Response of Unreinforced and Reinforced Soil Combinations

The California Bearing Ratio (CBR) test was performed confirming to the Indian standard IS:2720 (Part 16) (1987). The CBR response of all the soil combinations considering the field situation and inclusion of different reinforcement (Fig. 10.2) have been discussed in this section. Also, the physical and mechanical properties of geosynthetics were tabulated in Table 10.1 to 10.3. The schematic diagram showing the reinforcement position and two soil arrangements has been shown in Fig. 10.3. To ascertain the penetration of a 50 mm diameter plunger during loading, laboratory experiments were carried out immediately after sample preparation and after 96 hours of soaking conditions (prepared sample) that can be shown in Fig. 10.4 & Table. 10.4. The CBR samples of the soil were prepared at the maximum dry density compacted in three numbers of layers subjected to 56 numbers of blows to each layer. The fly ash exhibits higher CBR value for both soaked and unsoaked conditions followed by stratified and local soil. For the stratified soil subgrade or subgrade made up of two materials, it is recommended by IRC-37 (2018) to determine the effective CBR of combinations for the design. A subgrade is considered to be suitable for the pavement application when it exhibits effective CBR value greater than 5% to carry more than 450 commercial vehicles per day. The soaked CBR value of the present soil combination shows the lower value (< 2) than the required, hence in order to enhance its penetration resistance, the soil combinations have also been checked with different geosynthetic reinforcements. The reinforcement has been incorporated in the middle of the interface between fly ash and local soil only. Because the optimum position of geogrid was observed to be at the bottom of the layer for thickness <150 mm and between 1/3 to 1/2 of thickness from top for thickness >150 mm (Haas et al. 1988). The pictorial demonstration of the reinforcement has been shown in Fig. 10.2. The reinforcement used are high tensile strength geonet, biaxial geogrid, and Petroshield Rock Protection Padding Mesh (PRPP). With the inclusion of reinforcement, the CBR value is observed to be increased between 1.1 to 2.8 times that of the unreinforced CBR. The maximum improvement in the CBR has been observed in the case of biaxial geogrid whereas minimum in the case of PRPP reinforcement. The high performance of biaxial geogrid is due to the higher ultimate tensile strength in both machine and cross-machine direction (MD: 39.5 kN/m & CD: 38.9 kN/m) as compared to the other geosynthetics materials. In addition, the geogrid has grid pattern opening arrangements with junction strength equivalent to ultimate tensile strength, therefore it sustains high external load before failure. Hence, soil reinforced with geogrid provides great resistance from interlocking, frictional and lateral restraining along with the tensioned membrane effect for considerable penetration of plunger. Lateral restraint, increased bearing capacity, and tensioned membrane effect are three factors have been identified as the cause of the pavement's better performance as a result of the geosynthetic reinforcement (Giroud and Noiray 1981; Giroud et al. 1984; Perkins and Ismeik 1997; Holtz et al. 1998). Lateral restraining is the restriction of the lateral movement of pavement material provided by geosynthetics when subjected to vertical load. The interaction between the material and geosynthetics results in frictional and interlocking characteristics that ultimately increases the load resistance capacity. Similarly, the presence of geosynthetic layer reduces the shear stress coming from the external loading and enhance the failure mode from punching mode to general failure mode. Tensioned membrane effect comes into picture only when the layer above the reinforcement gets deformed. The deformation leads to sagging of the reinforcement that develops tension which contributes to support the vertical load. Similarly, the unsoaked CBR of the unreinforced specimens were significantly higher than that of the required for the pavement design, and also observed maximum improvement using biaxial geogrid. Therefore, from the experimental results, this can be concluded that the stratified soil-ash arrangement has a weak performance under soaked condition, but this can be improved by introducing biaxial geogrid. However, the stratified soil-ash deposit shows promising results under the unsoaked condition that can be directly used without any reinforcing materials.



(a) **(b)** **(c)** **(d)** **(e)**

Fig. 10.2. Pictorial illustration of different types of reinforcements, (a) Geonet 121, (b) Geonet 131, (c) Geonet 153, (d) Biaxial geogrid, and (e) PRPP (4 mm).

Table. 10.1. Physical and mechanical properties of Geonet considered in the present study.

Properties	**Unit**	**GN 121**	**GN 131**	**GN 153**
Length	m	25	25	25
Width	m	2.00 (±0.05)	2.00 (±0.05)	1.00 (±0.05)
Form	-	Sheet	Sheet	Sheet
Aperture shape	-	Diamond	Hexagonal	Square
Aperture size	mm	8×6(±0.25)	28×28(±3)	50×50(±5)
Thickness at joint	mm	3.1(±0.25)	5.75(±0.60)	6.7(±0.5)
Max. Strength	Kn/m	7.68	5.8	4.82

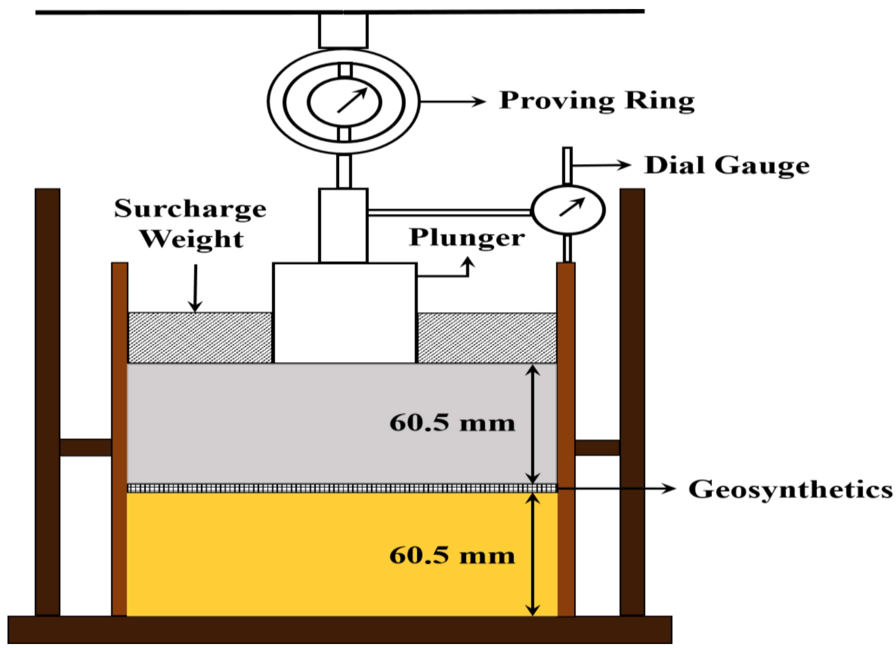


Fig. 10.3. Schematic diagram of CBR testing sample having equally distributed fly ash and local soil with reinforcement placed at center.

Table. 10.2. Physical and mechanical properties of Geogrid in machine direction (MD) and cross-machine direction (CD) considered in the present study.

Biaxial Geogrid	Aperture size		Roll width	Roll length	Ultimate Tensile strength		Strain at ultimate		Tension (at 2% strain)		Tension (at 5% strain)		Junction strength		Flexural rigidity	
	MD	CD			MD	CD	MD	CD	MD	CD	MD	CD	MD	CD	MD	CD
GG 4040	mm		m		Kn/m		%		Kn/m		Kn/m		Kn/m		mg-cm	
	38.7	40.5	3.9	50	39.5	38.9	18.8	9.9	13.8	14.6	28.1	28.8	37.1	35.7	3581386	3518476

Table. 10.3. Physical and mechanical properties of Petroshield rock protection padding mesh considered in the present study.

Group	Structure	Polymer composition	Length	Width	Thickness	Compressive strength	Impact resistance	Mesh angle	Mesh count
PRPP			m	m	mm	Kg/cm^2	Kg.cm/inch	Degree	Meshes/cm
	Extruded Mesh	Polyethylene	25	1.5	4.1	15	121	85	1.2

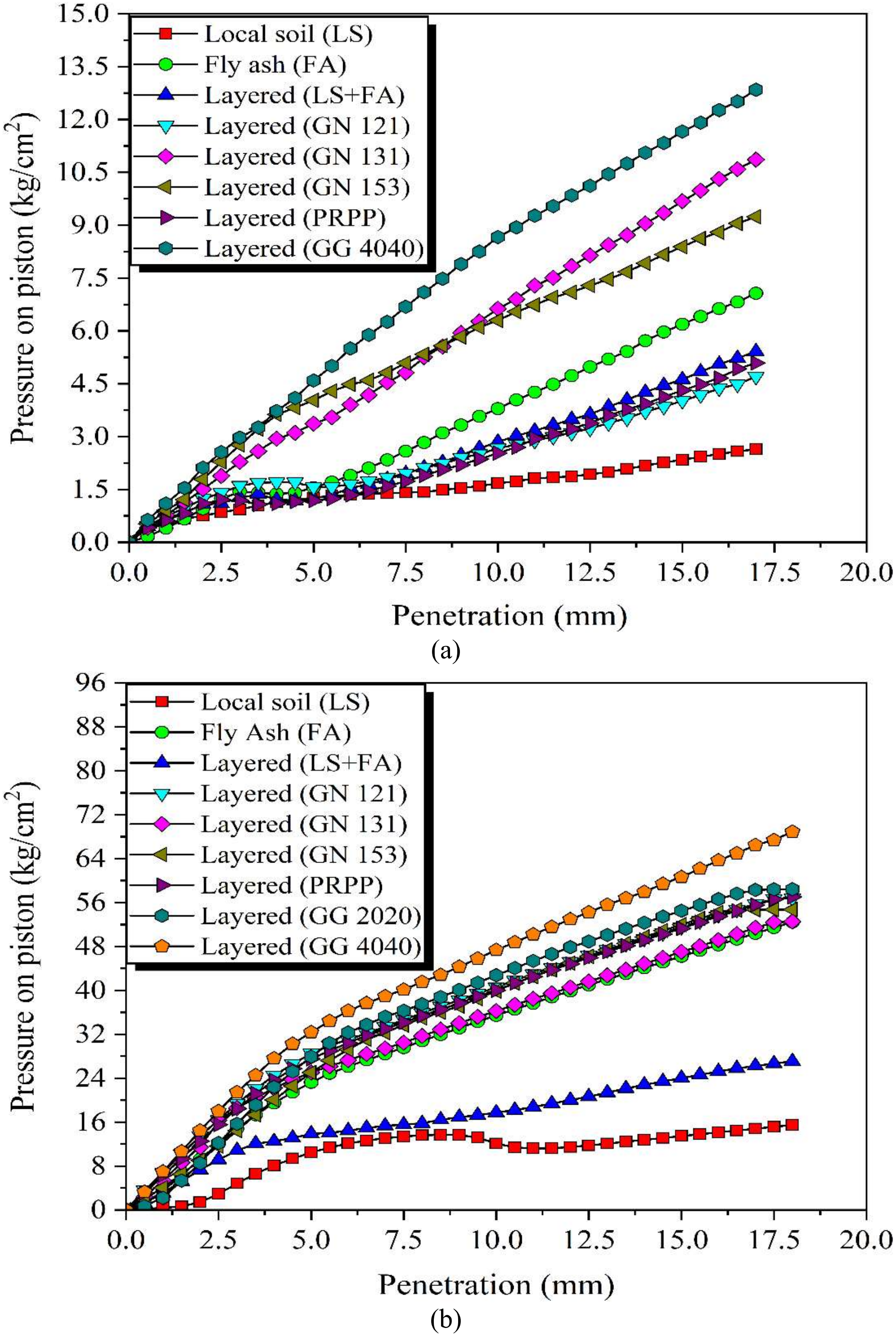


Fig. 10.4. California Bearing Ratio of all the soil combinations under, (a) soaked condition, and (b) unsoaked condition.

Table. 10.4. Tabular presentation of soaked and unsoaked CBR of various combinations of soil with and without reinforcement.

Testing arrangements	Penetration	Soaked CBR	Unsoaked CBR
	(mm)	(%)	(%)
Local soil	2.5	1.22	11.59
	5.0	1.20	12.05
Fly ash	2.5	1.77	17.29
	5.0	1.48	22.13
Layered soil	2.5	1.57	13.08
	5.0	1.18	13.23
Layered soil+Geonet121	2.5	2.00	23.46
	5.0	1.49	27.16
Layered soil+PRPP	2.5	1.73	22.20
	5.0	1.13	26.22
Layered soil+Geonet153	2.5	3.26	16.62
	5.0	3.84	23.84
Layered soil+Geonet131	2.5	2.69	23.38
	5.0	3.21	23.86
Layered soil+BG2020	2.5	2.48	17.40
	5.0	3.79	26.59
Layered soil+BG4040	2.5	3.65	25.73
	5.0	4.37	30.83

10.4.2 Thickness Evaluation of Pavement Resting on Present Materials as Subgrade

The fundamental regarding the flexible pavement thickness evaluation has been explained in the above section. The assessment of the thickness of the sub-base, base and surface course has been done by validating the necessary condition between the permissible strain and developed critical strain evaluated from the IIT-PAVE software as per IRC-37. The analysis was done considering the traffic loading intensity between 50-150 million standard axles (MSA), bituminous layer of viscosity grade 30 (M_R: 3000 MPa), Poisson's ratio of 0.35 which was subjected to a standard axle load of 80 kN. The pavement analysis was done for the unsoaked condition only due to the low CBR value under soaked condition. Although the consideration of soaked CBR underestimates the subgrade soil for the place with a low rainfall intensity (<1000 mm).

Considering the low rainfall condition in Varanasi area, the unsoaked CBR can also be applied for the analysis. Whereas for high rainfall intensity locations the reinforcement such as biaxial geogrid can be used for enhancing the load resistance behaviour. The outcomes of the pavement thickness estimated after multiple iterations are reported in the Fig. 10.5, 10.6, & 10.7. Fig. 10.5 shows the comparison of the pavement layer thickness between the unreinforced local soil and stratified soil-ash deposit. Since, the cost of the material and execution of the top bituminous layer is high as compared to the other layers, therefore the comparison was only carried out between thickness of the top layer by keeping the constant thickness of the other layers. The results depict the reduction in the thickness of 10 mm and 5 mm for low and high traffic condition in the case of stratified soil-ash deposit. In the same way, the stratified soil-ash arrangements have also been checked for thickness reduction (Fig. 10.6) and found a consistent reduction in thickness with reinforcement type. The biaxial geogrid exhibits a maximum reduction of 130 mm when compared to the unreinforced one. Furthermore, this analysis has also been performed for various traffic loading conditions, in order to evaluate the magnitude of reductions. The presented result observed high thickness reduction for low traffic condition and vice-versa. The reduction of thickness has been observed as 190 mm, 145 mm, and 130 mm for the traffic loading of 50 MSA, 100 MSA, and 150 MSA respectively (Fig. 10.7). These results confirm the satisfactory performance of the stratified soil-ash arrangement with and without reinforcement condition. Hence, these arrangements can be successfully implemented in the low rainfall areas, whereas for high rainfall areas some ground improvement or reinforcements are required for the sustainable applications.

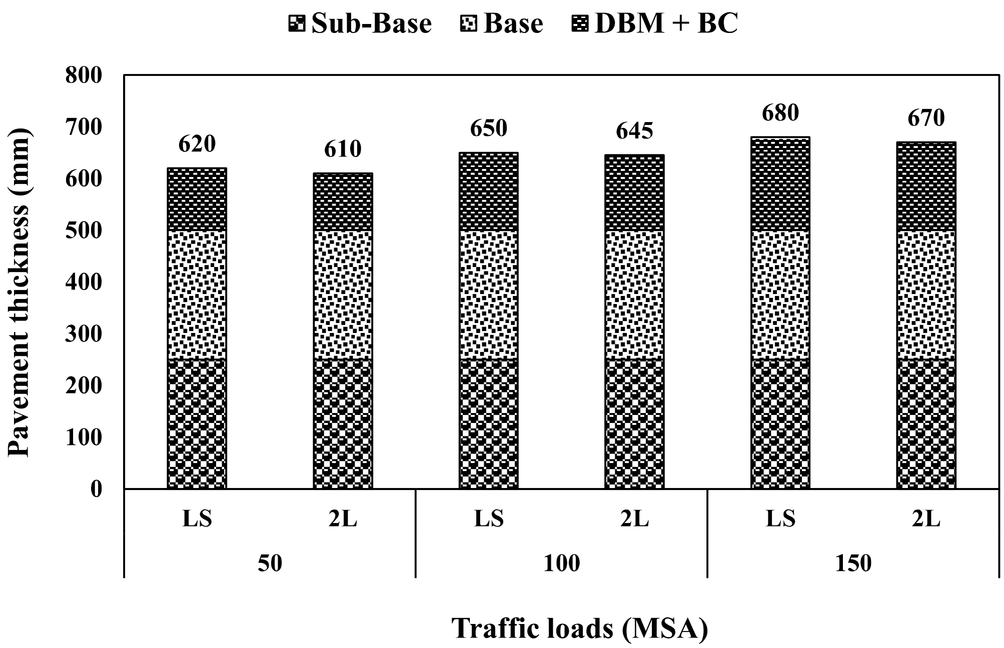


Fig. 10.5. Evaluation of the pavement thickness for local soil and layered soil-ash arrangements.

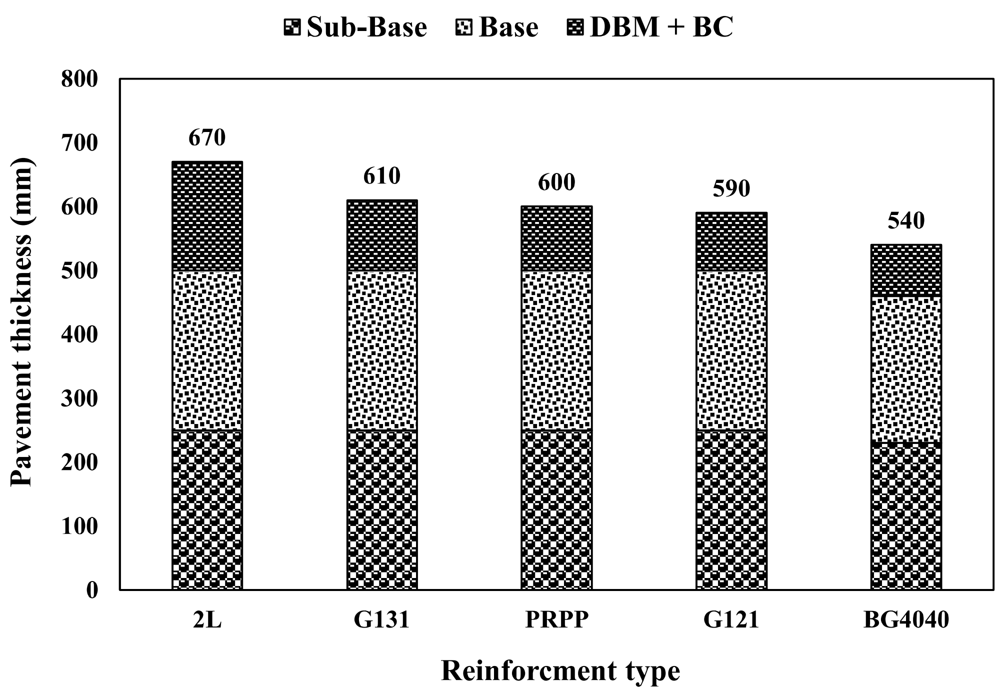
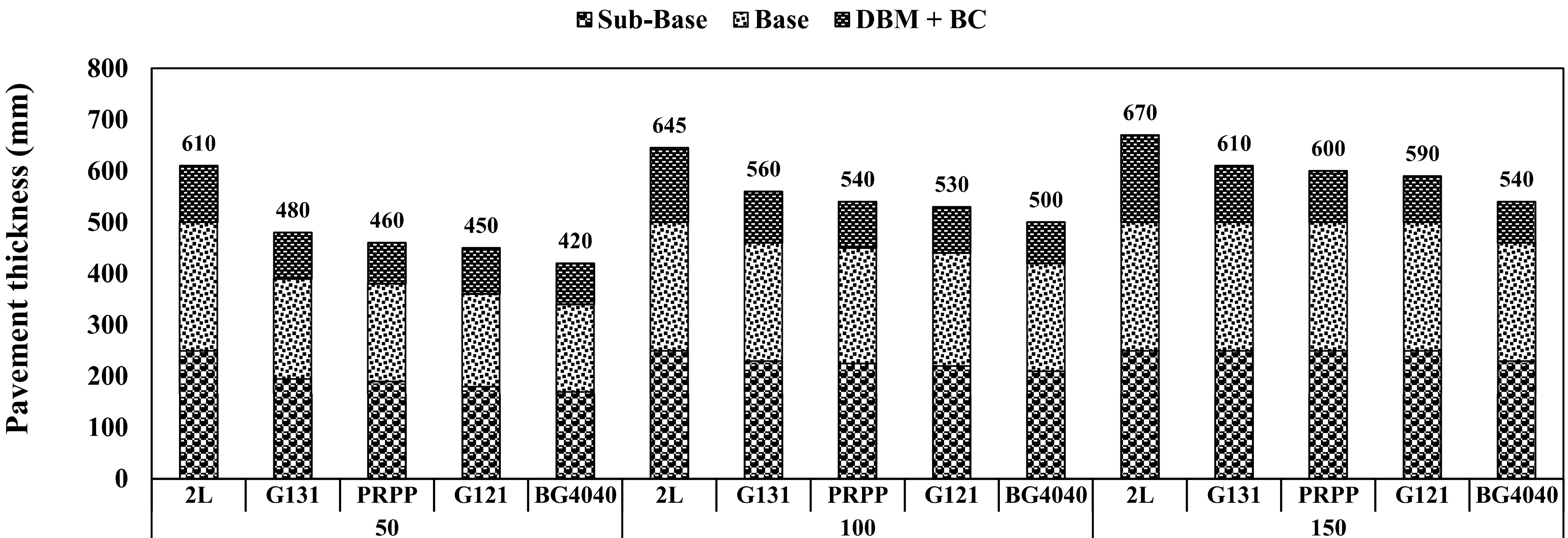


Fig. 10.6. Assessment of the pavement thickness for layered soil-ash arrangements with various reinforcements



	50					100					150				
	2L	G131	PRPP	G121	BG4040	2L	G131	PRPP	G121	BG4040	2L	G131	PRPP	G121	BG4040
Total	610	480	460	450	420	645	560	540	530	500	670	610	600	590	540
DBM + BC	110	90	80	90	80	145	100	90	90	80	170	110	100	90	80
Base	250	195	190	180	170	250	230	225	220	210	250	250	250	250	230
Sub-Base	250	195	190	180	170	250	230	225	220	210	250	250	250	250	230

Traffic load (MSA)

Fig. 10.7. Typical representation of the flexible pavement thickness considering various reinforcement under different traffic condition.

10.5 SUMMARY

The purpose of the current study was to determine the suitability of the stratified soil-ash arrangements (fly ash + local soil) in the application of pavement engineering. The design of the flexible pavement was done by following the guidelines of IRC-37 and it was analyzed using the software IIT-PAVE. Also, the complete pavement thickness analysis was performed considering the soaked/unsoaked condition with and without the inclusion of different reinforcements. The conclusions of this experimental investigation can be drawn as follows:

The study concluded that the stratified soil-ash arrangements offer a more economical solution for the flexible pavement design as compared to the local soil, regardless of the loading conditions considered. Among the various reinforcements tested, the biaxial geogrid exhibited promising strength improvement, as evidenced by higher CBR (California Bearing Ratio) values. The unreinforced stratified soil-ash deposits experienced significant thickness reduction, especially under low traffic conditions, while the inclusion of reinforcement helped in mitigating this effect. It was also found that the stratified soil-ash arrangements are suitable for the implementation in low-rainfall areas. However, in high rainfall areas, additional ground improvement measures or reinforcements are necessary for sustainable applications. These findings provide valuable insight for designing cost-effective and resilient, flexible pavements using stratified soil-ash arrangements.