Finite Element Modeling Of Lime Stabilized Low Volume Rural Roads

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ABSTRACT

The present study was undertaken to evaluate the strength characteristics of lime stabilized subgrade soils. Two types of soils (Soil – A and Soil – B) and one type of lime was selected for present study. Maximum Dry Density, L.L., P.L. and CBR test w1ere conducted on these soils stabilized with 2.5 %, 5 %, 7.5 % and 10 % lime content. It was observed that 7.5 % lime content will be the optimum for getting maximum benefits. The four days soaked CBR value of subgrade Soil – A and Soil – B was 1.0 % and 1.76 % respectively, it was increased to 6.51 % and 5.91 % respectively due to stabilization with 7.5 % lime content.

The static triaxial test were conducted on unstabilized and stabilized subgrade with 7.5 % lime content as well as on other pavement layers at a confining pressure of 40 kPa and stress- strain curve were plotted. These stress-strain data further used as a input parameter in elasto-plastic finite element modeling. The vertical compressive strain developed at top of unstabilized and stabilized subgrade further used for estimation of extension in service life or reduction in layers thickness of the low volume rural roads.

Keywords-CBR, Finite element modeling,lime stabilization,LTR,Rural road,TBR

1. GENERAL

Rural roads are the tertiary road system in total road network which provides accessibility for the rural habitations to market and other facility centers. In India, during the last six decades, rural roads are being planned and programmed in the context of overall rural development. While building rural roads, the provisions based on the parameters that affect the sustainability are to be made at minimum cost. Rural roads in India are constructed over difficult and poor sub grade. Such subgrades having low strength this leads to more thickness of the pavement. It is the duty of the engineers to spend every rupee of the tax payer's money with optional utility particularly under resource constraints. This calls for introduction of innovative approaches in rural road building for achieving cost-effectiveness. A finite element model of pavement layered structures provides the most moderate technology and sophisticated characterization of materials that can be easily accommodated in the analysis. The escalating cost of materials and energy and lack of resources available have motivated highway engineers to explore new alternatives in building rural roads and rehabilitating the existing ones. Stabilizing the subgrade with lime or cement is one such alternative. Recently considerable interest has been generated among both highway engineers and contractors for these materials as a stabilizer in the low volume rural roads. However, absence of a well documented design procedure for stabilized pavement has resulted in low confidence in highway engineers in using these materials.

2. LITRATURE REVIEW

The world wide literature review has been conducted on soils stabilized with lime, cement, fly ash, mixture of these material and fibre reinforced soil mixed with lime, cement or fly ash presented here.

Consoli et al[1] carried out drained triaxial compression test to study the individual and combine effect of cement and randomly distributed fibre inclusions on the properties of silty sand. Lima et al.[2] observed large increase in compressive strength due to addition of lime and cement to fibre reinforced soils. Schaefer et al.[3] reported amount of cement required for stabilizing expensive soils in the range of 2 % - 6 % by weight of soil. Cocka [4] reported that the lime, cement and fly ash reduced the swelling potential of expansive soil. The lime and cement were introduced as an admixture up to 8 % by weight of soil. Consoli.et al.[5] conducted unconfined compression strength test and triaxial strength test to evaluate the behavior of sandy soil stabilized with lime and fly ash. They conducted the test on sandy soil with 25 % fly ash and 1 % - 7 % carbide lime. The results have shown that maximum dry density of sandy soil decreases due to stabilization with lime and fly ash but there is marginal effect on optimum moisture content. They also reported that the rate of gain of strength is the function of percentage of lime and curing time. Schnaid et al. [6] studied the stress-strain behavior of cement stabilized sand through unconfined compression strength test and triaxial compression test. They reported that the initial tangent modulus of uncemented sand at confining pressure of100

kN/m^2 was 54 MN/m^2 this value was increased to 600 MN/m^2, 3090 MN/m^2 and 10000 MN/m^2 due to addition of 1%, 3% and 5% cement respectively.

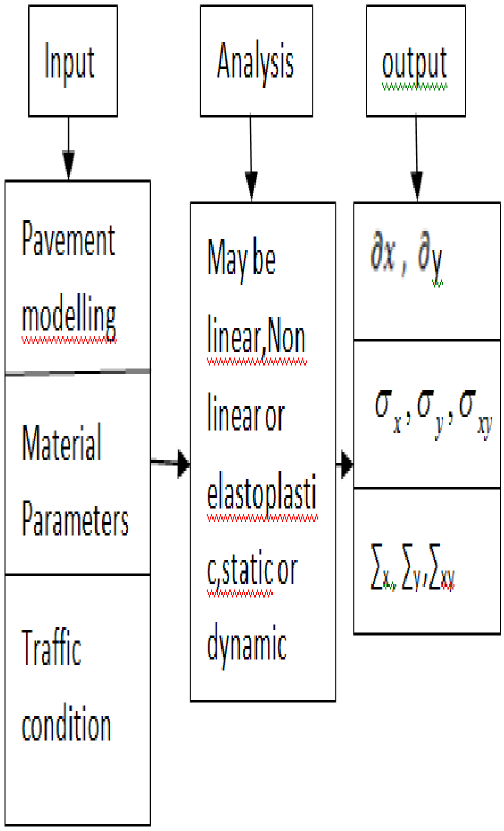
The soil stabilization with lime, cement and fly ash has been tried for many years and there is considerable improvement in strength properties. Available literature shows that most of the research works on cement and lime stabilization is related to geotechnical aspect only. Very few attempts have been made on use of cement or lime in highway subgrade. Conflicting results have been reported in literature regarding optimum percentage of lime or cement required for soil stabilization. The present study was undertaken to observe the effect of lime stabilization on properties of soil important in design and analysis of low volume rural roads.

3. NEED FOR DEVELOPMENT OF DESIGN CHARTS

The present study is concentrated on the development of design charts for rural roads to convert the locally available troublesome soil to suitable construction material. The design charts are particularly customized for low volume rural roads. The Indian Practice Code IRC 37-2001 [7]for different traffic intensity and IRC: SP: 72-2007 [8](Guidelines For the design of flexible pavements for low volume rural roads) are used as the standard for designing. These practice code uses 4 days soaked CBR value of subgrade soil for design. But the major draw back of CBR method is, it is penetration test and maximum penetration of plunger is as high as 12.5 mm which may not be realize in actual practice. Keeping in view the above draw backs of existing methods it is urgent need to develop the mechanistic – empirical pavement (M-E pavement) design approach for design of Low volume rural roads. This methodology has better capability to characterization of different material properties and loading conditions and has ability to evaluate different design alternatives on economical basis.

3.1 How Does Mechanistic Pavement Work?

The following description is necessarily somewhat generic and based primarily on the analysis of flexible pavement; however the system has been designed in a modular fashion, which with the modular nature of the software, allows the same elements of design with type-specific sub-modules. The M-E pavement design guide performs a time-stepping process, illustrated in the diagram below.



Outline of M-E pavement design guide process

4. EXPERIMENTAL PROGRAM

4.1 Material Selection Two types of soils and one type of stabilizer namely lime is selected in present study. The soil used available in the campus of Sardar Patel College of Engineering, Andheri (W) Mumbai. In the present study this soil is referred to as subgrade Soil – A and Soil - B. The index properties; liquid limit, plastic limit and plasticity index were determined as per ASTM-D 4318.[9] The Proctor's tests were conducted as per ASTM-D 1557[10] for deciding the maximum dry density (MDD) and the optimum moisture content (OMC) for of soil. Fig.1 shows the particle size distribution curve for subgrade soil – A and B. The important soil properties as per HRB and US soil classification systems are presented in Table 1. The Soil - A is clay of low Plasticity (A-6) and soil – B is of sandy silt of low plasticity (A-2-6).

TABLE 1 physical properties of subgrade Soil-A and soil-B

Property	Soil-A	Soil-B
Dry Density(kN/m^3)	14.25	16.70
OMC(%)	31.00	21.90
CBR(%)	1.00	0.90
D_{50}	0.07	0.05
Liquid Limit,(%)	50.00	36.00
Plastic Lit,(%)	28.12	29.12
Plasticity Index,(%)	21.88	12.08
Unified Soil Classification	CL	ML
Classification as per HRB	A-6	A-2-6
Typical Name	Clay Of Low Plasticity	Sandy Silt of law Plasticity

4.2 Determination of Optimum Quantity of Lime

4.2.1 Effect on Proctor's Test

The standard proctor tests were carried out on the unstabilized and stabilized soils subgrade Soil – A and B as per ASTM D1557 [10].Soil mixed with different percentages of lime varies from 2.5 % to 12.5 % at the step of 2.5 % by dry weight of soil. The dry density – moisture content relations were plotted for each test. Then optimum moisture content and maximum dry density at each percentage of lime content were evaluated. Fig. 2 gives typical plot showing variation of dry density with moisture content for unstbilised subgradet soil –A and soil stabilized with 2.5 % lime content. Similar plot are made for other test condition and variation of maximum dry density and optimum moisture content for stabilized soil at different lime content have been summarized in Table 2. The maximum dry density for these soils decreases gradually with increase in the lime content, which is due to the light weight of the lime replacing the soil particles and some of the applied energy of compaction absorbed by the lime.The change in OMC was quite marginal.

TALE 2 Effect of Stabilization on Dry Density of subgrade soils

Lime Content	Soil - A		Soil - B	
	Max. Dry Density (kN/m^3)	OMC	Max. Dry Density (kN/m^3)	OMC
0	14.25	31.00	15.80	25.00
2.5	14.15	32.00	14.53	25.73
5	14.05	31.50	13.49	24.30
7.5	13.85	29.88	13.48	23.07
10	13.70	29.98	13.22	21.96
12.5	13.80	29.00	13.32	22.89

4.

2.2 Effect on California Bearing Ratio (CBR) Test

CBR tests were conducted on unstabilised and stabilized soils with different lime contents as per ASTM D1883[11] The maximum limit of lime content was 12.5 percent. A total of 14 samples were tested for the subgrade soil - A and B at different lime content. Weight of soil sample at each percentage of lime content required for test is determined by volume of mould and corresponding maximum dry density obtained from proctor test. Soil sample and lime is mixed properly in dry state and water corresponding to Optimum Moisture Content (OMC) is added. The soil sample after mixing is filled in the mould and compacted by static compaction. It is soaked in water for four days and the sample is tested in CBR testing machine. The CBR was determined at 2.5 mm and 5.0 mm penetration levels and maximum of this is adopted as CBR value. CBR values at different lime content and percentage increase in CBR with respect to unstabilized soils are presented in Table 3. As can be seen, the CBR value of unstabilized soil – A and B is 1 % and 1.76 % this increase to 6.51 % and 5.91 % due to addition of 7.5 % lime respectively, thereafter it start decreasing. It shows that maximum improvement in CBR is observed when subgrade soil – A and B stabilized with 7.5 percent lime content. Also, it indicates that lime stabilization is more effective for weak soil compared to stronger one.

TABLE 3 Effect of Stabilization on CBR value of subgrade soil

Lime content	Soil – A		Soil - B	
	CBR Value	% increase	CBR Value	% increase
0	1.00	---	1.76	---
2.5	2.74	174	2.33	57.00
5	3.89	289	4.12	236.00
7.5	6.51	551	5.91	415.00
10	5.83	483	5.76	400.00
12.5	5.89	489	5.80	404.00

4.2.3 Effect on Elastic Modulus (E_i - value) and Failure Stress (σ_f) The triaxial tests were conducted on unstabilized and soil stabilized with optimum lime content. The sixteen specimens were prepared for unstabilized and stabilized subgrade soil and tested at confining pressures of 40, 70,120 and 150 kPa. The specimens of ize 10 cm diameter and 20 cm height were prepared in split mould. The soils were dried at 100^0 C for 24 hours, pulverized manually and sieve it through 4.75 mm sieve. The weight of lime and soil were calculated using the Proctor density and volume of split mould. The lime were mixed in soil in dry state and water corresponding to OMC was added and mixed thoroughly again. The moist mixture was transferred to split mould in three equal layers; each layer compacted by a light weight hammer of weight 1.5 kg. Around 25 numbers of blows were imparted for compacting each layer so that uniform density achieved throughout the depth. The curing time of 3 days were maintained for all specimens before test. The modulus of elasticity is usually calculated from straight portion of stress-strain curve. For most of the cases, however, the stress strain curve of the soil is nonlinear since onset of loading. So, the modulus of elasticity was calculated corresponding to the initial tangent of the stress-strain curve.

The values of Elastic modulus (E_i - value) of unstabilized and stabilized subgrade soil - A and B at a confining pressure of 70 kPa were 78 kg/cm^2 and 121 kg/cm^2 respectively. These values were increased to 102 kg/cm^2 and 147 kg/cm^2 respectively due to stabilization with 7.5 % lime content. The values of Elastic modulus (E_i - value) of unstabilized and stabilized subgrade soil - A and B at different confining pressure are presented in Table 4. The stress-strain curves were drawn for unstabilized and stabilized soil samples to study the effect of lime on failure stress. The unstabilized specimens of soil - A, and B at a confining pressure of 40 kPa attained a failure stress of 2.1 kg/cm^2 and 3.05 kg/cm^2 respectively, at an axial strain of 7.0 percent, and 6.5 percent respectively. The lime stabilized specimens exhibited highly ductile behavior. The value of failure stress of stabilized soils - A and B increased to 3.62 kg/cm^2 and 6.03 kg/cm^2 at an axial strain of 9 % and 8 % respectively. The variations in failure stress and failure strain with confining pressure are reported in Table 4.

TABLE 4 Effect of confining pressure on Elastic Modulus (E_i - value), Failure Stress (σ_f) and Failure Strain of Unstabilized and Stabilized Subgrade Soil – A and B.

Lateral pressure kg/cm^2	Unsterilized Soil - A			Stabilized Soil - A		
	E_i - value kg/cm^2	Stress at failure, σ_f kg/cm^2	Strain at failure %	E_i - value kg/cm^2	Stress at failure, σ_f kg/cm^2	Strain at failure %
0.40	56	2.1	7.0	76	3.62	9.0
0.70	78	2.9	8.5	102	4.34	9.0
1.2	96	3.43	9.0	122	5.22	9.5
1.5	108	4.54	11.0	141	6.34	10.0
	Unsterilized Soil - B			Stabilized Soil - B		
0.40	72	3.05	6.5	91	6.03	8.0
0.70	121	3.96	7.5	147	8.05	8.5
1.2	173	4.89	7.0	211	10.04	9.0
1.5	248	4.33	9.5	301	12.3	10.0

5. ELASTO-PLASTIC ANALYSIS

The finite element method was used to analyze the rural road pavement section resting on unsterilized and stabilized subgrade soils. The software **ANSYS** was used. The **ANSYS** element library contains more than 150 different elements and is capable of handling linear, nonlinear, static and dynamic 2-D and 3-D problems. In the present study, the section was modeled as a 2-D axisymetric problem with 8-noded structural solid element. A four - layer low volume rural road section was considered and analyzed. The thickness of each layer in the pavement section resting on unsterilized subgrade soil was designed as per Indian code of practice (IRC: SP:72-2007) [8] The thicknesses of each layer above the subgrade, initial tangent modulus of subgrade soil and other pavement layers required for analysis are reported in Table 5 (a) and (b). A pressure equal to single axle wheel load is assumed to be applied at the surface and distributed over a circular area of radius 150 mm. For application of FEM in the pavement analysis, the layered system of infinite extent is reduced to an approximate size with finite dimensions. Chiyyarath and Lymon adopted fixed right hand boundary at a distance 4a from axis of the symmetry where a is the radius of the loaded area. In the present study right hand boundary is provided at 110 cm from the outer edge of loaded area, which is more than 4 times loaded radius. This approximation will; however has little influence on the stress and strain distribution in finite element model ,(Desai and Abel, 1972)[12]

TABLE5 (a) Initial Tangent Modulus. of Subgrade for FE Analysis

Subgrade Soils	Subgrade Soil - A		Subgrade Soil - B	
	Unsterilized	Stabilized	Unsterilized	Stabilized
E-value (MPa)	7.56	10.36	11.18	14.68

Table 5 (b) Initial Tangent Modulus of Pavement Material for FE Analysis.

Pavement Layers	Subbase	Base	DBM
E-Value (MPa)	70.12	99.20	269.67

DBM – Dense Bituminous Macadam,
BC – Bituminous Concrete.

The elasto-plastic analysis was carried out to evaluate the primary response of the pavement resting on unsterilized and stabilized subgrade soils. The multilinear isotropic hardening model (MISO) available in **ANSYS** was used to evaluate the stresses, strains and deformations with in the pavement sections. The mixed incremental method is used in present study for elasto-plastic analysis of 2-D axisymetric finite element model. This method combines the advantages of both the incremental and the iterative schemes. The external load, here, is applied incrementally, but after each increment, successive iterations are performed to achieve equilibrium.

In general, for the j^{th} load increment, the state of deformation, stress and strain at the end of $(j-1)^{th}$ load increment is known, i.e. $\{\delta\}^{j-1}, \{\varepsilon\}^{j-1}, \{\sigma\}^{j-1}$ are known, the subscripts $(j-1)$ refers to the load increment. The general procedure of this method is follows,

i) For the first iteration of the j^{th} load increment,

$$\{\Delta F\}_1^j = [K^{j-1}]\{\Delta\delta\}_1^j \qquad (1)$$

which can be solved to obtain,

$$\{\Delta\delta\}_1^j = [K^{j-1}]^{-1}\{\Delta F\}_1^j \qquad (2a)$$

obtain, $\{\Delta\varepsilon\}_1^j = [B]\{\Delta\delta\}_1^j$ (2b)

and $\{\Delta\sigma\}_1^j = [D]\{\Delta\delta\}_1^j$ (2c)

ii) Accumulated displacements, strains and stresses at the end of 1^{th} iteration can be expressed as,

$$\{\delta\}_1^j = \{\delta\}^{j-1} + \{\Delta\delta\}_1^j \qquad (3a)$$

$$\{\varepsilon\}_1^j = \{\varepsilon\}^{j-1} + \{\Delta\varepsilon\}_1^j \qquad (3b)$$

$$\{\sigma\}_1^j = \{\sigma\}^{j-1} + \{\Delta\sigma\}_1^j \qquad (3c)$$

iii) Obtain the principal stresses, $\{\sigma_p\}_1^j$ and strains, $\{\varepsilon_p\}_1^j$ and then,

$$E_{t1}^j \text{ and } \nu_{t1}^j = f\left(\{\sigma_p\}_1^j, \{\varepsilon_p\}_1^j\right) \qquad (4a)$$

and $[D] = [D(E_{t1}^j, \nu_{t1}^j)]$ (4b)

iv) Equilibrated force vector will then be given by,

$$\{F_{eq}\}_1^j = \int_v [B]^T [D][B]\{\delta\}_1^j dv \qquad (5a)$$

Therefore, the residual force vector,

$$\{\psi\}_1^j = \left(\{F\}_1^j - \{F_{eq}\}_1^j\right) \qquad (5b)$$

v) Check for convergence,

$$\frac{\left[\{\psi_1^{\,j}\}^T\{\psi_1^{\,j}\}\right]^{0.5}}{\left[\{F_1^{\,j}\}^T\{F_1^{\,j}\}\right]^{0.5}} \times 100 \le \text{ToleranceLimit} \qquad (6)$$

In general, for any i^{th} iteration of the j^{th} load increment, force-displacement equation system will be -

$$\{\psi\}_{i-1}^j = [K^{j-1}]\{\Delta\delta\}_i^j \qquad (7a)$$

where $[K^{j-1}]$ is the constant stiffness matrix obtained from the state of stress and strain attained at the end of the $(j-1)^{th}$ load increment.

Therefore,

$$\{\Delta\delta\}_i^j = [K^{j-1}]^{-1}\{\Delta\psi\}_i^j \qquad (7b)$$

$$\{\Delta\varepsilon\}_i^j = [B]\{\Delta\delta\}_i^j \qquad (7c)$$

$$\{\Delta\sigma\}_i^j = [D]\{\Delta\sigma\}_i^j \qquad (7d)$$

The accumulated state of deformation, strain and stress is given by,

$$\{\delta\}_i^j = \{\delta\}^{j-1} + \{\Delta\delta\}_i^j \qquad (8a)$$

$$\{\varepsilon\}_i^j = \{\varepsilon\}^{j-1} + \{\Delta\varepsilon\}_i^j \qquad (8b)$$

$$\{\sigma\}_i^j = \{\sigma\}^{j-1} + \{\Delta\sigma\}_i^j \qquad (8c)$$

The state of principal stresses and strains will be given by $\{\sigma_p\}_i^j$ and $\{\varepsilon_p\}_i^j$ respectively, and the tangent modulii by E_{ti}^j, ν_{ti}^j and the elasticity matrix,

$$[D] = [D(E_{ti}^j, \nu_{ti}^j)] \qquad (9)$$

The equilibrated force vector,

$$\{F_{eq}\}_i^j = \int_v [B]^T [D][B] dv \qquad (10)$$

and the residual force vector,

$$\{\psi\}_i^j = \{\psi\}_{i-1}^j - \{F_{eq}\}_i^j \qquad (11)$$

The check for convergence will be given by,

$$\frac{\left[\{\psi_i^j\}^T\{\psi_i^j\}\right]^{0.5}}{\left[\{\psi_{i-1}^j\}^T\{\psi_{i-1}^j\}\right]^{0.5}}\times 100 \le \text{ToleranceLimit} \quad (12)$$

the equilibrium and therefore the convergence for j^{th} load increment is considered to have been achieved when this force residual is bellow certain tolerance level, otherwise iteration are continue until the above iteration satisfied. Once the convergence is achieved, the next increment ΔF_1^{j+1} is applied and the process is repeated until the final load level is reached. In this method, the equilibrium can be achieved at the end of every load increment. It makes use of a variable stiffness matrix for each new load increment while maintaining it constant within a given load increment so as to achieve convergence and therefore the equilibrium iteratively.

6. EVALUATION OF STABILIZATION BENEFITS

The Mechanistic – Empirical pavement design approach has been used in present study to evaluate the benefits of stabilizing subgrade soil in terms of reduction in layer thickness and extension in service life of the pavement. The present methodology has better capability to characterization of different material properties and loading conditions and has ability to evaluate different design alternatives on economic basis. The various design alternatives consider in present study are.

(1) Same service life of stabilized and unstabilized section, it would leads to reduction in layer thicknesses and has been expressed in terms of layer thickness reduction (LTR).
(2) Same pavement section for stabilized and unstabilized pavement section, it would result in more service life of pavement due to stabilization and has been expressed in terms of traffic benefits ratio (TBR).

The vertical compressive strain developed at top of unstabilized and stabilized subgrade soil - A and B was captured for different thicknesses of subbase and base for subgrade soil - A and B. the thickness of base 150 mm was maintained constant and subbase thickness was varied. Again, the subbase thickness 200 mm was maintained constant and base was varied. The vertical compressive strain developed at top of subgrade was captured for each of these alternatives. Fig. 3 and 4 shows the variation of vertical compressive strain with subbase thickness for constant base for pavement section resting on unsterilized and stabilized subgrade soil - A and B. whereas, Fig. 5 and 6 shows the variation of vertical compressive strain with base thickness for constant subbase for pavement section resting on unsterilized and stabilized subgrade soil - A and B. these plots further used for evaluating the stabilization benefits in terms of extension in service life and reduction in layer thicknesses.

Haas et.al[13], Webstar, [14], quantified the benefits of geogrid reinforcement in a pavement in terms of traffic benefits ratio (TBR). It gives the extension in service life of the pavement. The TBR can be written in the equation form as

$$TBR = \frac{N_R}{N_U} \quad (13)$$

N number of traffic passes required for producing a pavement surface deformation (rutting) upto the allowable rut depth, mm
R and U denote reinforced and unreinforced pavement section.

The structural failure in flexible pavements are of two types, fatigue failure, due to horizontal tensile strain at the bottom of bituminous layer and rutting failure, which is due to vertical compressive strain at top of subgrade. The rutting is considered as a failure criterion in the present study. IRC 37-2001 consider a rut depth of 20 mm with rutting equation given as

$$N_{20} = 4.1656*10^{-8}\left(\frac{1}{\varepsilon_V}\right)^{4.5337} \quad (14)$$

Where, N_{20} = Number of cumulative standard axle required to produce a rutting of 20 mm.

ε_v = Vertical compressive strain at top of subgrade

As there is no separate equation available in the literature to relate the vertical compressive strain at top of stabilized subgrade to the number of load repetition necessary to produce a allowable rutting. The equation 14 was used for both unstabilized and stabilized sub grade. Using equations 13 and 14 the benefits of stabilizing subgrade in terms of extension in service life of the pavements can be expressed as:

$$TBR = \frac{N_S}{N_{US}} = \left(\frac{\varepsilon_{VS}}{\varepsilon_{VUS}}\right)^{-B} \quad (15)$$

Where: N is the number of traffic passes required to produce an allowable rut depth in the pavement.

ε_v is vertical compressive strain at the top of subgrade that can be obtained through FEM. Symbols S and US denote stabilized and unstabilized pavement section. B is constant = 4.5337.

The result of elasto-plastic finite element analysis shows that for constant thickness of base equal to 150 mm, the vertical compressive strain developed at top of subgrade soil - A in pavement section designed for unstabilized subgrade is 4876.1 micron, the same strain level is obtain for subbase thickness of 165 mm in case of stabilized subgrade soil – A (Fig. 3). similarly, for a constant thickness of subbase equal to 200 mm, the vertical compressive strain developed at top of subgrade soil - A in pavement section designed for unstabilized subgrade is 4876.1 micron, the same strain level obtained for a base thickness of 120 mm (Fig.5). It indicates that if the same service life for both unstabilized and stabilized subgrade pavement is considered than the thickness of subbase is reduced by 35 mm and that of base reduced by 30 mm. similar types of results observed when pavement resting on stabilized subgrade – B.

If the pavement section is kept same for both unstabilized and stabilized subgrade the vertical compressive strain developed at the top of unstabilized subgrade – A is found to be equal to 4876.1 micron this value reduce to 3550 micron due to stabilization, it gives TBR of 4.22. It means that the stabilized pavement will have life which would be 4.22 times that of unstabilized pavements. Similar types of exercise can be done for other conditions also.

All these results have been summarized in

Table 6

TABLE 6 Stabilization Benefits in Subbase and Base of Low Volume Rural Road

Subgarde Soils	Constant Base Course			Constant Sub base Course		
	Subbase	$\frac{\varepsilon_{vunstb}}{\varepsilon_{vstb}}$	TBR	Base	$\frac{\varepsilon_{vunstb}}{\varepsilon_{vstb}}$	TBR
Soil –A	200	1.37	4.216	150	1.37	4.216
	175	1.05	1.25	125	1.04	1.18
	165	1.00	1.04	120	1.02	1.07
Soil –B	200	1.11	1.61	150	1.11	1.61
	175	1.02	1.06	125	1.00	1.00
	150	--	--	--	--	--

7.CONCLUSIONS

1. The four days soaked CBR value of subgrade soil increases considerably due to lime stabilization and it is the function of soil type and lime content. Also, it observed that the CBR value of subgrade soil increase rapidly up to 7.5 % lime content thereafter it start decreasing, hence optimum quantity of lime assumed to be 7.5 %. 2. The initial tangent modulus of subgrade soil - A and B at a confining pressure of 40 kPa is found to be 56 kg/cm^2 and 72 kg/cm^2 respectively it increase to 76 kg/cm^2 and 91 kg/cm^2 respectively due to stabilization with optimum percentage of lime.

3. If the pavement section is kept same for both unstabilized and stabilized subgrade the vertical compressive strain developed at the top of unstabilized subgrade – A is found to be equal to 4876.1 micron this value reduce to 3550 micron due to stabilization.

4. For constant thickness of base, the thickness of subbase is reduced by 17.5 % and 12.5 % for pavement section resting on stabilized subgrade A and B respectively.

5. For constant thickness of subbase, the thickness of base is reduced by 20 % and 16.67 % for pavement section resting on stabilized subgrade A and B respectively.

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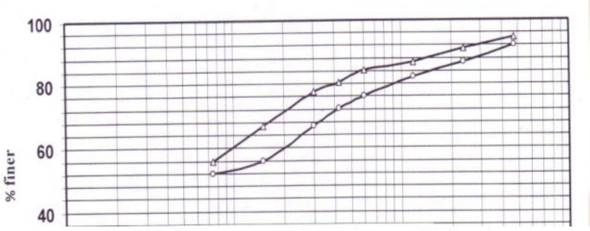

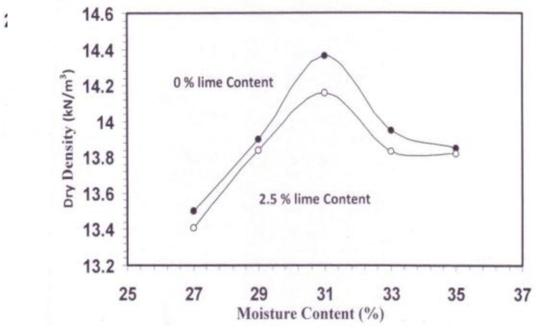

Figure 2 Variation of Dry Density with Moisture Content

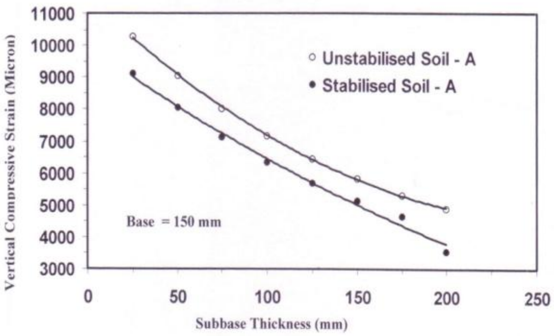

Figure 3. Variation of Vertical Compressive Strain at the top of Unstabilized and Stabilized Subgrade – A with Subbase Thickness for Constant base

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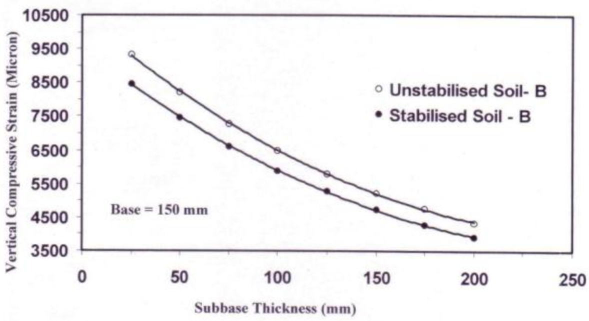

Figure 4 Variation of Vertical Compressive Strain at the top of Unstabilized and Stabilized Subgrade – B with Subbase Thickness for Constant base

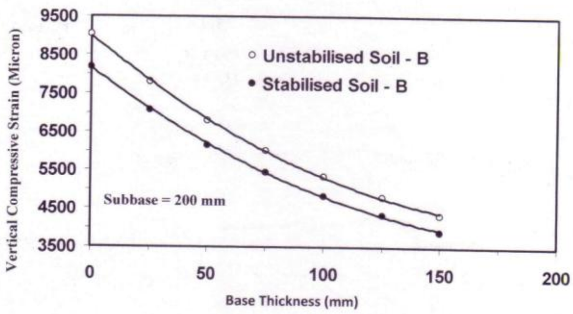

Figure 6. Variation of Vertical Compressive Strain at the top of Unstabilized and Stabilized Subgrade – B with Base Thickness for Constant base