GUIDELINES FOR SOIL AND GRANULAR MATERIAL STABILIZATION USING CEMENT, LIME & FLY ASH

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CONTENTS

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CHAPTER 1

INTRODUCTION

It was discussed in the first meeting 07.03.2009 of newly constituted "Embankment, Ground Improvement and Drainage Committee" (H-4) that following IRC guidelines which deals with soil stabilization need revision in the light of current practices and latest technological developments in the field. The identified guidelines were:

1) IRC:28-1967: "Tentative Specifications for the Construction of Stabilized Soil Roads with Soft Aggregates in areas of Moderate and High Rainfall"

2) IRC:33-1969: "Standard Procedure for Evaluation and Condition Survey of Stabilized Soil Roads"

3) IRC:49-1973: "Recommended Practice for the Pulverisation of Black Cotton Soils for Lime Stabilization"

4) IRC:50-1973: "Recommended Design Criterion for the use of Cement Modified Soils in Road Construction"

5) IRC:51-1992: "Guidelines for the use of Soil-Lime Mixes in Road Construction"

6) IRC:88-1984: "Recommended Practice for Lime-Fly Ash Stabilized Soils Base/Sub-base in Pavement Construction"

All these guidelines were reviewed and it was found that IRC-33, 49, 50, 51 and 88 are related to soil stabilization with admixtures. However, IRC-28 deals with soil and soft aggregates stabilization. The revised guidelines presented through this document encompass the review of soil stabilization which is the process whereby soils and related materials are made stronger and more durable by mixing with a stabilizing agent. Although many stabilizing agents can be used, cement and lime are by far the most important and the guidelines mainly concentrate on use of Lime, Cement, Lime-fly ash/Lime-cement fly ash as stabilizer. The guidelines include, general features of stabilization, guidelines for soil/granular material stabilization, specifications and test requirements for stabilized materials, construction procedure, quality control and limitations on the use of stabilized materials. These guidelines have been made considering prevailing Indian and International practices.

The draft of the guidelines was prepared by Shri Sudhir Mathur, Member-Secretarey, H-4 Committee and his collegues namely S/Shri R.K. Swami, Mrs Uma Arun and U.K. Guru Vittal. The guidelines were finalised by H-4 Committee under the Convenorship of Shri Mahesh Kumar, Engineer-in-Chief, Haryana, Public Works (Building and Roads) Department.

During the Seventh Meeting of Embankment, Ground Improvement & Drainage Comittee (H-4) (Personnel given below) held on 09.04.2010, the draft document was approved for circulation to the Highways Specifications & Standards Committee (HSS).

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The draft document was subsequently approved with some remarks by the Highways Specifications and Standards Committee in its meeting held on 01.05.2010. The draft document was approved by the Executive Committee in its meeting held on 10.05.2010. The Council in its meeting held at Munnar, Kerala on 22.05.2010 approved the document with some comments. The document after incorporating comments of Council Members was approved by the Convenor of Highways Specifications & Standards Committee for printing.

1.1 Purpose

These guidelines suggest the criteria for improving the engineering properties of soils and granular materials used for pavement base courses, sub-base courses and subgrades by the use of additives/stabilizers, which are mixed into the soil/granular materials to effect the desired improvement. A number of additives are available to improve the physical and engineering properties of these materials; however, this document restricts itself to stabilizers such as lime, cement, fly ash or a mixture of the above additives.

1.2 Scope

These guidelines prescribe the appropriate type or types of additives to be used with different soil types, procedures for determining a design treatment level for each type of additive and recommended construction practices for incorporating the additive into the soil. These criteria are applicable to all type of roads and airfields having a stabilized pavement layer.

1.3 Definitions

a) *Soils:* Naturally occurring materials that are used for the construction of all except the surface layers of pavements (i.e., concrete and asphalt) and that are subject to classification tests (IS 1498) to provide a general concept of their engineering characteristics.

b) *Additives:* Manufactured commercial products that, when added to the soil in the proper quantities, improve some engineering characteristics of the soil such as strength, texture, workability, and plasticity. Additives addressed in this manual are limited to cement, Lime and Fly ash.

c) *Stabilization:* Stabilization is the process of blending and mixing materials with a soil to improve certain properties of the soil. The process may include the blending of soils to achieve a desired gradation or the mixing of commercially available additives that may alter the gradation, texture or plasticity, or act as a binder for cementation of the soil.

d) *Mechanical Stabilization*: Mechanical stabilization is accomplished by mixing or blending soils of two or more gradations or mixing soil with aggregates to obtain a material meeting the required specification. The soil blending may take place at the construction site, a central plant, or a borrow area. The blended material is then spread and compacted to required densities by conventional means.

e) *Additive/Chemical Stabilization*: Additive stabilization is achieved by the addition of proper percentages of cement, lime, fly ash, or combinations of these materials to the soil. The selection of type and quantity or the percentage of additive to be used is dependent upon the soil classification and the degree of improvement in soil quality desired. Generally, smaller amounts of additives are required when it is simply desired to modify soil properties such as gradation, workability and plasticity. When it is desired to improve the strength and durability significantly, larger quantities of additive are used. After the additive has been mixed with the soil, spreading, sprinkling water and compaction at OMC are achieved by conventional means.

f) *Modification:* Modification refers to the stabilization process that results in improvement in some property of the soil but does not, by design, result in a significant increase in soil strength and durability.

1.4 Effectiveness of Stabilization

Pavement design is based on the premise that minimum specified structural strength will be achieved for each layer of material in the pavement system. Each layer must resist shearing, avoid excessive deflections that cause fatigue cracking within the layer or in overlying layers and prevent excessive permanent deformation through densification. As the quality of a soil layer is increased, the ability of that layer to distribute the load over a greater area is generally increased so that a reduction in the required thickness of the pavement layers may be permitted. Some of the attributes of soil modification/stabilization are indicated below.

a) *Quality improvement*: The most common improvements achieved through stabilization include better soil gradation, reduction of plasticity index or swelling potential and increase in durability and strength. In wet weather, stabilization may also be used to provide a working platform for construction operations. These types of soil quality improvement are referred to as soil modification. Stabilization can enhance the properties of road materials and give pavement layers the following attributes:

- A substantial proportion of their strength is retained even after they become saturated with water.
- Surface deflections are reduced.
- Resistance to erosion is increased.
- Materials in the supporting layer cannot contaminate the stabilized layer.
- The elastic moduli of granular layers constructed above stabilized layer are increased.
- Lime stabilized material is suitable for use as capping layer or working platform when the in-situ material is excessively wet or weak and removal is not economical.

b) *Thickness reduction:* The strength and stiffness of a soil layer can be improved through the use of additives to permit a reduction in design thickness of the stabilized material compared with an un-stabilized or unbound material.

c) *Possible problems:* The increase in the strength of pavement layers is also associated with the following possible problems:

- Traffic, thermal and shrinkage cracks can cause stabilized layers to crack.
- Cracks can reflect through the surfacing and allow water to enter the pavement structure.
- If carbon dioxide has access to the material, the stabilization reactions are reversible and the strength of the layers can decrease.
- The construction operations require more skills and control than for equivalent un-stabilized materials.

These issues have been further highlighted in Chapter 7.

CHAPTER 2

MECHANICAL STABILIZATION

2.1 Mechanical Stabilization

Mechanical stabilization is a process in which materials are proportioned to obtain desired gradation and plasticity of the mix. Correctly proportioned material (aggregate and soil) can be adequately compacted to form a mechanically stable pavement layer. This method is called mechanical stabilization. Thus the basic principles in this method of stabilization are : a) Proportioning and b) Compaction. If a granular soil containing negligible fines is mixed with a certain proportion of binder soil, it is possible to increase the stability. Similarly the stability of a fine grained soil can be considerably improved by mixing a suitable proportion of granular material to get a desired gradation.

Mechanical stabilization has been successfully applied for sub-base and base course construction. It has also been used as a surface course for low cost roads such as village roads when the traffic and rainfall are low. The desirable properties of soil aggregates mixtures are strength; incompressibility; fewer changes in volume and stability with variations in moisture content; good drainage; less frost susceptibility and ease of compaction. It is generally believed that the stability of a soil aggregate mix can be increased by increasing its dry density. Hence proportioning of mixes is done to attain maximum dry density.

The factors to be considered in the design of mix are gradation, density, index properties and stability. Of these, the gradation is the most important factor. The particle size distribution that gives maximum density is generally aimed at. Fuller's formula may be used to obtain the theoretical gradation for maximum density and is given by:

$$P = 100\ (d/D)^{1/2}$$

where

P = per cent finer than diameter 'd' (mm) in the material

D = diameter of the largest particle, mm

The following are the recommended values of the liquid limit and plastic index for the material passing 425 micron sieve, to be used for mechanical stabilization.

	Base Course	**Surface Course for Gravel Roads**
Liquid limit	25 per cent max.	35 per cent max.
Plasticity Index	6 per cent max.	5 to 10 per cent

2.2 Design of Mechanically Stabilized Mixes

When a few materials are available in the near vicinity of site, it is necessary to mix them in such a proportion, which would produce a mix with highest density. As an example if coarse aggregate, sand and fine soil are available from three deposits or borrow pits, it is first essential to decide the proportion of these component materials. The most commonly adopted graphical method for proportioning is the Rothfutch's method. Details of Rothfutch method are presented in Section **2.2.3**. The design based on combining two materials (soil and aggregates) on the basis of their sieve analysis to achieve specified gradation is given below:

- Column 2 and 6 in the Table give the particle size distribution of material A and B which do not satisfy the gradation requirement of the specification.
- Column 3 shows the standard sieve sizes, column 4 shows the recommended limits for a particular pavement course and column 5 shows the average value of corresponding limits shown in column 4.
- The inverse ratio of the totals in columns 1 and 7 gives the proportion of the materials to be mixed to obtain the desired mix.

  A : B = 45 : 139 (1 : 3).
- Mixing 25 percent of the material A and 75 percent of material B would give the desired gradation as shown in the Table.

Table 1 Mixing of Aggregates for Desired Gradation

Numerical Difference between material A and average percent passing	Material A percent passing	Sieve size	Recomm-ended Limits percent passing	Average percent passing	Material B percent passing	Numerical difference between material B and average percent passing
Col 1	Col 2	Col 3	Col 4	Col 5	Col 6	Col 7
-	100	40 mm	100	100	100	--
8	98	20 mm	80-100	90	73	17
26	94	10 mm	55-80	68	55	13
33	83	4.75 mm	40-60	50	42	8
32	72	2.36 mm	30-50	40	35	5
33	55	600 µm	15-30	22	21	1
7	17	75 µm	5-15	10	9	1
Total = 139						Total = 45

2.2.1 *Stabilization using soft aggregates (Mehra's method of stabilization)*

When hard variety of aggregates is not locally available, the local soft aggregates may have to be used for construction in order to keep the construction cost as low as possible. The soft aggregate have low crushing strength and low aggregate impact value. Still they have been adopted in the construction of mechanically stabilized sub-base, base course and even in wearing course layers. Commonly used soft aggregates for road construction are kankar, moorum, laterite and broken brick aggregates. Because of the low strength, these aggregates are likely to break down at their points of contact. If these aggregates are mixed with suitable proportion of soil so that each particle of soft aggregate is enveloped by soil, there would not be any problem of crushing of these aggregates during compaction or under traffic load.

Mehra's method of construction can be adopted for construction of low volume rural roads. In this method, base course material consists of compacted soil with sand content (of size less than 0.425 mm and greater than 0.075 mm) being not less than 50 percent and plasticity index 5 to 7. Wearing course material consists of brick aggregates and soil mixed in the ratio of 1:2. The sand content in the soil should be less than 33 percent and plasticity index 9 to 12. However, when bituminous surface treatment is required/desired, the plasticity index should be limited to 8 to 10. This method, proposed by Prof.S.R.Mehra, is briefly given below:

1) Soil is collected from approved borrow pits and stacked on roadside.
2) Water is added upto OMC and soil is mixed and spread to desired camber and grade.
3) 11.5 cm thick loose base course material (sandy soil) is spread and rolled by 8 tonnes roller to a compacted thickness of 7.5 cm.
4) Surface course material (brick aggregate and soil in the ratio 1:2) mixed with adequate water is spread to 11.5 cm loose thickness and the layer is rolled by 8 tonnes roller to a compacted thickness of 7.5 cm.
5) After rolling, the surface is watered and left overnight. The surface is again rolled and finished.
6) The road is closed to traffic for 4-5 days and kept sprinkled with water. For next few days, only rubber-tyred traffic is allowed and after about 2 weeks, the road is opened to all traffic. Mehra's method of construction can carry 50 tonnes of traffic per day in places of light rainfall. With bituminous surfacing, the road gives satisfactory service upto 200 tonnes per day even in places with heavy rainfall.

2.2.2 *Design of mechanically stabilized mixes: combining two materials based on plasticity*

Let there be two soils A and B which are to be mixed to get a soil of required plasticity index P.

Step-1 Determine the plasticity index of the two soils. Let these be P_A and P_B for soil A and Soil B respectively.

Step-2 Determine from sieve analysis for each soil, the percentage of material passing 425 micron sieve. Let these be S_A and S_B for the Soil A and Soil B respectively. Then the percentage of Soil A to be mixed with Soil B to get the desired plasticity index i.e., P, is given by the relation:

$$\text{Material A \%} = \frac{S_{B.}(P - P_B)}{S_{B.}(P - P_B) - S_A(P - P_A)}$$

2.2.3 *Rothfutch method for design of soil-aggregate mixes:*

Rothfutch method is adopted when a number of materials are to be mixed together to obtain a combined material conforming to a desired gradation. It is to be noted that none of these individual constituents of combined material would be able to satisfy the desired gradation. The ratio of mixing these individual constituents is determined based on methodology proposed by Rothfutch. In this process, the first step would be to determine the desired gradation. This may be based on the specification limits or as per theoretical equation given by Fuller or other researchers. The procedure involves drawing the gradation curves on graph paper and then finding out optimum mix proportion as described below:

- On a graph sheet percent passing is marked on Y axis in a suitable linear scale. X axis represents the particle size distribution, which is to be marked, based on desired gradation.
- The corner O and O' are joined by straight line.
- OO' represents required gradation line.
- Sieve sizes are marked corresponding to percent passing of required gradation. This can be done by locating the average percentage specified for any particular sieve, locating that point on the Y axis and then proceeding to cut the line OO'. At the point of intersection of this horizontal line and OO', a vertical line is drawn to cut X axis. This intersection point on X axis represents the sieve size selected for the desired gradation. In this manner, all the sieve sizes are marked one by one on the X axis.
- Gradation distribution of materials A, B and C are then drawn, by using the sieve sizes marked on the X axis and the percentage finer marked on Y axis.
- Balancing line for materials A, B and C are drawn in such a way that area on either side of balancing line and gradation curve are equal. Balancing lines are straight lines which represent the particle size distribution of respective material in a best possible manner. This can be accomplished by using a transparent plastic scale, moving it on either side of the material gradation curve and counting the number of squares enclosed between balancing line and material gradation curve. The balancing line should be drawn in such a manner that number of squares (area) on either side of balancing line and the selected gradation curve should be equal.

- The opposite ends of the two adjoining balancing lines are then joined.
- The points where these joining lines cut OO' represent the percentage of that material in the mix.

The method is illustrated in **Fig. 1** below:

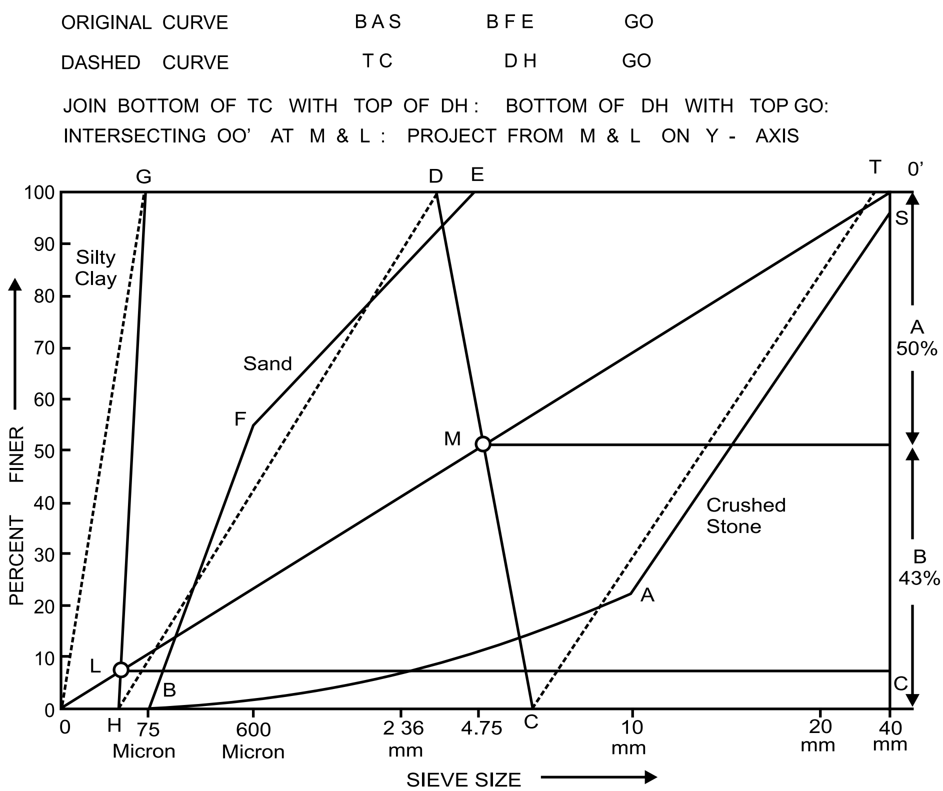

Fig. 1 Rothfutch Method of Designing Soil-Aggregate Mixes

CHAPTER 3

GENERAL GUIDELINES FOR SOIL/GRANULAR MATERIALS STABILIZATION

3.1 Factors to be Considered

In the selection of a stabilizer, the factors that must be considered are the type of soil to be stabilized, the purpose for which the stabilized layer is used, the soil improvement desired, the required strength and durability of the stabilized layer and the cost and environmental conditions. The following parameters are required to be considered while selecting the type of stabilizer.

a) *Soil types and additives:* There may be more than one candidate stabilizer applicable for particular soil type. However, there are some general guidelines that make specific stabilizers more effective based on soil granularity, plasticity, or texture. Portland cement for example can be used with a variety of soil types; however, since it is imperative that the cement be mixed intimately with the fines fraction (less than 0.075 mm size), the more plastic materials should be avoided. Generally, well-graded granular materials that possess sufficient fines to produce a floating aggregate matrix (homogenous mixture) are best suited for cement stabilization. Lime will react with soils of medium to high plasticity to produce decreased plasticity, increased workability, reduced swell and increased strength. Lime is used to stabilize a variety of materials including weak subgrade soils, transforming them into a "working table" or sub-base; and with marginal granular base materials, i.e., clay-gravels, "dirty" gravels, to form a strong, high quality base course. Fly ash is a pozzolanic material, i.e. it reacts with lime in powdered form in presence of water and is, therefore, almost always used in combination with lime in soils that have little or no plastic fines. It has often been found desirable to use a small amount of Portland cement with lime and fly ash for added strength. This combination of Lime-Cement-Fly Ash (LCF) has been used successfully in sub-base course stabilization.

b) *General Guidelines*: The following are general guidelines when considering stabilization with different additives.

3.1.1 *Lime stabilization*

- Clayey soils including heavy clays, moorum and other soils met within alluvial plains can be effectively treated with lime. For effective stabilization, a soil must have a fraction passing 425 micron sieve not less than 15 percent and Plasticity Index (PI) should be at least 10 percent.
- For effective stabilization, it is desirable that the percentage retained on 425 micron sieve should be well graded with uniformity coefficient not less than 5.

- Organic matter in the soil selected for soil stabilization should not be more than 2 percent and sulphate content should not exceed 0.2 percent.
- pH value of 10 or 11 is desired for pozzolanic reaction to take place between clay minerals and lime for the formation of cementitious compounds.
- Soils having organic matter and soluble carbonate/sulphate contents in excess of 2.0 percent and 0.2 percent respectively require special studies.
- Some materials contain amorphous silica which although has low plasticity but reacts with lime to form the necessary cementation products and should thus be considered for stabilization with lime.
- Materials containing high Kaolinite as the basic clay mineral usually have a fairly low PI with a high liquid limit and in such cases lime should be considered for stabilization.
- In case of highly plastic soils, two stage stabilization is adopted. In this case soil is first treated with a small quantity of lime. Later on the soil may be treated with remaining quantity of lime or with cement to achieve the desired strength and stability.

3.1.2 *Cement stabilization*

- Generally granular soils free of high concentration of organic matter or deleterious salts are suitable for cement stabilization. For checking the suitability of soils, it would be advantageous to keep the following criterion in view:

  a) Plasticity Product (PP), expressed as product of PI of soil and percentage fraction passing 75 micron sieve should not exceed 60.

  b) Uniformity coefficient of soil should be greater than 5 and preferably greater than 10.

  c) Highly micaceous soils are not suitable for cement stabilization.

  d) Soils that are having organic content higher than 2 percent and also those soils having sulphate and carbonate concentration greater than 0.2 percent are not suitable for cement stabilization.

  e) Silty or fine sandy materials may exhibit a high liquid limit because of the high surface area of the particles. This material generally will not react with lime because of lack of clay particles and can be stabilized with cement. However, cement stabilization with high doses of cement may tend to make stabilization uneconomical.

3.1.3 *Lime-fly ash (LF) and lime-cement-fly ash (LCF) stabilization*

Stabilization of coarse-grained soils having little or no fines can often be accomplished by the use of LF or LCF combinations. Fly ash, also termed coal ash, is a mineral residual from the combustion of pulverised coal. It contains reactive silica and aluminium compounds that, when mixed with lime and water, form a hardened cementitious mass capable of obtaining high compressive strengths. Lime and fly ash in combination can often be used successfully in stabilizing granular materials since the fly ash acts as an agent, with which the lime can react. Thus LF or LCF stabilization is often appropriate for base and sub-base course materials.

3.2 Desirable Properties of Lime, Cement and Fly Ash for Stabilization

3.2 (a) *Lime:* Lime is a broad term which is used to describe calcium oxide (CaO) - Quick lime; calcium hydroxide Ca $(OH)_2$ - Slaked or hydrated lime and calcium carbonate $CaCO_3$ - carbonate of lime. The relation between these three types of lime can be represented by the following equations:

1) $CaCO_3 + \text{heat} = CaO + CO_2$

2) $CaO + H_2O = Ca(OH)_2 + \text{heat}$

3) $Ca(OH)_2 + CO_2 = CaCO_3 + H_2O$

The first reaction which is reversible does not occur much below 500ºC and is the basis for the manufacture of quicklime from chalk or limestone. Hydrated lime is produced as a result of the reaction of quicklime with water (equation 2). Quicklime (by the reversal of equation 1) and hydrated lime (by equation 3) will both revert to calcium carbonate which is not used for stabilization, although it is used in agriculture as a soil additive to adjust pH.

In dolomitic lime, some of the calcium is substituted by magnesium. These types of limes can also be used for stabilization. Hydraulic lime also known as grey lime is produced from impure forms of calcium carbonate which also contain clay. They, therefore, contain less "available lime" to initiate effects on plasticity and strength. However, to compensate for this, they contain reactive silicates and aluminates similar to those found in Portland cement. Thus, whilst their immediate effect may be less than that of high calcium limes in the long term they may develop higher strengths. Generally, use of dolomitic lime is not considered suitable.

Consequently in the context of this guidelines, lime stabilization refers to the addition of calcitic dry lime, commercially available, slaked at site, or pre-slaked lime delivered at site in suitable packing. Hydrated/Slaked lime comes in the form of a fine dry powder.

Quicklime is available either in granular form or as a powder. It reacts violently with 32 percent of its own weight of water to produce considerable amounts of heat (approx 17×10^9 Joules per kg of quicklime are released).

Hydrated lime and quicklime are both usually added to soil in the solid form but they may also be mixed with water and added to the soil as slurry. The advantages and disadvantages of the three methods of application are summarised below:

- Dry hydrated lime

  a) *Advantages:* Can be applied two to three times faster than slurry and is very effective in drying out soils.

b) *Disadvantages:* Produces a dust problem that makes it undesirable for use in urban areas and the fast drying action of the lime requires an excess amount of water during hot, dry weather.

- Quicklime

  a) *Advantages:* More economical than hydrated lime as it contains approximately 25 percent more available lime. Faster drying action than hydrated lime on wet soils.

  b) *Disadvantages:* Field hydration is less effective, producing a coarser material with poorer distribution in soil mass; quicklime requires more water than does hydrated lime for stabilization, which may present a problem in dry area and greater vulnerability of site personnel to skin and eye burns.

- Slurry lime

  a) *Advantages:* Dust free application is more desirable from an environmental standpoint; better distribution is achieved with the slurry; spreading and sprinkling operation are combined, thus reducing processing costs and during hot and dry weather slurry application pre-wets the soil and minimizes drying action.

  b) *Disadvantages:* Application rates are slower. High capacity pumps are required to achieve acceptable application rates; extra equipment is required and cost is therefore higher; extra manipulation may be required for drying purposes during cool, wet, humid weather; not practical for use with very wet soils and if prepared from quicklime any benefits arising from the heat of hydration of quicklime are largely lost.

The purity of lime affects the strength of lime-soil stabilization. The effectiveness of lime in reaction with its clay minerals is dependent to a good extent on its chemical composition i.e., amount of calcium oxide present in the lime. The purity of lime is expressed as the percentage of available calcium oxide present in the lime. It is generally recommended that the lime used for soil stabilization should have purity of 50 percent or above. The addition of lime should be correspondingly increased whenever the field tests show a lesser purity. Calcium oxide content in lime should be determined as specified in IS 1514 or IS 712. Slaked lime supplied in airtight bags should not be stored for more than three months. Since lime deteriorates with storage the purity must be checked at site before use.

For effective stabilization with lime, uniform mixing is a pre-requisite and the degree of mixing depends on the fineness of lime. When using fine powdered lime, there will be a quick and effective reaction with clay minerals to form cementitious compounds. Lime for stabilization shall conform to the fineness requirement of class C hydrated lime as specified in IS 1514 or IS 712, which is as under **(Table 2):**

Table 2 Requirement of Fineness for Lime Stabilization

S.No	Sieve Size (Micron)	Percentage Passing
1)	850	100
2)	300	99 (Minimum)
3)	212	95 (Minimum)

3.2 (b) *Cement:* Cement for cement stabilization should comply with the requirements of IS 269, 455 or 1489.

3.2 (c) *Fly ash:* Fly ash may be from anthracitic coal or lignitic coal. Fly ash to be used for the purpose of soil-lime-fly ash stabilization should conform to the requirements given in **Table 3 and 4**.

Table 3 Chemical Requirements for Fly Ash as a Pozzolana

Sl. No.	Characteristics	Requirements for Fly Ash		Method of Test
		Anthracitic fly ash	Lignitic fly ash	
1)	$SiO_2 + Al_2O_{3+}$ Fe_2O_3 in percent by mass, Min	70	50	IS 1727
2)	SiO_2 in percent by mass, Min	35	25	IS 1727
3)	MgO in percent by mass, Max	25	5.0	IS 1727
4)	SO_3 in percent by mass, Max	2.75	3.5	IS 1727
5)	Available alkalies as Na_2O/ K_2O in per cent by mass, Max	1.5	1.5	IS 4032
6)	Total chlorides in percent by mass, Max	0.05	0.05	IS 1727
7)	Loss on ignition in percent by mass, Max	5.0	5.0	IS 1727

Table 4 Physical Requirement for Fly Ash as a Pozzolona

Sl. No.	Characteristics	Requirement
1)	Fineness-specific surface in m^2/kg by Blaine's permeability test, Min	250
2)	Particles retained on 45 micron IS sieve, Max	40
3)	Lime reactivity in N/mm^2, Min	3.5
4)	Soundness by autoclave test expansion of specimen in per cent, Max	0.8
5)	Soundness by Lechatelier method-expansion in mm, Max	10

3.3 Selection of Stabilizer

The selection of the stabilizer is based on plasticity and particle size distribution of the material to be treated. The appropriate stabilizer can be selected according to the criterion shown in **Table 5**. Some control over the grading can be achieved by limiting the coefficient of uniformity to a minimum value of 5; however, it should preferably be more than 10. If the coefficient of uniformity lies below 5, the cost of stabilization will be high and the maintenance of cracks in the finished road would be expensive. If the plasticity of soil is high there are usually sufficient clay minerals which can be readily stabilized with lime. Cement is more difficult to mix intimately with plastic material but this problem can be alleviated by pre-treating the soil with approximately 2 percent lime.

Table 5 Guide to the Type of Stabilization likely to be Effective

Type of Stabilization	Soil Properties					
	More than 25% passing the 0.075 mm sieve			Less than 25% passing the 0.075 mm sieve		
	PI < 10	10 <PI< 20	PI > 20	PI < 6, PP < 60	PI < 10	PI > 10
Cement	Yes	Yes	*	Yes	Yes	Yes
Lime	-	Yes	Yes	No	*	Yes
Lime-Pozzolana	Yes	-	No	Yes	Yes	*

3.4 Two Stage Stabilization Using Lime followed by Cement

Cement can be used to stabilize most of the soils. The principal exceptions are those that contain organic matter in a form which retards the hydration of cement and soils which are difficult to mix with cement on account of their high clay content. In comparison with cement, the potential use for lime in soil stabilization is more restricted; used in equivalent amount it generally produces lower strengths than does cement and its main application is for use with clayey soils which are difficult to stabilize with cement. For these reasons the use of a two stage lime/cement stabilization process appears attractive as it offers the possibility of extending the range of soil which can be effectively stabilized. To achieve the maximum effect the lime and cement would not be blended but would be added separately. Lime would first be added to modify the properties of soil and this would be followed by the addition of cement to bring out a long term increase in strength, when lime treated soil is stabilized with cement.

3.5 Modification and Cementation

There is a distinction between a stabilized soil "modified" for subgrade improvement and "cemented" (a cemented soil in this context can be one stabilized with lime as the clay-lime pozzolanic reaction products can be regarded as a cement) for use as a sub-base or base. The term "modification" and cementation are used in specifications to describe the degree and type of treatment.

The rapid action of lime on soil, which brings about a reduction in plasticity and a marginal increase in CBR is referred to as Modification. If conditions are favourable for the pozzolanic action to proceed, the lime stabilized soil will develop significant compressive and tensile strengths and it is then regarded as a "cemented" material.

If a very small quantity of cement is added to a soil, the properties may also be modified without much hardening or the development of significant compressive or tensile strength. In such cases the degree of cementation is relatively poor, but the properties of a material can nevertheless be considerably improved in this way. This treatment is also referred to as modification in the specifications.

When a material has developed sufficient tensile strength, it is regarded as a cemented material but there is no clearly defined boundary between modification and the division between the two is arbitrary. However, it has been suggested (NAASRA 1986) that a 7 day unconfined compressive strength of 0.8 N/mm^2 could be set as the boundary between the two.

CHAPTER 4

SPECIFICATIONS AND TEST REQUIREMENTS FOR STABILIZED MATERIALS

4.1 General Requirement

The pavement performance of a stabilized road will be largely governed by the gradation and the type of soil/granular material used for the purpose of stabilization. The quality of material to be stabilized should meet the minimum standard set out in specifications. Stabilized layers constructed from such material is likely to perform satisfactorily even if, it is affected by carbonation during its life time. Materials which do not comply with the requirements given in the specifications can be stabilized but more additive will be required and the risk from cracking and carbonation will increase. The strength of stabilized materials can be evaluated in many ways, of which the most popular are the Unconfined Compressive Strength (UCS) test and the California Bearing Ratio (CBR) test.

4.2 Stabilization with Cement

4.2.1 *Requirement for soil modification/subgrade improvement*

Cement stabilized materials can be used for soil modification or improvement of subgrade soil. It is recommended from economic consideration that mix in-place methods of construction be used for subgrade improvement and only granular materials and silty cohesive materials be used. (The assumption being that more clayey materials would be more effectively stabilized with lime). The main requirements for cement modification or stabilization of subgrade soil are summarised in **Table 6**.

Table 6 Soil Characteristics for Cement Modified Soil/Improved Subgrade/Capping Layer

Properties	Specified Value
Liquid Limit (%)	<45
Plasticity Index	<20
Organic content (%)	<2
Total SO_4 content	0.2 % Max
Minimum Laboratory CBR at specified density (%)	15
Minimum cement content (%)	2*
Degree of pulverisation (%)	>60
Temperature for mixing	More than 10ºC
Time for completing compaction	2 hrs Max

* In case better mechanical equipment for spreading of cement, for breaking clods and blending is used, the minimum percentage of cement for stabilization could be 0.5 percent. However extensive lab testing must be done to arrive at this minimum percentage. Sample at site of blended loose soil be collected and remoulded in lab to confirm that the desired CBR can be achieved.

4.2.2 *Requirement for bound sub-bases/bases*

Granular materials, gravel, sand, lateritic soils, sandy silty material, crushed slag, crushed concrete, brick metal and kankar, etc. stabilized with either cement or lime-fly ash-cement or lime-fly ash, etc. may be allowed for use as capping layer over weak subgrade, as sub-base and base layer of pavement. The main requirements of stabilized layers for different layers of a pavement structure as indicated above are summarised in **Table 7 and 8**. The gradations indicated in **Table 8** are intended as tentative specifications. Gradation for cement bound materials as per MoRTH specifications can also be adopted. However, thickness of different stabilized layers, selection/choice for adoption of a particular grading and strength requirements of these layers are to be decided on the basis of pavement design and with specific approval of the Engineer-in-Charge.

Table 7 Material Characteristics for Cement Modified Granular Materials

Properties	Specified Value
Liquid Limit (%)	<45
Plasticity Index	<20
Organic content (%)	<2
Total SO_4 content (%)	0.2
Water absorption of coarse aggregates	<2% (If the is value is >2% the soundness test shall be carried out on the materials delivered to site as per IS 383)
10 percent fines value when tested as per BS 812(III)	≥ 50 kN

4.3 Stabilization with Lime

4.3.1 *Requirement for soil modification/subgrade improvement*

The properties of soil-lime mixes is usually assessed on the basis of strength tests made on the materials after the stabilizer has been allowed sufficient time to harden. The strength of stabilized soils can be evaluated in many ways, of which the most popular is California Bearing Ratio (CBR) test for lime stabilized soils. Lime stabilization is generally recommended to improve the subgrade soils which are cohesive in nature. Lime is recommended for such soils because of its beneficial effects on plasticity, workability and strength gain. The main requirements for lime stabilized improved subgrade are summarised in the **Table 9**.

Table 8 Gradation Requirement for Cement Bound Materials for Base/Sub-bases/Capping Layer

Sieve size	Grading I	Grading II	Grading III	Grading IV
75.0 mm		100		100
53.0 mm	100	80-100	100	--
45.0 mm	95-100			
37.5 mm			--	95-100
26.5 mm		55-90	70-100	55-75
22.4 mm	60-80			
11.2 mm	40-60			
9.5 mm		35-65	50-80	
4.75 mm	25-40	25-55	40-65	10-30
2.36 mm	15-30	20-40	30-50	-
0.600 µ		-	-	
0.425 µ	8-22	10-35	15-25	-
0.300 µ		-		
0.075 µ	0-8	3-10	3-10	0-10
7 days Unconfined Compressive Strength (MPa) for cement bound materials or 28 days strength for lime-fly ash & lime-cement-fly ash bound materials	12*/6**	7*/4.5**	3*/1.5**	1.5*/0.75**

* Average value of a batch of 5 cubes

** Minimum strength of an individual cube within the batch. For Grading IV the unconfined compressive strength and CBR requirement are equally acceptable alternatives.

Table 9 Material Characteristics for Lime/Modified Soils

Properties	Specified Value
Passing 75 mm sieve	100%
Passing 26.5 mm sieve	95 - 100%
Passing 75 micron sieve	15 - 100%
Plasticity Index	$\geq$ 10
Organic content	< 2%
Total SO_4 content	< 0.2%
Minimum lime content	2.5%*
Degree of pulverisation	> 60%
Minimum temp for mixing at field	10ºC
UCS (MPa)	As per Contract Specifications

* In case better mechanical equipment for spreading of lime, for breaking clods and blending is used, the minimum percentage of lime for stabilization could be 0.5 percent. However extensive lab testing must be done to arrive at this minimum percentage. Sample at site of blended loose soil be collected and remoulded in lab to confirm that the desired CBR can be achieved.

The quality of lime shall be the same as given in Section **3.2**.

4.4 Stabilization with Lime Fly Ash (LF)

Pulverised fuel ash (PFA) or fly ash has been recognised for many years as a valuable material for modifying and enhancing the properties of soils. Stabilization of coarse grained soils having little or no fines can be accomplished by the use of LF or LCF Combination. Fly ash also termed as coal ash is a mineral residual obtained from the combustion of the pulverised coal. It contains silicon and aluminium compound which, after mixing with lime and water forms a hardened cementitious mass capable of obtaining high compressive strengths. Lime and fly ash in combination can often be used successfully in stabilizing granular materials since the fly ash provides an agent with which the lime can react. Thus LF or LCF stabilization is often appropriate for base and sub-base course materials. Fly ash may be either from anthracitic coal or lignitic coal. Fly ash to be used in lime and fly ash stabilization shall conform to the requirements given in **Table 3 and 4**. Lime shall conform to the requirement as given in Section **3.2**.

Design of lime-fly ash stabilized mix is somewhat different from stabilization with lime or cement. For a given combination of materials (aggregate, fly ash and lime) a number of factors can be varied in the mix design process such as percentage of lime-fly ash, moisture content and the ratio of lime-fly ash. It is generally recognised that engineering characteristics such as strength and durability are directly related to the quality of the matrix material. The matrix material is that part, which consists of fly ash, lime and passing 10 mm aggregates fines. Basically, higher strength and improved durability is achieved, when the matrix material is able to fill the coarse aggregate particles. For each coarse aggregate material, there is a quantity of matrix required to effectively fill the available void spaces. The quantity of matrix required for maximum dry density of the total mixture is referred to as the optimum fines content. In LF mixtures it is recommended that the quantity of matrix be approximately 2 percent above the optimum fines content. At the recommended fines content, the strength development is also influenced by the ratio of lime to fly ash. Adjustment of the lime-fly ash ratio will yield different values of strength and durability properties. The mix design process is described below:

Step 1: The first step is to determine the optimum fines content that will give the maximum density. This is done by conducting a series of moisture-density tests using different percentages of fly ash and determining the mix level that yields maximum density. The initial fly ash content should be about 10 percent based on dry weight of the mix. It is recommended that material larger than 20 mm be removed and the test conducted on the minus 20 mm fraction. Tests are run at increasing increments of fly ash, e.g. 2 percent, upto a total of about 20 percent. Moisture density tests should be conducted following procedures indicated in IS 2720, Part 7 or Part 8. The design fly ash content is then selected at 2 percent above that yielding maximum density.

Step 2: Determine the ratio of lime to fly ash that will yield highest strength and durability. Using the design fly ash content and the optimum water content determined in Step 1, prepare triplicate specimen at three different lime fly ash ratio. Use LF ratios of 1:3, 1:4 and 1:5. If desired about 1 per cent of Portland cement may be added at this time.

Step 3: Conduct durability test as per ASTM D 559 and compare the results of the unconfined compressive strength and durability tests with the requirements shown in **Table 8**. The lowest LF ratio content, i.e., ratio with the lowest lime content which meets the required unconfined compressive strength requirement and demonstrates the required durability, is the design LF content. If the mixture meet the durability requirements but not the strength requirements, it is considered to be a modified soil. If the results of the specimens tested do not meet both the strength and durability requirements, a different LF content may be selected or additional portland cement be used and Steps 2 to 4 repeated to ascertain strength and durability requirements as per stipulated specifications.

Gradation Requirements for LF or LCF stabilization for stabilized sub-base /bases may be as indicated in **Table 8**.

4.5 Stabilization with Lime, Cement and Fly Ash

Portland cement may also be used in combination with LF for improved strength and durability. If it is desired to incorporate cement into the mixture, the same procedures indicated for LF design should be followed except that, beginning at Step 2, the cement shall be included. Generally, about 1 to 2 percent cement is used. Cement may be used in place of or in addition to lime, however, the total fines content should be maintained. Strength and durability tests must be conducted on samples at various LCF ratios to determine the combination that gives best results. Gradation requirements for LF or LCF stabilization for stabilized sub-base/bases should be as indicated in **Table 8**.

4.6 Cement Stabilized Fly Ash

This work shall consist of laying and compacting a sub- base/base course of fly ash as treated with cement on prepared subgrade/sub-base, in accordance with requirements of the specifications. This technique can be adopted for improvement of poor subgrade also. Fly Ash to be used for cement fly ash stabilization shall conform to **Table 3 and 4**. Pond ash or bottom ash, which do not meet the requirements of **Table 3 and 4** can also be used for cement stabilization work. However, in all cases the cement stabilized fly ash/bottom ash/pond ash mix should develop adequate strength.

The objectives of the mix design procedures, is to provide a pavement material having the required proportions of fly ash and cement to meet the following requirements:

1) Provide adequate strength and durability

2) Be easily placed and compacted

3) Be economical

Amount of cement less than 2 percent is not generally amenable to proper mixing and hence not recommended. After deciding cement and fly ash content for trial mix moisture density relationship has to be determined in accordance with IS 2720 (Part-7 or 8). The unconfined compressive strength test is done on samples compacted at maximum dry density and optimum moisture content. The mix proportion should be designed to obtain minimum unconfined compressive strength of 17.5 kg/cm^2 after 7 days moist curing in a humidity chamber for samples with a length to diameter ratio of 2:1. Curing may be carried out in the temperature range 30ºC to 38ºC. The design mix should not only indicate the proportions of fly ash and cement, but also mention quantity of water to be mixed and a specified compacted density that is required to satisfy specified strength.

Cement: Cement conforming to IS 269 or IS 8112 can be used. Portland pozzolona cement should not be used for stabilization, when fly ash is used as an ingredient.

Water: Water used for mixing and curing for stabilized mixes shall be clean and free from injurious amounts of oils, salt and acid etc. It shall meet the requirement as IS 456. Potable water is generally considered to be acceptable for stabilization works. The permissible limits for solids in water should be as given in **Table 10**.

Table 10 Permissible Limit for Solid in Water for Soil Stabilization

Solids	Permissible Limit (Maximum)
Organic	200 mg/litre
Inorganic	3000 mg/litre
Sulphates (as SO_4)	400 mg/litre
Chloride (as Cl)	2000 mg/litre
Suspended matter	2000 mg/litre

4.7 Test Requirements

4.7.1 *Unconfined compressive strength test*

This test is carried out on cylindrical or cubical specimens prepared by mixing the soil at a pre-determined moisture content and stabilizer content and compacting the mixed material into a mould at either a pre-determined density or at a given compactive effort. The choice of specimen size and shape depends on the grading of the soil; it is clearly desirable to keep as small as possible the ratio of the maximum particles size to the smallest dimension of the mould. The following sizes of specimen for different group of material are recommended **(Table 11)**.

Table 11 Suggested Size of Moulds for Casting Materials Samples

Fine grained material	Cylindrical specimens 100 mm high and 50 mm diameter, or 150 mm cubic specimens
Medium grained materia	Cylindrical specimens 100 mm high and 50 mm diameter, or 150 mm cubic specimens
Coarse-grained material	150 mm cubic specimens

Compressive strength results on identical materials from strength tests on cubical specimens would be higher than those obtained from cylindrical specimens; and cylindrical specimens with a height/diameter ratio of 2:1; have lower strength than cylindrical specimens with a height/ diameter ratio of 1:1. Allowance therefore has to be made for this when comparing results obtained with specimens of different shapes. For the relatively low strengths encountered in cement-stabilized soils the results on different sized test specimens may be multiplied by the correction factors given in **Table 12** to calculate the approximate equivalent strength of a 150 mm cube. However, there is no unique relation between the strengths of specimens of the two shapes as the ratio depends primarily on the level of strength of the material.

Table 12 Correction Factors for Various Size and Shape of Test Specimens

S.No.	Specimen Size	Correction factor
1)	150 mm cube	1.00
2)	100 mm cube	0.96
3)	200 mm × 100 mm diameter cylinder	1.25
4)	115.5 mm × 105 mm diameter cylinder	1.04
5)	127 mm × 152 mm diameter cylinder	0.96

4.7.2 *Durability of stabilized materials*

In order to check the durability of the stabilized mix for sub-base/base, the following two methods are recommended. Method 1 is recommended for moderate temperature and climatic conditions, whereas method 2 is recommended for those regions where there is large variation in temperature and climatic conditions. The decision regarding the adoption of a particular method should be as directed by the Engineer-in-Charge.

Method 1: Prepare two identical set (containing 3 specimens each) of UCS specimen which are cured in a normal manner at constant moisture content for 7 days. At the end of 7 days period one set is immersed in water while the other set is continued to cure at constant moisture content. When both sets are 14 days old they are tested for UCS. The strength of the set immersed in water as a percentage of the strength of set cured at constant moisture content is calculated. This index is a measure of the resistance to the effect of water on strength. If this value is lower than 80 percent it is considered that the stabilizer content is low and its value should be increased.

Method 2: This test is done as per ASTM standard No. ASTM D 559. It is generally known as Wetting and Drying test for determining durability of stabilized soil mixes, which determines the weight losses, moisture changes and volume changes (swell and shrinkage) produced by repeated wetting and drying of hardened stabilized soil specimens. The other is a freezing and thawing test which follows a similar procedure except that wetting and drying is replaced by cycles of freezing and thawing.

In the wetting and drying test, the test specimens are subjected to 12 cycles of wetting and drying, consisting of immersion in water for 5 hours followed by drying at 71ºC for 42 hours. After each cycle the specimens are brushed in a standardised manner with a wire scratch brush (18-20 strokes on the sides and 4 strokes at each end). The loss in weight of the brushed specimens, after each cycle are determined. In a parallel test the volume and moisture changes of the specimens after each cycle is recorded.

The freezing and thawing test is similar to the wetting and drying test but the test cycles consist of subjecting the specimens to freezing conditions at -23ºC for 24 hours followed by thawing at 21ºC for 23/24 hours. The specimens are brushed, as in the wetting and drying test, after each thawing cycle. For climatic conditions prevailing in India, durability under wetting and drying would have to be taken into consideration and durability under freeze/thaw condition does not generally apply.

The principal criterion set by the PCA is that the loss in weight of the specimens after 12 cycles of both freezing & thawing and wetting and drying should not exceed certain limits, depending on soil type. Granular soils of low plasticity are permitted to lose up to 14 percent of their original mass and cohesive clay soils are permitted to lose only 7 percent of their original mass. The reason of the difference is that granular materials abrade more readily than cohesive soils and the wire brushing removes some material in addition to that loosened by the alternate cycles of freezing and thawing and wetting and drying. However, as per some other studies, the above requirements were found to be too stringent and following values have been recommended:

Base: Less than 20 percent

Sub-base: Less than 30 percent

Shoulder: Less than 30 percent

CHAPTER 5

CONSTRUCTION OPERATIONS

5.1 Procedure of Stabilization

The construction of stabilized road pavement layers follow the same basic procedures whether the stabilizing agent is cement, lime or other hydraulic binder. The procedures can be divided in to two main groups:

1) Mix-in-place stabilization

2) Plant-mix stabilization

5.2 Mix-in-Place Stabilization

The main advantage of the mix-in-place procedure is its relative simplicity and hence it is particularly suitable for work in remote areas where plant mixing could prove logistically difficult. Its disadvantages are not obtaining efficient mixing i.e. good distribution of the stabilizer, constructing thicknesses of more than 200 mm and of poor levels.

In this process the material is stabilized in-situ which requires the stabilizing agent to be spread before or during the pulverisation and mixing of the soil and stabilizer. This is generally carried out with a purpose made machine although for small scale work in remote areas agricultural machinery can be adapted for use. In-situ stabilization generally involves the following operations:

Initial Preparation

This involves excavating down to the in-situ material to be stabilized or placing imported material on the formation. The material to be stabilized then has to be graded to approximately the required levels. After which it is usually necessary to plough to loosen the material, one or two passes is normally sufficient.

Spreading the Stabilizer

Spreading the stabilizing agent at the required dosage rate can be carried out manually or by machine. When manual methods are used bags of stabilizer are spotted at a set spacing, they are then broken open and the stabilizer raked across the surface as uniformly as possible. Where quicklime is being used, necessary precaution need to be taken to protect the operators. This is especially true when the stabilizer is being spread by manual method.

Lime has a much lower bulk density than cement and it is possible, therefore, to achieve a more uniform distribution with lime when stabilizers are spread manually. The uniformity of the layer of stabilizer spread over the surface, before the mixing operation, determines the uniformity of the mixed material produced.

Mechanical spreaders automatically monitor the required amount of stabilizer to be spread on the surface of the soil. Their use results in a much more uniform spread of stabilizer over the surface than can be achieved by hand spreading. The equipment need to be calibrated before use to ensure that the correct rate of spread is achieved and subsequently checked at regular intervals to ensure that the rate of spread remains within specified tolerances.

Addition of Water

If it is necessary to add water to bring the moisture content to the required value this can either be done as part of the mixing operation or after the material has been prepared prior to the addition of the stabilizer. To ensure a thorough distribution of the added water, it is preferable to add water as part of the mixing operation. Water added during the mixing process should be through a spray system such that it is added in a uniform manner over the required area and mixed uniformly to the required depth. Where the mixing plant does not enable water to be added or where it is not possible to add enough water during mixing it should be added to the prepared material using a spray system that enables the amount to be controlled over the whole area. The material to be stabilized should then be mixed prior to the addition of the stabilizer to ensure the distribution of the water throughout the layer.

Mixing Soil, Water and Stabilizer

Robust mixing equipment of suitable power for the layer being processed is required to pulverise the soil and blend it with the stabilizer and water. The most efficient of the machines available carry out the operation in one pass, enabling the layer to be compacted quickly and minimising the loss of density and strength caused by any delay in compaction. Multi pass machines are satisfactory, provided the length of pavement being processed is not excessive and each section of pavement can be processed within an acceptable time.

The plasticity of the material is overriding factor in the ability of mixing plant to mix the soil with stabilizer. A review of work showed that all plastic soil could be satisfactorily mixed with cement using the plant. For cohesive soils a factor of the plasticity index of the soil multiplied by the percentage of the fraction of the soil which was finer than 425 micron in particle diameter may be used to suggest the values for the different types of mixing plant available, which are given in **Table 13**.

Table 13 Soil Plasticity Limits for Stabilization Using Different Types of Plant

Type of Plant*	Plasticity Index × Percentage of fraction finer than 425 micron	Normal maximum depth (mm) capable of being processed in one layer
Agricultural Disc harrows, Disc ploughs, rotavators	Less than 1000	120-150
Light duty rotavators (< 100 hp)	Less than 2000	150
Heavy duty rotavators (> 100 hp)	Less than 3500	200-300 (depending on soil type and horsepower of mixer)

* Selection of the appropriate plant should be left to the decision of the Engineer-in-Charge.

Graders have been used to mix stabilized material but they are inefficient for pulverising cohesive soil and even with granular materials a large number of passes are needed before the quality of mixing is acceptable. For these reasons, the use of grader for mixing is not suggested.

Compaction

Compaction is carried out in two stages:

a) An initial rolling and trimming which may be carried out followed by a final mixing pass of the rotovator.

b) Final compaction and levelling in the case of cement stabilized material, must be completed within two hours of mixing. Delay in lime stabilization are less critical and for soil modification there may even be benefits in completing the final mixing, levelling and compaction between one and seven days after the initial mixing. This time gap allows for the reactions between the lime and clay to take place and thus provide a more workable soil. However, for lime stabilization as distinct from modification, the aim should be to complete compaction within three hours after mixing lime with soil. This is particularly true in hot climates where problems of evaporation and carbonation are more likely to occur. In case of cement stabilization, this time period should be reduced to two hours.

Curing

Proper curing is very important for three reasons:

a) It ensures that sufficient water is retained in the layer so that the hydration reactions between the stabilizer, water and the soil can continue

b) It reduces shrinkage, and

c) It reduces the risk of carbonation from the top layer.

In temperate climate curing presents few problems. It is usually carried out by sealing the compacted surface to prevent escape of water during the curing period (usually seven days) during which time all construction traffic must be kept off the stabilized material. Before spraying is started the surface should be swept free of loose material and any damp areas should be free of standing water. The following methods of curing are suggested:

a) covering with an impermeable sheeting with joints overlapping at least 300 mm and set to prevent ingress of water.

b) spraying with a bituminous sealing compound.

c) spraying with a resin based aluminous curing compound similar to those used for concrete. This has particular application where it is desirable to reduce the increase in temperature immediately under the surface which would result from the use of a black (bituminous) seal.

In a hot dry climate, the need for good curing is most important but the prevention of moisture loss is very difficult. If the surface is constantly sprayed and kept damp day and night the moisture content in the main portion of the layer will remain stable but the operation is likely to leach stabilizer from the top portion of the layer. If the spraying operation is intermittent and the surface dries from time to time (a common occurrence if this method is used) the curing will be completely ineffective.

Curing through spraying water can be much more efficient if a layer of sand 30 mm to 40 mm thick is first spread on top of the layer. In this case, the number of spraying cycles per day can be very much less and there is a considerable saving in the amount of water used.

When the stabilized layer is to be covered by other pavement layers the construction of the upper sections will provide a very good curing seal but care has to be taken to ensure that this work does not damage the top of the stabilized layer. During the period of time prior to the construction of the next layer some system of curing is required because, this is the most critical period in terms of shrinkage in the layer.

Primer can also serve as a curing membrane but, results have shown that a prime coat breaks down when it penetrates into the surface and completely loses any ability to seal it. A portion of any curing membrane must sit on the surface to achieve an effective seal if the top of the stabilized layer is sprayed lightly with water followed by an application of a viscous cutback bitumen, the loss of moisture is effectively reduced to zero. Similarly the top of the stabilized layer can be sprayed with an emulsion to achieve the same result. It is essential, however, that all traffic is kept off the curing membrane for several days at which time excess bitumen can be absorbed by the surface.

5.3 Plant-Mix Stabilization

In this process, the materials are separately batched and mixed at a mixing plant. They are then transported to the site where they are laid by a bituminous paver and compacted. The advantages of the process are the good control on proportioning of the materials, multi-layer work can be executed and good compacted levels are readily obtainable. The disadvantages are that output is lower than in the mix in place process, cohesive materials cannot usually be mixed and in the case of cement stabilization, the mixing plant has to be relatively close to the site so that mixing, laying and compaction can all be completed within the stipulated two-hour time limit. The process is not, therefore, applicable to small-scale projects unless there is a mixing plant near at hand.

To ensure complete distribution of the relatively small quantities of stabilizer, mixing should be carried out in a forced action mixer and except for non-cohesive granular materials, free fall mixes of the type used for mixing concrete should not be used. If it is proposed to use a mixer other than one with a forced action preliminary trials should be made to ensure that satisfactory mixing is achieved.

Vehicles transporting the mixed material should be of sufficient number and capacity to meet both the output of the mixer and spreading and compaction operations. International standards and specifications, for plant mixed cement stabilized material require it to be spread by a bituminous paver and spreading by grader is not permitted. If graders are used for spreading, much of the advantage of plant-mix stabilization is lost as it is difficult to control levels and thicknesses of construction.

5.4 Compaction

Whatever method is used for mixing the soil with water and stabilizer material, the methods used for compaction are the same. In the case of cement stabilized materials, once the cement has begun to harden, it is important that the matrix is not disturbed; hence the requirement that compaction must be completed within two hours of mixing. The compacted density of the stabilized layer is a measure of the effectiveness of compaction and hence of its strength. The degree of compaction to be achieved in the field can be specified in two ways. In an end product specification, the density of the layer in the field is determined and compared with a specified target density. Provided that the measured field density is greater than or equal to this limit the compaction in the field is deemed to be satisfactory. The main disadvantages of an end product specification are that a large amount of site testing is required and many of the methods in use are time consuming. This means that the results of the tests may not be available in time to remedy any deficiencies in compaction.

CHAPTER 6

QUALITY ASSURANCE

6.1 General

During the construction process regular checks are to be made on the stabilized material to ensure that all the requirements of the specification are being met. Many of the checks carried out are merely "good housekeeping" i.e. continual supervision to ensure that the construction process allows the design objectives to be achieved in full. In addition to this there are production control tests carried out to monitor the work in progress to ensure, for example, that the correct thickness of stabilized layer is being laid and that a consistent product is being produced. Finally compliance tests are to be carried out on the finished product to demonstrate that it meets all the requirements of the specification.

This Chapter, therefore, describes the tests that may need to be carried out to check on the quality of the material. It also discusses the various factors that influence the choice of a particular test that is used to establish the values for parameters such as moisture content, compacted density, strength, etc., set out in the specification.

6.2 Preliminary Trial

As part of the quality control and in order to make a final decision on moisture content and stabilizer content, the information gained in the laboratory tests should be related to a preliminary field trial. At least 10 days before the main work begins, a trial area should be laid using the materials, mix proportions, mixing, laying and compaction plant to be used, to check the suitability of the methods, etc.

6.3 Sampling and Testing Frequency

Samples for checking the moisture content, strength, etc. are most conveniently taken from the laid material before compaction. Frequency of testing depends on the size of the project and the facilities available on site but regular checks should, at least, be made on the moisture content, strength and in-situ density. Whatever the frequency, sampling should be spread out over the site so as to give a representative indication of the quality of the material within a given area. In order to achieve the specification for stabilized sub-bases and road bases, it is suggested that samples at equally spaced locations along a diagonal that bisects the area to be tested may be taken. For satisfactory performance of soil stabilized road, strict quality control measures are essential. It is prudent to conduct periodic testing during construction to confirm that the properties of materials being used are within the range of value anticipated during the design. For each consignment of cement, lime and fly ash; testing should be done to check quality. Quality control tests and their minimum desirable frequency are as given in **Table 14**. Strict control should be excised during the mix in-place operations, with frequent checks on mixing efficiency. This can be done by trenching through the in-place material and inspecting the colour of the mixture. Unmixed streaks or layers indicate poor mixing and the material in that area should be remixed until uniformity of colour is achieved.

Table 14 Quality Control Tests for Cement-Fly Ash and Lime-fly Ash Stabilization

Test	Test Method	Minimum Desired Frequency
Quality of Cement	As per relevant IS Specifications	Once initially for approval of the source of supply and later for each consignment of the material
Quality of lime	IS 1514	Once initially for approval of the source of supply and later for each consignment of the material subject to minimum of 1 test per 5 tonnes of lime
Quality of Fly Ash	IS 3812	Once initially for selection of the source of supply and later for each lot of 10,000 kg
Degree of pulverisation	IS 2720 (Part-4)	Periodically as considered necessary
Moisture content	IS 2720 (Part-2)	One test per 250 sq.m
Density of compacted layer	IS 2720 (Part-28 or 29)	One test per 500 sq.m
Deleterious constituents	IS 2720 (Part-27)	As required
CBR or unconfined compressive strength test on a set of 3 specimens	IS 2720 (Part-16) IS 4332 (Part-5)	1 test per 3000 cum of mix
Thickness of layer	-	Regularly
Lime/Cement content	-	Regularly through procedural checks

6.4 Storage and Handling of the Stabilizer

Unless cement and lime are properly stored and used in a fresh condition the quality of the pavement layer will be substantially reduced. Cement must be stored in a sound water-tight building and the bags stacked as tightly as possible. Doors and windows should only be opened if absolutely necessary. The cement which is delivered first should be used first. According to a study it was found that even if cement is properly stored the following losses in strength will still occur:

Age	Percentage Reduction
After 3 months	20
After 6 months	30
After 1 year	40
After 2 years	50

Lime should be stored in sealed bags, tightly stacked and covered with a water proof tarpaulin. The material which has been stored for more than three weeks should be tested for available lime content before use. Lime which is older than 6 months should be discarded.

6.5 Control of the Moisture Content

Throughout the stabilization work, the moisture content should be maintained slightly above its specified value. This means that rapid determination of the in-situ moisture content is necessary to allow adjustments to be made so as to bring the moisture content of the stabilized material to the required value.

The definitive oven-drying method is, in general, too time-consuming to be of much practical use in the field and more rapid means have to be employed. Rapid heating methods may be used, but where these are inappropriate, the calcium carbide method may be used to give rapid result. The method depends on the reaction between calcium carbide and water in the stabilized material to produce acetylene at the ambient temperature according to the equation:

$$CaC_2 + 2H_2O = Ca\,(OH)_2 + C_2H_2$$

If the reaction is allowed to occur under standardised conditions in a closed container, the pressure of the acetylene generated in the container is a measure of the moisture content of the stabilized material.

Nuclear density gauges for the determination of the in-situ density of compacted materials usually include a facility for the in-situ moisture content at the same time. This method can be used to determine moisture content when construction starts and also during the processing.

6.6 Control of the Stabilizer Content

Whatever method of spreading the stabilizer is employed, it is important that a uniform spread rate is achieved as this will affect the uniformity of the stabilized material. If the stabilizer is placed in bags and spread by hand, the accuracy of the spotting of the bags must be checked and the manual spreading of the stabilizer should be visually assessed. If a mechanical spreader is used, metal trays or canvas sheets, one metre square, should be placed at regular intervals along the road to check the application rate.

Determination of the stabilizer content, after mixing, is in principle easy to perform but in practice is time consuming and needs to be carried out with care if meaningful results are to be obtained. Both the methods described in the codes BS:1924:Part 2 and in ASTM D 806 involve a comparison of the calcium contents of the stabilized material, the stabilizer and the material in an un-stabilized condition. However, the method given for the determination of calcium in BS:1924 is to be preferred. Neither method is applicable if the calcium content of the un-stabilized materials is high or variable.

6.7 Routine Strength Determinations

Continuous monitoring of the strength of processed material is required to ensure that the specified strength is being achieved. Representative samples of the full depth of mixed material should therefore be taken from the site immediately prior to compacting the material. As stated previously, the frequency of sampling should be related to the size of the processed area and its structural importance. In the case of cement stabilized materials, preparation of the test specimens should be completed within two hours of mixing.

The moisture content to be used for the preparation of the test specimens will clearly be that of the mixed material and precautions should be taken to ensure that no drying out of the material occurs between taking the samples and completing the preparation of the test specimens.

The density at which the test specimens are to be compacted depends on the density requirements of the specification and various methods which are in use. The test specimens should be prepared at the same density as the compacted material in the field. This has some logic because it means that there should be no differences in strength, which can be attributed to differences of density, between the laboratory test specimens and the strength of the material in the field. The difficulty is that an immediate measure of the in-situ density is required and this can only be achieved if nuclear density gauges are used.

CHAPTER 7

PRECAUTIONS TO BE TAKEN WHILE USING STABILIZED MATERIALS

7.1 General

The two major problems that arise with the use of stabilized materials in road pavement layers are cracking and the long -term durability of the material. The extent to which either of these is a problem is intimately related to the purpose of the stabilized layer in the road pavement as a whole and it is, therefore, difficult to divorce the two factors. However, in this Chapter the problems that can arise are discussed.

7.2 Cracking in Stabilized Layers

Many factors contribute to the cracking and crack -spacing of stabilized pavement layers. Some of them are listed below:

1) Tensile strength of the stabilized material;
2) Shrinkage characteristics;
3) Volume changes resulting from temperature or moisture variations;
4) The subgrade restraint;
5) Stiffness and creep of the stabilized material, and
6) External loadings such as those caused by traffic.

As in the case of compressive strength, the tensile strength of stabilized materials takes time to develop. On the other hand, stabilized material in a road pavement layer will be subject to volume changes from at least one of the factors listed above as soon as it is compacted. Cracking in stabilized layers due to changes in temperature or moisture content cannot, therefore, be avoided and must be accepted as inevitable although steps can be taken to reduce the effect. Cracking may also occur as a result of fatigue failure due to trafficking and is an entirely separate phenomenon from the initial cracking due to environmental changes.

Cracks in stabilized layers used at capping and sub-base level are unlikely to cause significant problems but at base level the cracks may be reflected through the surfacing. The existence of cracks in a road surface may be assumed to indicate need for remedial action. The consequences of not doing so, may range from no problems at all to loss of interlock or to eventual failure when the stabilized layer has been reduced to unconnected blocks. Cracks may also permit ingress of water leading to weathering of materials at crack faces, de-bonding between pavement layers, or deterioration of moisture-susceptible layers beneath the stabilized layer.

7.3 Primary Cracking

Cracks appear in cement-stabilized materials as a result of shrinkage and temperature fluctuations. The initial crack pattern is dependent on the early strength of the material and the properties of the material used. Materials which have low strength, normally also contain a higher proportion of plastic fines. The stabilized materials with lower strength and with high proportion of plastic fines have frequent but narrow cracks. Whether or not these frequent but fine cracks prove to be a problem depends to a large extent on the mechanical interlock at the face of the cracks. If the interlock is good the material performs satisfactory and the cracks are sufficiently fine for them not to be reflected through the pavement layer above. However, the lower strengths of these stabilized materials mean that they are generally only suitable for use in the lower layers of the road where cracking is less of a problem anyway.

On the other hand, stabilized materials with high strength criteria and which have little, if any, plastic fines have fewer but wide cracks. These cracks are often wide enough for them to be reflected through the surface. In order to restrain the propagation of lateral reflective cracks, such materials therefore have to be covered with a greater thickness of construction material than would otherwise be required.

The temperature at which the material is laid also plays a part in the type of crack pattern that is produced. Layers placed in cooler weather tend to develop fewer and narrower cracks as there is less thermal shrinkage. The stabilized layer may subsequently be in compression, apart perhaps from prolonged cold spells, so that cracks remain closed with good load transfer. Fewer cracks develop when the temperature difference between day and night during construction is not large, as thermal warping is reduced.

Lime- stabilized materials are also subjected to cracking for the same reasons. However, the effects are not so pronounced; if the cracks occur before unreacted lime in the layer has been used up, either in pozzolanic reactions or by carbonation, the continuing pozzolanic reaction of the lime can result in self- healing (autogenous healing) of the cracks.

Given that cracking is inevitable, the ideal condition is for materials to have low early strength which lead to numerous fine cracks but high long- term strengths which mean good mechanical interlock at the face of the cracks. As lime is slower to react than cement, this is another reason for favouring lime, provided it can achieve high long- term strengths.

Another possibility is to use secondary additives to modify the hardening action of the cement to reduce its early strength without affecting its long-term strength.

7.4 Traffic Associated Cracks

Quite separately and much more importantly than the primary transverse cracks, cracks may appear in stabilized bases of inadequate strength or inadequate construction thickness in relation to the traffic and the sub-grade strength. Such cracking takes the form of "map" cracking which, in extreme cases, causes the stabilized material to deteriorate into small slabs with poor load transfer. Once started, deterioration is likely to continue until the stabilized base becomes little more effective than granular sub-base.

When extensive cracking has developed as a result of the combined action of free water and traffic, then it often results in the "pumping" to the surface of fine material from the underlying pavement layers where it is deposited in the cracks. The fines discolour the surface along the cracks making them clearly visible.

Unlike the primary cracking, the appearance of traffic-associated cracks is not inevitable. It should not occur if the road pavement has been properly designed to take account of the traffic likely to be encountered during the design life of the road.

7.5 Durability of Stabilized Materials

The failure of stabilized materials by disintegration into a loose mass is not common. It is most likely to be due to deficiency either in the amount of stabilizer, deficiency in the quality of the stabilizer, or deficient compaction or curing. These problems should not occur if a good standard of preliminary testing for suitability and of quality control are maintained.

It is reported that the most common type of failure of stabilized layers is the peeling-off of surface dressings from stabilized layers. This is usually due to failure of top of the layers itself rather than any of the shortcoming of the surface dressing. The surface of the layer tends to disintegrate under traffic, the most likely cause of which is considered to be as a result of overstressing of the surface layer during the compaction of the stabilized material at the time of construction. This induces a series of shallow shear planes in the surface layer and result in a sharp falling-off density of the material towards the upper surface. Overstressing is most prevalent with uniformly graded non-cohesive sands. It can be avoided if special care is taken with the compaction and if towed vibrating rollers are used.

A survey of known causes of lack of durability of stabilized layers confirmed that the most common problem was surface disintegration of the primed layer during construction and scabbing of the seal in service due to an inadequate bond with the stabilized material. These problems are a result of inadequate compaction and curing and are more likely to occur in hot, dry climates. Apart from the problem of surface disintegration, long-term durability may also be impaired by the effects of sulphates and by carbonation.

7.6 Control of Reflective Cracking in Cement Stabilized Pavements

Although the potential exists for reflection cracking when a cement-stabilized base is used in a pavement structure, proper construction and design techniques can minimize the potential that the pavement will be adversely affected.Proper construction practices to minimize drying, pre-cracking soon after construction, and designing for stress relief are all valid methods that will reduce or eliminate the formation of reflection cracks in cement-stabilized bases.

There are several factors as discussed in the previous chapter, which contribute to the cracking in a cement-stabilized base/sub-base. With regard to material characteristics, the type of soil, cement content, degree of compaction and curing, and temperature and moisture changes directly influence the degree of shrinkage.

There are a number of preventative measures and design concepts that can be used to minimize shrinkage cracking in the cement-stabilized base, and to reduce the potential,that base cracks will reflect through the asphalt surface. Methods of controlling reflective cracking include proper construction and curing of the stabilized base, reduction of crack size through the use of "pre-cracking", and relief of stress concentrations through the use of flexible layers in the pavement structure.

A cement-stabilized base provides excellent support for asphalt surfaces. The stabilized base material is stronger, more uniform and more water resistant than an un-stabilized base. Loads are distributed over a larger area and stresses in the subgrade are reduced. However, cement-stabilized bases can also be the source of shrinkage cracks in the stabilized base layer, which can reflect through the asphalt surface. The cracks that develop are not the result of a structural deficiency, but rather a natural characteristic of cement-stabilized bases. The surface cracks tend to follow the same pattern as the cracks in the base, and are referred to as "reflection" cracks.

In most cases, reflection cracks are narrow (less than 3 mm) and will not adversely affect the performance of the pavement. However, wider cracks can result in a rough riding surface and deterioration of the pavement. The wide cracks create an environment for water infiltration and subsequent pumping of the underlying subgrade.

Several factors contribute to the cracking and crack spacing in a cement stabilized base which include material characteristics, construction procedures, traffic and restraint imposed on the base by the subgrade. With regard to material characteristics the primary cause of cracking is due to drying shrinkage of the cement stabilized base. The degree of drying shrinkage is affected by the type of soil, degree of compaction and curing, cement content , temperatures and moisture changes.

Cement stabilized fine grained soils e.g. clays exhibit greater shrinkage than cement stabilized granular soils. Although stabilized clay soils develop higher total shrinkage than granular soils, the cracks are typically finer and more closely spaced often of hairline variety spaced 0.6 to 3.0 m apart. The granular soilsgenerally produce less shrinkage but develop larger cracks typically spaced at 3.0 to 6.0 m apart.

Fine grain grained soils have large surface area than granular soils and typically require higher moisture content for compaction purposes. In addition, cement content for fine grained soils are generally 2 to 5 percent higher than granular soils in order to achieve adequate durability and strength. Both these factor contribute to higher moisture contents for stabilized fine grained soils and consequently higher drying shrinkage.

The effect of compaction on shrinkage characteristics of cement stabilized material plays an important role. A well compacted mixture exhibits reduced shrinkage potential, because the soil/ aggregates particles are packed tightly together resulting in reduced voids. It has been reported that compacting cement stabilized soil at modified proctor effort, reduces shrinkage significantly as compared to stabilized soil compacted to standard proctor density. The reason for the same can be attributed to the fact that the optimum moisture contents at modified proctor compaction are typically less than at standard proctor compaction which helps to reduce shrinkage.The least amount of shrinkage is obtained for the stabilized material at the highest density and lowest moisture content.

Cement hydration contributes less to shrinkage than does many other factors. In fact, for soils that exhibit volume change without cement, increasing cement will decrease total shrinkage. However, excessive amounts of cement can exacerbate cracking in two ways: First, increased cement contents cause greater consumption of water during hydration, thus increasing drying shrinkage. Also, higher cement levels cause higher rigidity and excessive strength (both tensile and compressive).

Methods of controlling reflective cracking basically fall into the two categories :

- Pre-cracking
- Providing for stress relief at the base-surface interface

Pre-cracking: Minimizing crack width with proper construction and curing procedures, as discussed in the previous sections, will eliminate much of the potential for wide cracks. Another method to reduce crack width is a relatively new procedure called "pre-cracking", where hundreds of tiny micro-cracks develop instead of single transverse cracks. The method has been successfully tried on several projects in the United States. The procedure involves several passes of a large vibratory roller over the cement-stabilized base one to two days after final compaction. This introduces a network of closely spaced hairline cracks into the cement-stabilized material, which acts to relieve the shrinkage stresses in the early stages of curing, and provides a crack pattern that will minimize the development of wide shrinkage cracks. In addition, since the pre-cracking is performed shortly after placement, the "micro- cracking" will not impact the pavement's overall structural capacity as the cracks will heal and the cement-stabilized material will continue to gain strength with time.

Stress Relief: Another method of reducing the potential for reflection cracking is to relieve the stress concentrations that result from cracks in the cement-stabilized base. The following three methods have been successfully usedto reduce the stresses that cause reflection cracks:

1) A bituminous surface treatment (chip seal) between the stabilized base and the asphalt surface. The additional flexibility of the surface treatment layer will help to reduce stress concentrations. This surface treatment also provides an excellent temporary surface during construction for traffic control.

2) A geotextile between the stabilized base and surface, or between the asphalt binder and wearing courses. Similar to the surface treatment, the geotextile provides flexibility and acts to intercept cracks without letting them pass through the material.

3) A 50 mm to 100 mm layer of unbound granular material between the stabilized base layer and the asphalt surface. This use of a "sandwich" or "inverted" pavement design adds additional structure to the pavement, and will prevent the propagation of cracks through to the surface layer.