INNOVATIONS IN DESIGN AND CONSTRUCTION OF A LOW VOLUME LOW COST ROAD ON WINDBLOWN SANDS

P.J. Strauss, Bruinette Kruger Stoffberg Incorporated, Consulting Engineers, Pretoria, Republic of South Africa
F. Hugo, Professor of Civil Engineering, University of Stellenbosch, Stellenbosch, Republic of South Africa

The design of a low cost pavement structure for low volume roads in some arid parts of Southern Africa is described. Only two available sources of material exist, namely calcrete and windblown sand, both of which vary greatly in quality. Several methods of possible improvement in strength are described and results are based both on laboratory as well as field measurements. Two pavement systems are considered in design namely a thin surfacing on a stabilized base and an asphaltic concrete made of calcrete/sand on in situ compacted sand. Both systems are susceptible to variance in material quality and this is considered in a method of pavement design. The statistical method of design in which variance in material properties is taken into account, considers a simplified two layer system which allows failure of the pavement area to be predicted. It is concluded that the improvement of the quality of available materials through selection and stabilization is necessary but the extent thereof depends on the variance and the mean strength of the product.

The design and construction of a highway as an indispensable part of the infrastructure of a developing country takes on a new dimension when a deficiency in quality material exists. The in situ material in parts of South West Africa mainly consists of an aeolian or windblown sand, often called a Kalahari-type sand, that compacts well under optimum moisture conditions but by its very nature is single-sized and hence very unstable in its uncompacted state. Thus the bearing capacity drops considerably under uncompacted saturated conditions.

The majority of vehicles which use these roads can be classified as heavy vehicles, i.e. bigger than 6 ton trucks, which take a heavy toll in riding quality on dirt roads. Although the designed for number of equivalent 80 kN (18 000 lb) axle loads is only of the order of 80 000 to 200 000 for a 20 year design life, it is economical to surface these roads in order to cut on maintenance costs.

Gravel material that can be used in pavement construction is calcrete with variable quality as far as material strength is concerned. The usable strength of calcrete is further impaired in the course of time by the presence of soluble salts. These salts lead to the destruction of any bituminous surfacing if not checked by preventive measures such as stabilization or proper construction procedures.

The use of sand/calcrete asphaltic concrete as surfacing is generally favoured since crushed stone has to be imported over long distances with the result that treatment or conventional asphaltic concrete as surfacing becomes too expensive. If high quality surfacing is used, it is implicit that it contributes to the structural strength of the pavement.

The final result is that two types of pavements can be considered namely a 75 mm (3 in) asphaltic concrete layer, or a 12 mm ($\frac{1}{2}$ in) asphaltic surface layer and 150 mm (6 in) stabilized calcrete base, on the well compacted windblown sand subgrade.

Characteristics of Materials

Considerable areas of the northwestern region of Southern Africa can be classified as arid or semi-arid and in situ materials mainly consist of an aeolian or windblown sand. The most common source of road building material is calcrete which primarily occurs in lower lying, poorly drained areas. Other than calcrete which consists of nodules, hardpans and strongly calcified sands, no other source of aggregate is economically available. Thus the engineer has to make the most of the meagre supply of good quality calcrete and sand to substitute for the commonly required high standard materials.

Windblown Sands

The single-sized sand virtually all passes the 2 mm (0,078 in) sieve and is non-plastic with a A-2-4 or A-3 classification. No more than 12% passes the 0,075 mm sieve which tends to cause low stability in the sand. Thus the CBR ranges from 7 to 45 with a mean value of 22 and standard deviation of 8 at a 95% Modified AASHTO density.

Apart from relatively low strength considerable variation of in situ density occurs. Field densities can be as low as 76% Modified AASHTO which implies a possibility of severe settlement under favourable conditions such as saturation and heavy loading (dynamic or dead loads). The CBR strength of the material at this density is 2 when fully saturated but has a CBR of 15 at field moisture conditions. Thus the obvious means of ensuring strength is by proper compaction and drainage.

Calcrete

Calcrete, a pedogenic material, is formed by the cementation or replacement of existing soils through the deposition of calcium carbonate from soil water (1). The material can be indentified as strongly calcified sands, hardpans or nodules, or a mixture of these.

Usually chunks of calcareous material up to a size of 75 mm (3 in) occurs in a matrix of fine material of which as much as 35% passes the 0,075 mm (No 200) sieve. The fine material can be classified as calcified sand and the plasticity index is generally lower than 13. The strength of the calcrete ranges from a CBR of 7 to 110 at 95% Modified AASHTO density. Borrow pit material, however, can be quarried with discretion to obtain a mean CBR strength of 58 and standard deviation 18 or under consistent control a mean CBR of 70 with standard deviation 12.

The biggest concern when using calcrete material in road layer work is the existence of soluble salts. Several investigators (2, 3, 4) have reported on this phenomenon and it was found that the most common deleterious salts are NaCl and Na_2SO_4. The actual damage is caused by the crystallization of salt between the bituminous surface seal and base course due to the evapotransporation of soil water. If the calcrete material is not chemically stabilized, attacks from sulphates may occur with a resultant break-up of the layer so that the limitation of both soluble salts as well as sulphates in calcrete are of prime importance.

Variations in calcrete quality makes it suspect as a natural base material, especially since soluble salts contribute to a degeneration of the engineering properties. Thus, mechanical or chemical improvement is essential in most cases, not only to avoid salt damage but also to increase mean strength.

Improvement of Materials

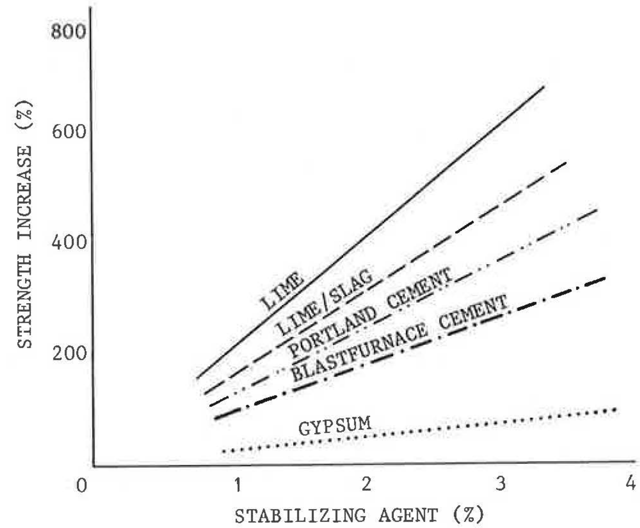
Several routes are available to the engineer for improving the engineering characteristics of the available materials. Amongst these are chemical alterations, mechanical stabilization and the addition of a suitable bituminous binder material. Although laboratory testing is necessary, the measuring of actual field performance by experimental sections is a more reliable way to evaluate the relative success of each method of stabilization on an equal basis. The laboratory methods of assessment is through testing using unconfined compression (UCS), Marshall, CBR and indirect tensile strength (ITS). Although the laboratory testing cannot fully take into account the effects of climate and traffic especially on chemical reactions, weathering and stability of the layer work, it nevertheless gives an indication of relative strength.

Chemical Improvement

Improvement of the bearing capacity of calcrete is possible by ion exchange and/or cementation through the addition of a chemical substance such as a calcium type road lime, milled blastfurnace slag, Portland cement or a combination of these. A beneficial byproduct of stabilization with lime in some calcretes is the prevention of eventual salt damage.

The improvement in strength can be measured in terms of CBR, UCS or ITS. Figure 1 gives an indication of the increase in strength through different stabilizing agents and stabilizer contents for a calcrete material from one borrow pit.

Figure 1. A change in strength with type and quantity of stabilizing agent.



Due to the great variation in calcrete quality, considerable scatter can be expected in the strengths of stabilized materials and it is therefore quite possible that the other types of stabilizing agents may perform better with other calcretes than the lime or lime/slagment as indicated. However, road lime is generally favoured because of better reaction with deleterious salts and slower gain in tensile strength in due course. The latter is important since block cracking of the stabilized layer must be curtailed in poor drained areas.

The change in soluble salt content with the addition of lime can best be illustrated by actual field results as indicated in table 1. Table 1 shows results of laboratory testing on samples taken from two sections of pavement in service. One section was built with natural calcrete material and the second section with exactly the same calcrete material but stabilized with 3% road lime. Samples were taken at 50 mm (2 in) intervals to a depth of 200 mm (8 in). The results show that a concentration of soluble salt can be found at the surface. It is also clear from the values in table 1 that lime treatment assists in reducing the soluble salt as well as the sulphate content.

Table 1. Soluble salt content* on a typical sample of natural and lime stabilized calcrete.

Depth of Sample	Unstabilized (% Salt)	3% Lime stabilized (% Salt)
0 - 50 mm	1,67	0,37
60 - 100 mm	0,96	0,13
100 - 150 mm	0,59	0,07
150 - 200 mm	0,25	0,10
Mean Sulphate content of layer	0,257	0,035

*Tested according to test method CSIR CA 21 (2).

No salt damage is experienced with the sand, therefore the chemical stabilizing of sand is aimed at improving strength. However, unless the quality of available calcrete is excessively poor, stabilized sand is not used as base material. Several reasons for this can be put forward :

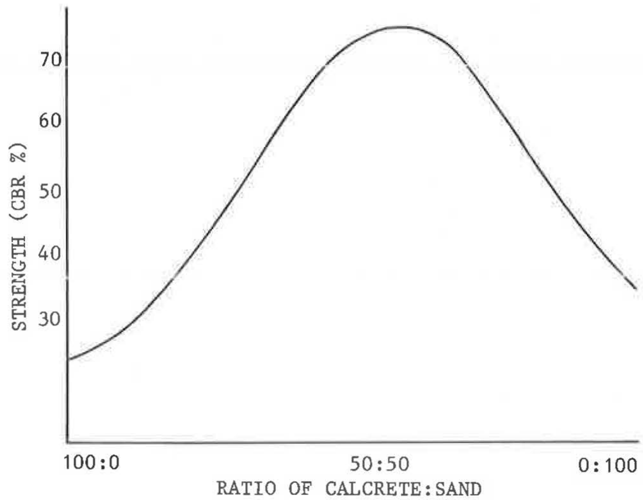
1. The pH of sand is low, of the order of 3,4 to 6,2. This will call for an increase in the quantity of alkaline stabilizing agents.
2. Only cementitious agents can be used for stabilizing since the sand is a non-cohesive, non-active material.
3. Shrinkage crack widths can be expected to increase as tensile strength increases. This reduces the structural performance of the layer.
4. Chemical stabilization of sand with Portland cement is more expensive since more agent is required than when calcrete is stabilized with lime for the same final cost benefit.

Thus, lime stabilized calcrete is favoured as a base under thin surfacing. The mean CBR of a good quality calcrete properly mixed with 1% lime, is 118 with a standard deviation of 22 compared to a mean CBR of 70 and standard deviation of 12 for the unstabilized calcrete.

Mechanical Improvement

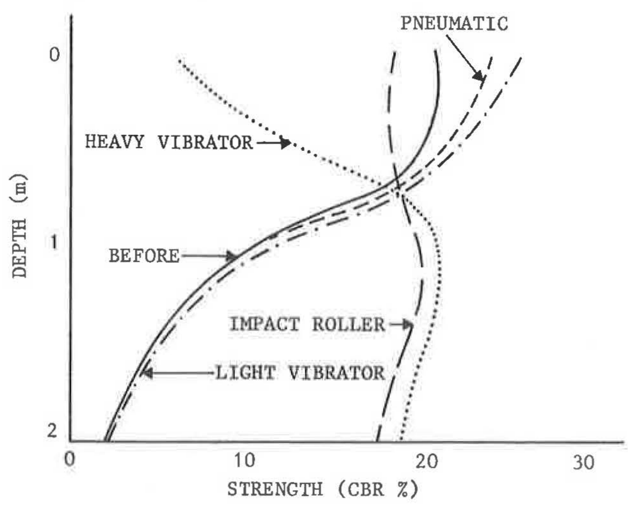
Two methods by which the strength of available materials can be improved mechanically are by mixing calcrete and sand, and by means of heavy compaction. The first method involves the improvement of the grading of calcrete by adding sand whereby density and mechanical interlock, and thus strength, is increased. The relative success of adding the sand to calcrete is shown in figure 2. In this particular case the maximum strength was reached at a 50 : 50 ratio of calcrete to sand. The increase in strength in terms of CBR is generally of the order of 25 per cent which implies an expected mean CBR of around 70 at 95% Modified AASHTO density for the mixture. This method of improvement, however, does not decrease variation in strength since it was found that at a fixed ratio of calcrete : sand, the increase in strength may vary between 0 and 50 per cent for materials from the same borrow pit. Chemical stabilization may thus be necessary to further increase strength to a level high enough for use as a base material.

Figure 2. Mechanical stabilization by mixing calcrete and sand.



In the evaluation of these strengths it is assumed that a high degree of compaction is attained. Keeping in mind that in situ densities of undisturbed material may in some cases be as low as 76% Modified AASHTO, improvement of strength through proper compaction is essential. This can be achieved by using heavy compactors directly on in situ material without necessarily scarifying or adding moisture to the material. Figure 3 indicates the increase in strength in terms of CBR by using a pneumatic, light vibratory, impact or square roller, and heavy vibrator with static weights of around 12 ton, 8 ton, 10 ton and 12 ton respectively. (The impact roller is a 10 ton roller with pentagonal drum which affects compaction through impact (5)). The relative success of the different rollers is very much dependent on the types of materials, moisture content and depth below surface at which measurements are taken. Generally it can be said of a windblown sand that the combination of a heavy vibratory followed by a light vibratory roller achieves the best results.

Figure 3. The effect of various roller on strength with depth.



The results of this combination are illustrated in table 2 which lists the change in CBR as well as density with depth on a windblown sand. The results in table 2 clearly indicate the benefit derived from heavy compaction in the sand. Not only has the mean strength increased but the variance of strength decreased over the compacted area because of more uniform densities.

Table 2. Average change in density and CBR of a windblown sand when compacted by a combination of heavy and light vibratory rollers.

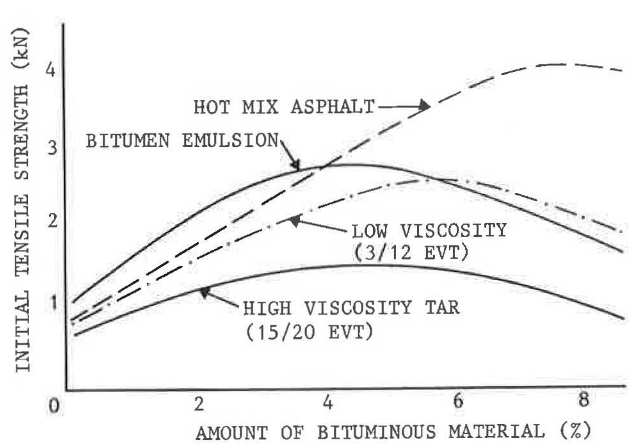
Depth (mm)	Density (kg/m^3) Before	Density (kg/m^3) After	CBR* Before	CBR* After
0 - 200	1 710	1 760	15	20
200 - 500	1 680	1 760	12	20
500 - 800	1 650	1 755	8	19
800 - 1 200	1 615	1 740	6	18
1 200 - 1 600	1 610	1 710	5	5

*CBR deduced from laboratory curves of CBR versus density.

Improvement by Asphaltic Materials

Several types of bituminous binders can be utilized as base stabilizers. Materials that have been used include different grades of tar, bitumen and bitumen emulsion. Figure 4 indicates the change in strength of sandy calcrete stabilized with different types of binders, as a function of binder content. The ability of the materials to be mixed properly appear to have a marked effect on the final strength. Thus the low viscosity tar (3/12 EVT) as well as the aniomic bitumen emulsion showed greater relative laboratory strength when compared with high viscosity tar (15/20 EVT). Also shown in figure 4 is the relative strength of a hot mix asphalt which contains 50 : 50 good quality calcrete and Kalahari sand mixed with 60/70 pen bitumen.

Figure 4. Variation in strength of a bituminous stabilized sandy calcrete.



Experimental sections of the different types of binder materials were constructed. These sections consisted of a 150 mm (6 in) stabilized layer without any seal and with a traffic load of about 10 twelve ton tandem axle trucks per day for 6 months. A panel rating of these sections is shown in table 3.

Table 3. Relative success* of bituminous stabilised sandy calcrete.

Time (months)	Property	Emulsion	3/12 EVT Tar	15/20 EVT Tar
3	Ravelling	4	4	3
	Deformation	3	3	3
	Cracks	5	5	5
12	Ravelling	2	2	3
	Deformation	3	3	3
	Cracks	3	2	3

*Scale : 5 excellent
4 good
3 average
2 poor
1 very poor

The low viscosity tar, showed inferior performance under actual field conditions since ravelling of the layer occurred within twelve months under traffic. The high viscosity tar was the best performer after 12 months as less ravelling and very little rutting occurred under traffic. This is somewhat contrary to laboratory strength results as indicated in figure 4. The reason for this difference may be found in the ease of mixing low viscosity tar with sandy calcrete but with a gradual loss of strength due to ageing.

Pavement Design

The foregoing background to the different types and varying quality of the available materials now leads to the question of how to implement the information into the most economical design/construction strategy. The strategy can be discussed under two headings namely the application of methods to improve the strength characteristics and the accommodation of variance in the strength of the materials.

Application of Methods of Improvement

The availability of material does not leave many options open to the engineer. However as an important first step the subgrade can be compacted. Not only does this improve strength but it also decreases variability which will find application in the statistical design method.

The improvement in base strength can be achieved by the various means of stabilization already discussed. The extent of improvement however depends on the strength required which is dependent on the type of surfacing or wearing course to be used.

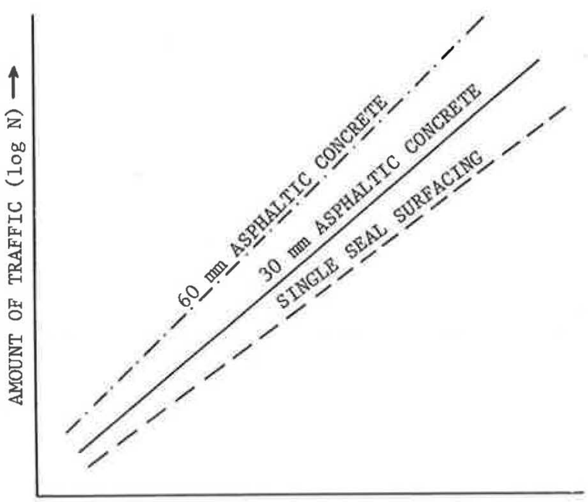
As far as the wearing course is concerned, an asphaltic concrete consisting of a high quality calcrete/sand mixture and bitumen gives the best performance. Some typical Marshall criteria are listed in Table 4. Although the voids are high, low air permeability is measured which implies a dense mix not prone to excessive ageing. Since the stability is reasonably high, the layer can also be utilized as a structural layer.

Table 4. Characteristics of a Typical Calcrete/sand Asphaltic Concrete

Binder Content (%)	Voids in Mix (%)	Stability (kN)	Flow (mm)	Air Permeability ($X\ 10^{-8}\ cm^2$)
7	16	3,5	2,0	0,9
8	14	3,7	1,5	0,7
9	12	3,3	1,3	0,5

The contribution made by a thickness of asphaltic concrete can conceptually be illustrated as in figure 5 where the thickness of asphalt required to withstand a certain amount of loading without overstressing the unstabilised sublayers is shown. Also shown in the figure is the alternative design to an asphaltic concrete layer namely the use of a single seal surfacing on top of an even stronger base. Stabilization may become inevitable unless a relatively salt free, high strength calcrete can be selected from the borrow pit areas and a relatively thick asphaltic concrete can be used as surfacing.

Figure 5. The conceptual relationship between base strength and thickness of surfacing.



where

z = the standard normal variable from which the area failed can be determined using statistical tables

n = the expected total number of load applications e.g. equivalent 80 kN (18 000 lbs) axle loads

N = the total number of load applications that can be tolerated on the pavement system, i.e. the mean number of load application designed for

$S^2_{\log n}$, $S^2_{\log N}$ = variance of log n and log N respectively

The value of n can be determined with the expected life, growth as well as present day traffic in mind. The value of N is determined from known pavement design procedures such as multi-layer and fatigue analysis and by using the mean strength of materials to be incorporated in the pavement structure. The value for $S^2_{\log n}$ is purely a function of the accuracy with which n can be estimated while $S^2_{\log N}$ has to be calculated.
The most convenient way to calculate the value of $S^2_{\log N}$ is by making use of Taylor's Theorem (7):

$$f(x,y) \simeq f(\bar{x},\bar{y}) + (x - \bar{x})\ f'\ (\bar{x}) - (y - \bar{y})\ f'\ (\bar{y}) + \text{second order derivatives} \quad (2)$$

where x,y = value of variables with means $\bar{x}$, $\bar{y}$

f' = first derivative of a function

Equation 2 can be expanded to the form for variance and can be written as follows (6):

$$\text{Variance } f(x,y) \simeq f^2(\bar{x}) \text{ variance } (x) + f^2(\bar{y}) \text{ variance } (y) + \text{ second order derivatives} + \text{ factors with covariance} \quad (3)$$

The function (x,y) which simulates a function of N in this case, has to be derived from a mathematical equation for N which relates strain with fatigue characteristics :

Materials Variation and Design

The influence of variation in material strength on the ultimate performance of a pavement can be taken into account and the principle can be illustrated by the expression (6):

$$z = \frac{\log n - \log N}{\sqrt{S^2_{\log n} + S^2_{\log N}}} \quad (1)$$

$$N = f(\tfrac{1}{\epsilon})^t \quad (4)$$

where

t = factor relating strain ϵ to number of loads to failure

ϵ = f (Modulus of elasticity E, stress S)

The principle can be illustrated by assuming an end loaded continuous beam on an elastic foundation. The simplified form for the stress in the top of the bottom layer can be written as (8):

$$\text{stress } S = f\left[\frac{P}{h^{0,6}}\sqrt[5]{\frac{E_2}{E_1}}\right] \quad (5)$$

where

P = magnitude of load

h = thickness of top layer

E_1, E_2 = stiffness values for top and bottom layer

Assuming a constant relationship between stress and strain and by substituting equation 5 in 4 :

$$\log N = C - t \log\left[\frac{P}{Eh^{0,6}}\sqrt[5]{\frac{E_2}{E_1}}\right] \quad (6)$$

where

C = constant

Differentiating equation 6 and deriving an equation in the format as shown in 3, a value for variance S^2 can be written for failure in the bottom layer.

$$S^2_{\log N} = (t \log e)^2\left[\left(\frac{S_p}{p}\right)^2 \quad \left(\frac{0,6S_h}{h}\right)^2 + \left(\frac{0,80\ S_E}{E_2}\right)^2 + \left(\frac{0,20\ S_E}{E_1}\right)^2\right] \quad (7)$$

In all cases the values for material stiffness E are assumed to be related to strength and thus can be substituted by UCS, ITS or CBR, whatever is convenient, thus assuming a fixed relationship between these factors and E. Since the ratio of standard deviation to mean strength is employed, the actual relationship is of little importance in this calculation.

The fact that a two layered system is assumed may be a simplification but the main consideration is that a relatively thin wearing course, about 12 mm (0,5 in), with virtually no structural contribution will be used on a stabilized base which means that the stabilized layer is considered as the top layer for all practical purposes. The only question is whether the subgrade is uniform enough in depth to be considered as one layer. In the event of no stabilized base course being used, the sand asphalt, about 75 mm (3 in) thick, can be considered as the top of two layers on a uniform bottom layer, the subgrade material.

As an example the previously mentioned design of a 150 mm stabilized material with thin surfacing on an in situ compacted subgrade can be used. Using the mean and standard deviations for the subgrade and stabilized layer as reported on previously and a standard deviation of 25 mm for the thickness of the base, a value of 0,37 is derived at for the variance of log N by using equation 7. Assuming no variance in determining log n, the value $S^2_{\log n} + S^2_{\log N}$ in this case is calculated to be 0,37. If 80 000 axle loads i.e. log n is equal to 4,90, are expected in the life of the pavement, the value of z becomes 0,99 since the value of N is approximately 320 000 from multi layer and subgrade strain analysis based on mean material characteristics. This implies 16% of the area will have failed in terms of rutting and unevenness after the design life of 80 000 axle loads have expired.

Conclusions

The use of relatively low quality material in low volume roads is becoming increasingly necessary, especially since the cost has to be kept to a minimum. The quality of these materials can be increased substantially by different techniques of which mechanical and chemical procedures show the biggest potential for base materials. Since relatively thin layers of wearing course are used on stabilized bases the incidence of the reflection of stabilized cracking increases. Thus small amounts of stabilizing agents are normally added whereby strength is increased to the required level. In the event of a calcrete/sand asphalt being used as wearing course, an increase in thickness of this layer may decrease the required strength of the subbase and only mechanical stabilization may be required.

The single most important aspect in the design and construction of a low volume low cost road is the variation in material quality. It can be mathematically shown that a decrease in the variance of log N, the designed for number of axles loads, decreases the possibility of failure provided mean strength remains constant. This implies that good quality control as well as sound construction practice is most important especially where calcrete base material is chemically stabilized. On the other hand the mean strength may be increased and greater variance be accepted with the same result. This philosophy of increasing mean strength can be followed especially where more difficult processes of construction such as bituminous stabilization is considered and also where reflection cracking from chemical stabilization is of no concern. One important aspect that needs consideration is the possibility of increasing the mean thickness of the top layer. Not only does this action increase the expected life log N, but also decreases the variance of log N with a resultant decrease in failure area.

Finally, a few important points need to be stressed :

1. Mechanical stabilization especially in the form of deep compaction by heavy equipment is beneficial to improve the strength characteristics of windblown sands.
2. Provided the materials are selected carefully, a mixture of sand and calcrete produces a material that can be used as base as well as aggregate for asphaltic concrete.
3. Unless the calcrete has a low soluble salt content, chemical stabilization is almost certainly a requirement in order to obtain a material of base quality under a thin surfacing.
4. Bituminous materials can be used with success in stabilizing the available borrow pit material provided the mixing-in process is of high standard or the mean strength is sufficiently high.

5. Asphaltic concrete manufactured from calcrete/ sand mixtures can be used both as a wearing course and as a base.
6. The variation in quality of material is as important as the mean strength in designing a successful pavement structure. At the present time the knowledge is available permitting the application of the variance and mean of pavement characteristics, such as strength and thickness, in design and construction control.

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