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Performance Study of Asphalt Road Pavement With Bituminous-Stabilized-Sand Bases

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The possibility of using the windblown sands that occur in the northern areas of South West Africa for the construction of all-weather roads to carry heavy truck traffic has been investigated. Laboratory investigations and field trials in Pretoria, South Africa, showed that bituminous stabilization of these sands was promising, and a full-scale road experiment to test a limited number of bases of bituminous-stabilized sand was constructed in the homeland of Owambo, South West Africa. This paper describes the laying of the experiment and the construction techniques and control measures used. A new technique that establishes the optimum time for the compaction of a cutback bituminous-stabilized sand mixture after aeration by using a vane shear apparatus is described. The vane shear apparatus was also used to measure the in situ shear strengths of the various experimental bituminous-stabilized sand bases after compaction and during service; the results of these measurements, together with performance data after 8 years service with respect to deformation and cracking, are discussed. Laboratory and field studies are described and predictions about the performance of a bituminous-stabilized sand base under varying traffic conditions are made by using the best known techniques available at this time.

Vast areas of the southern subcontinent of Africa are covered with a deep blanket of aeolian sand. Probably these sands were originally derived from preexisting sedimentary rocks in the general area and first emplaced by wind during the lower most Pleistocene epoch (approximately 2 000 000 years ago). They were subsequently redistributed by wind and water during the Pleistocene; the latest major redistribution was brought about by wind action, probably some 10 000 to 15 000 years ago, although some minor redistribution is still occurring (1).

Because of their widely spread occurrence, these sands, apart from various types of calcrete (caliche), are sometimes the only natural building material available to the civil engineer. From an economic point of view, they are therefore extremely important and have been studied for use in concrete structures, building construction, and, more recently, pavement construction by the National Institute for Transport and Road Research (NITRR) of the Council for Scientific and Industrial Research in Pretoria, South Africa (2, 3, 4).

This paper describes the use of these aeolian sands as the base layer of a road pavement in the recently proclaimed homeland of Owambo, in the northern part of South West Africa (SWA). It discusses the performance results of the experiment and relates these to the probable performance that might be expected under much heavier traffic on a normal freeway.

The accelerated development of the infrastructure of Owambo during the past decade necessitated upgrading the existing gravel road linking Owambo to the more developed, southern part of SWA to an all-weather, 8-m-wide, black-topped facility. The construction of the R60 000 000 ($84 000 000) hydroelectric facility at Ruacana Falls and other major building schemes in Owambo have resulted in a significant increase in heavy freight vehicles using this, the only surface transportation route to the south.

Initial laboratory work by the NITRR in the early 1960s showed that the most suitable method of improving the engineering properties of the in-place sand was to blend it with 15 percent calcareous filler (mechanical stabilization) and then to bind the blend with a bituminous binder. Both the hot-mix and cold wet-mix processes were studied; the latter was adopted as the more practical because of the length of road required and the problems associated with the establishment of a hot-mix facility in this remote area.

After extensive preliminary research into the wet-sand process of bituminous stabilization of fine-grained wind-blown sands, a full-scale road experiment was carried out in May 1965 in Owambo to test the techniques developed during the preliminary study (3, 4).

DETAILS OF EXPERIMENT

The experiment was designed and constructed with the following objectives:

1. To demonstrate in the field the feasibility of in situ bituminous stabilization of sand by using cutback binders and a cationic bitumen emulsion;
2. To investigate the stability and durability, under the traffic conditions and climatic environment of the site, of various bituminous-sand mixtures containing cutback bitumens, a cutback tar, and a cationic bitumen emulsion at binder contents considered suitable from

laboratory work and previous experimental trials;

3. To investigate the effect on stability of adding a proportion of calcareous filler to the natural sand before stabilization;

4. To investigate the effect of laying the bituminous-stabilized sand mixtures over a compacted sand-clay subbase of low strength [California bearing ratio (CBR) approximately 30 percent];

5. To investigate the relative performances of 76.2 and 152.4-mm compacted layers of bituminous-stabilized sand bases;

6. To obtain data on the temperature distribution throughout a bitumen-sand mixture on the road under the climatic conditions of Owambo; and

7. To study the setting up of bitumen-sand mixtures on the road over a period of 2 years.

The experimental pavement was constructed in Owambo from April 27 to May 26, 1965, on the alignment of the road route from Oshivelo to Oshakati. The experiment consisted of eighteen sections each 91.4 m long by 7.3 m wide (width of carriageway). The stabilized sand bases were supported by 3.7-m-wide shoulders on either side of the carriageway. The layout of the experiment with details of the base compositions and compacted thicknesses is given in Figure 1. All of the experimental sections, with the exception of sections 1 and 2, were laid over a firm foundation comprising a number of layers of approved subbase and base material consisting of silcrete and calcrete to which binding material was added. These layers were designed to obviate any deformation distress below the bituminous-sand base layer.

The area in the vicinity of the experiment consists of flat to slightly undulating plains with shallow, localized depressions that have been prospected for calcrete (5). As distinct from river channels, there is a network of shallow watercourses (oshanas). These watercourses drain the level country and do not reach the sea. The center of the drainage system is the Etosha pan. During years of good rainfall, large quantities of slowly flowing water are carried southward along this network of oshanas; this phenomenon is of great importance for the underground water of Owambo (6). The annual rainfall of the area is approximately 500 mm, but it is very variable and results in frequent droughts. Rain falls during a period of less than 60 d/year, mainly during January to March. The area is about 1000 m above sea level, has a subtropical climate, and is composed of open grasslands with scattered palms (Hyphaene ventricosa) in various stages of development in the area near Ondangwa; it changes to Mopani veld near the experimental site. In this region, Mopani (Colophospermum mopani) displays an interesting tendency to form large copses of even-sized trees varying from scrub bush 1.2 m high to large trees 6 to 12 m high (7).

Since the opening of the experimental sections, the traffic pattern has increased significantly, as was anticipated because of the large development program initiated in Owambo at about the same time as the experiment. The table below gives the traffic counts recorded over the 8-year period and the calculated equivalent 80-kN axle loads per day (8).

Date of Survey (year)	Vehicles per Day	Equivalent 80-kN Axles per Day
1965 to 1967	100	4
1971	147	6
1972	195	8
1973	226	9

This traffic is light by normal standards in developed countries, where the equivalent 80-kN axle loads per day would be expected to vary between 200 and 500 for freeways carrying medium to medium-heavy traffic.

MATERIALS

Windblown Sand

The local sand used for the stabilization was cohesionless and had an average particle-size distribution as given in Figure 2. The particle shape, as seen by microscopic examination, can be described as subrounded to subangular with relatively few well-rounded grains. The color is grey-white to a light reddish-brown caused by iron oxide stains on the grains. In the dry state, the sand has very poor inherent stability. Its apparent relative density is 2.60.

Blend of Windblown Sand and Calcareous Filler

The calcareous filler added to the windblown sand was selected from a natural powder-calcrete deposit in the vicinity of the experimental site. Its maximum particle size was generally 4 mm and approximately 50 percent passed a 0.074-mm sieve (Figure 2). The properties are as follows:

Property	Value
Liquid limit, %	50.9
Plasticity index, %	17.0
Linear shrinkage, %	8.0
Apparent relative density	3.61

The blend of windblown sand and 15 percent (by volume of dry sand) of calcareous filler was nonplastic, with an average particle size distribution as shown in Figure 2.

Bituminous Stabilizers

The following bituminous binders were used:

1. Cutback bitumens—A special cutback bitumen manufactured from an 80 to 100 penetration bitumen (MX) and a cutback bitumen manufactured from a 40 to 50 penetration bitumen and cutback to an intermediate grade of rapid-to-medium cure (250) were used. Both were straight-run bitumens, refined in South Africa from Middle East crudes (9, 10). The nominal binder contents were 4.0 and 6.0 percent (by mass of dry sand) for the 152.4-mm-thick compacted bases and 6.0 and 8.0 percent for the 76.2-mm-thick compacted bases (MX only).

2. Cationic bitumen emulsion—The cationic bitumen emulsion used was manufactured from an 80 to 100 penetration bitumen that was fluxed with 10 percent (by mass of emulsion) of a fluxing oil having a boiling point of 160°C. The base bitumen was straight-run and refined in South Africa from a Middle East crude. The nominal binder contents were 4.5 and 6.5 percent.

3. Cutback tar—The cutback tar was manufactured from a high-temperature coke-oven tar and cut back to a 30 to 35°C Evt grade. The nominal binder content was 6.0 percent.

All of the binders were tested with the windblown sand only and with a blend of wind-blown sand and 15 percent calcareous filler. The results of laboratory tests on samples of the binders used are reported elsewhere (4).

Borehole Water Used for Wetting Sand and Sand-Calcrete Filler Blend

The local borehole water used for wetting the sand and the sand-calcrete filler blend before the addition of the bituminous stabilizers had a total dissolved solids content of 52 800 ppm with an alkalinity of residue as carbonate of 3980 ppm.

TECHNIQUES OF CONSTRUCTION

A working platform was constructed to correct the longitudinal and transverse levels and ensure that, once the stabilized sand base was laid, the final road profile would be in accordance with the design requirements. Two 3.66-mm-wide shoulders were constructed to give a 7.32-m-wide trough into which the sand could be spread before treatment. The full experimental length of working platform was primed with a medium-cure (30) cutback bitumen at a rate of 0.72 L/m^2.

In situ CBR measurements were made on the primed platform. The length of platform constructed with calcrete had an average value of 142 percent (dry), and that constructed with sand-clay had an average value of 78 percent (dry) and 28 percent after 24 h soaking.

Where a blend of sand and calcareous filler was required, the correct quantity of sand was spread over the section, and then the calcareous filler was spread uniformly over the sand layer with a mechanical gritter. The sand and calcareous filler were mixed with a disc harrow and a motor grader to form a homogeneous blend.

Watering of Sand and Sand-Filler Blend Before Addition of Binder

The laboratory work carried out before the beginning of the experiment showed that water in the fine sand aids in coating of the sand particles by the cutback binder. It also showed that an initial excess of fluids gave a mixture that, after a specified period of aeration, had the highest density and shear strength when compacted (11).

For these reasons, the moisture content of the sand was increased to approximately 10 to 12 percent before stabilizing with binder. At this range of moisture content, after arbitrary compaction of the wet sand with a pneumatic-tired vehicle, the inherently poor stability of the dry sand improved to such an extent that the stabilization plant was able to move over the sandbed at the required speed without undue slippage.

The watering was done with a water tanker fitted with a gravity-feed spray bar. The moisture content of the sand was controlled by a nuclear gauge that was very useful in obtaining a rapid result. After the required quantity of water had been added, motor graders mixed the water into the sandbed to uniformly distribute the moisture throughout the sand layer.

Heating of Binders

The binders were supplied in drums. The contents of the drums were transferred to four 2300-L binder heaters, which were fired with liquid petroleum gas.

The heating of cutback binders is a fire hazard, and because of this, the temperature to which the cutback bitumens were raised in the binder-heating tanks was limited to approximately 5°C below that required for spraying. This procedure was satisfactory, and no fires occurred.

The heated binder was then pumped into a 4500-L distributor and further heated to a temperature that gave a Saybolt-Furol viscosity of 30 to 40 s, which was satisfactory for spraying.

Spraying and Mixing of Binder Into Wet Sand

The stabilization train consisted of a tractor, pulvimixer, and 1370-L binder-storage tanker. The 4500-L distributor pumped hot binder into the storage tanker during the stabilization process, thus enabling the work to proceed with a minimum of delay.

The binder was sprayed through a specially fitted spray bar located in front of the rotor of the pulvimixer. The depth of cut of the pulvimixer rotor blades was set

Figure 1. Diagrammatic layout of bitumen-sand stabilization experiment in Owambo, South West Africa.

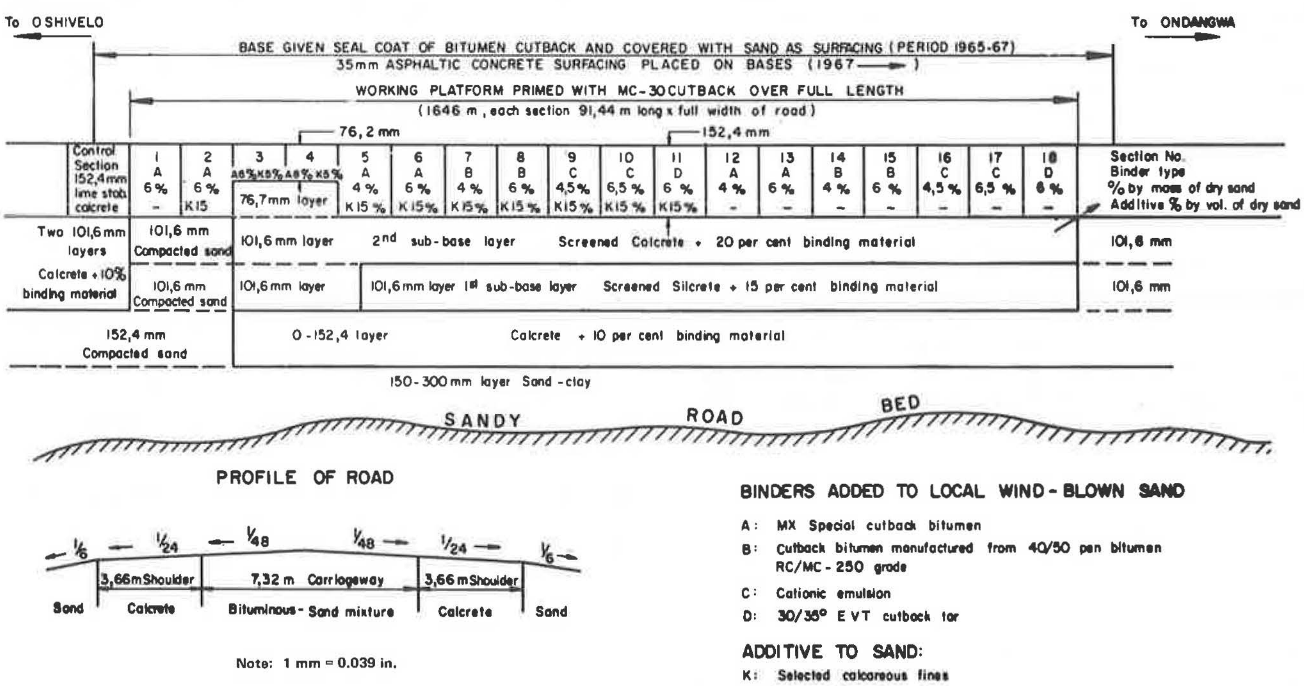
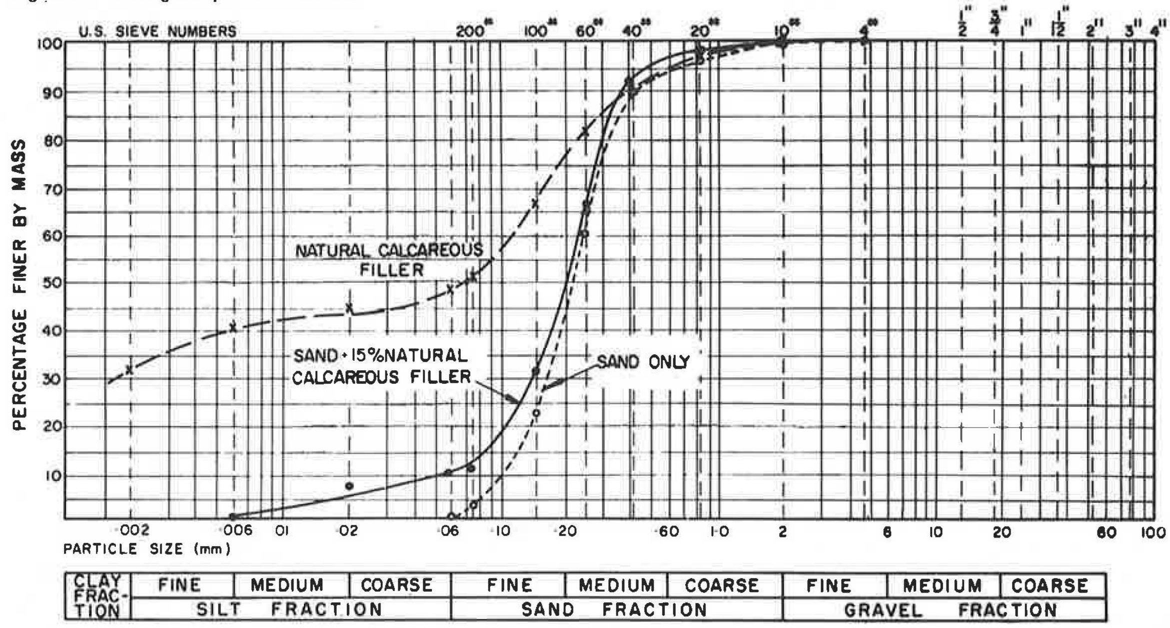


Figure 2. Grading analysis of materials.



to just touch the working platform, thus ensuring that the full depth of sand layer was mixed with the binder. The spray bar nozzles were adjusted so that the hot binder was sprayed just ahead of the rotor to obtain an intimate mix while the binder was still in a relatively fluid state. The distribution of the binder after this spraying pass was inadequate, and a second mixing pass was necessary to improve the overall distribution of the binder through the sand mass.

A single pass of the pulvimixer covered a width of approximately 2 m, which made five passes over the full width of the carriageway necessary to give an adequate overlap between successive passes.

Control of the quantity of binder introduced was achieved by calibrating the spray bar to give a known output at a particular pump speed and by operating the train at the forward speed required to give the desired binder content. The relative density of the hot binder was used to convert the amount sprayed from a volume to a mass basis. The forward speed of the mixing train was controlled by charts relating the true forward speed to the various tractor gears and engine revolutions that could be selected. The times taken over measured distances during each mixing run were determined with a stopwatch and, where necessary, adjustments were made to the forward speed. This method of speed control gave very satisfactory results.

At the outset of the experiment, it was intended to dilute the cationic emulsion with 30 percent (by volume of emulsion) of water so that it could be sprayed in a cold condition because a trial experiment had shown that the best distribution and coating were obtained with this dilution (after a spraying and mixing pass of the pulvimixer alone). The highly alkaline water available on the site, however, precluded any form of dilution because the acidic emulsifier reacted immediately with the water, which caused the emulsion to break. The viscous, cationic bitumen emulsion was therefore heated to lower its viscosity and sprayed in the same manner as the cutback binders.

Mixing of Stabilized Sand With Motor Grader

An inspection of the stabilized sand after the mixing operation with the pulvimixer showed that the coating of the sand particles was still not adequate. Further mixing with the pulvimixer would have delayed the progress of the work and so mixing by means of a motor grader was begun as soon as possible after the pulvimixer had completed the last mixing run. The high shearing action of the motor blade, producing a spiraling motion of the material during the cutting operation, was very effective in improving the coating of the sand particles. There was a significant improvement in the appearance of the mixture after each passage of the grader, and adequate coating resulted after two movements of the material from one side of the carriageway to the other.

Finally, the grader leveled the now homogeneous mixture to an even, loose thickness.

Aeration of Mixtures

The laboratory control measures used during this experiment indicated that aeration was essential to obtain the high stabilities required for the traffic that would use the road.

Controlled aeration was therefore carried out with a disc harrow pulled by a pneumatic-tired tractor. The discs were set to cut to the full depth of the loose layer. During aeration, the resistance of the mixture to the movement of the discs increased; this became evident when the tractor required more power to maintain a constant forward speed.

The aeration was continued until laboratory tests showed that the mixture had reached the condition at which maximum stability would be obtained on compaction.

Compaction of Mixtures

The most satisfactory compaction plant was a 30-Mg pneumatic-tired compactor (fully ballasted), used with a sheepsfoot roller. The sheepsfoot roller was fitted with a cleaning device to avoid excessive pickup of material during compaction. The individual feet measured

21 by 13.5 cm and were arranged in 24 rows of four, around the periphery of each roller drum. If it is assumed that four feet made contact at one time, the pressure per foot was 1.31 MPa when the roller drums were fully ballasted.

Various methods of compaction were tried, but since they all resulted in similar densities, the most practical method was chosen. This consisted of compacting in approximately 50-mm lifts by using the pneumatic-tired compactor and the sheepsfoot roller while a motor grader spread uncompacted material over the already compacted layer. It is important to obtain a good bond between the successive lifts, and the impressions left by the sheepsfoot roller assisted in this. Compaction was achieved by a continuous operation with a final leveling of each section with the motor grader. Each section was given a minimum of eight complete coverages with each compactor.

The sections containing calcareous filler and sand were compacted to such high stabilities that the motor grader had difficulty in trimming the material to the final profile required and, in some cases, an imperfect finish resulted. This difficulty was not experienced where only sand was stabilized, as these mixtures were more workable.

On completion of the experiment and before opening the sections to traffic, in situ density measurements of each section were made in duplicate by the sand replacement method. The densities obtained on the 152.4-mm-deep sections were fairly consistent, varying generally between 1794 and 1826 kg/m^3. Sections 3 and 4, which were 76.2 mm deep, however, had densities of between 1700 and 1715 kg/m^3, i.e., significantly lower than those for the 152.4-mm-deep sections.

Surfacing and Opening of Sections to Traffic

On completion of the stabilized base sections, the type of surfacing to be used to protect the base material was considered. Because of the high stability of most sections, it was decided that initially only a light sand seal should be provided; the position could be reviewed if the traffic caused serious rutting of the weaker sections.

Sufficient cutback bitumen binder was available on site for all of the surfacing and so a blend of equal quantities of the two types of cutback bitumen used for the stabilization work was used for the sand seal. The rate of application varied between 1.37 and 1.76 L/m^2. The application of the sand to the binder film was delayed to enable the cutback binder to penetrate into the stabilized sand base and to allow the viscosity of the binder remaining on the surface to increase, which gave a more stable surfacing layer. The same wind-blown sand that was used for the base stabilization was used for the surfacing and was spread at approximately 0.0065 m^3/m^2 and then well-rolled with a 15-Mg self-propelled pneumatic-tired roller.

Traffic was not allowed onto the sections until June 21, 1965, so that the sections laid toward the latter part of the experiment could set sufficiently and thus not be at a disadvantage during the initial trafficking. This light surfacing was overlaid with a 30-mm thick asphalt-concrete surfacing in mid-1967; it was therefore in service for a period of 2 years.

Placing of Level Pegs for Future Observations

Purpose-made pegs were placed in lines transversely across each section at distances of 30.5 and 61 m from the beginning of each section. The pegs were spaced at 304-mm intervals.

Level observations were made before trafficking and at regular intervals after trafficking. A number of permanent bench marks were installed so that all precise level observations could be compared in relation to a common datum. When the asphalt-concrete surfacing was placed in 1967, the level pegs were replaced in exactly the same positions as the original pegs and releveled.

LABORATORY CONTROL DURING LAYING OF EXPERIMENT

Sampling Mixtures

Representative samples of the mixed material were taken with a sampling tool that consisted of a 75-mm-diameter thin-walled aluminum tube that was introduced vertically into the loose mixture to make contact with the firm working platform. On withdrawal, the mixture remained in the tube and could be extruded into a suitable container. Samples were taken at the following times: (a) after blade mixing (before the beginning of the aeration) and (b) before compaction (after the aeration was complete). These samples were analyzed for the residual binder and fluid contents, and generally close agreement with the designed binder content was found.

A certain amount of intermixing of material from adjacent sections was unavoidable during construction; the first and last 15 m of each section were therefore not sampled.

Field Test to Establish When to Compact Mixtures

In the field, the strength of the mixtures produced could be evaluated by a vane shear apparatus. These tests also made it possible to establish when a particular mixture was in the optimum condition for compaction (11).

Accelerated Aeration of Field Sample

The technique followed was to first obtain a representative sample from the road as soon as the blade-mixing operation was complete. This sample was then divided into at least eight portions, each weighing approximately 6 kg. Each portion was placed in a tray of approximately 1-m^2 area and left to aerate in the sun. Gentle agitation of the mixture in the trays accelerated the aeration process. The material from each tray was tested after various periods of aeration by the following procedure:

1. The mixture from the tray was compacted in a CBR mold under modified AASHO compaction.
2. After compaction, the wet density of the compacted material was determined.
3. The compacted sample in the mold was introduced into the vane shear apparatus, which was fitted with a special base plate to retain the CBR mold.
4. The shear strength of the mixture was determined with the vane shear apparatus, and the temperature of the mix was measured at the middepth of the vane (Figure 3a).
5. After shearing, a representative sample of the mixture from the mold was analyzed for fluid content by evaporating the volatile oils and water.

The binder content of the mixture tested was determined from field measurements. The constitutions of the cutback binders and emulsion were known from previous laboratory tests in terms of residual binder and volatiles (oils or water) on a mass basis. From these data, it was possible to calculate the dry density of the compacted bitumen-sand mixture.

The dry densities and the vane shear strengths converted to 40°C were then plotted against the fluid contents. Typical results for mixtures with 6.0 percent (nominal) cutback bitumen (80 to 100 penetration base) MX, sand, and 15 percent calcareous filler (section 6) are given in Figure 4. Both the density and the shear strength pass through maximum values. To achieve maximum stability of the mixture, the highest possible shear strength and density should be obtained when compaction takes place. However, the peak of the shear strength curve always occurs on the dry side of the maximum density, so that it is not possible to obtain both maximum shear strength and maximum density at a particular fluid content. Eighty to 90 percent of the maximum shear strength and 95 to 100 percent of the maximum density were chosen as a suitable compromise for the sand used, and this criterion was used to establish the time when compaction of the mixture on the road should take place.

Field Aeration Control and Subsequent Compaction

Control of the aeration process on the road was achieved by taking regular samples from the road mix and testing these for shear strength and density. The shear strengths were converted to 40°C values by using factors determined from laboratory and field tests. The values for section 6 shown in Figure 4 indicate the good correlation between field and accelerated test values. The fluid content at which compaction was begun on the road is also shown.

Temperature Records at Experiment Site

A clockwork temperature recorder was installed during the experiment and again later for obtaining long-term temperature records at the site. Thermocouples were

Figure 3. Measurement of vane shear strength of bituminous-stabilized sand: (a) in field laboratory and (b) on pavement.

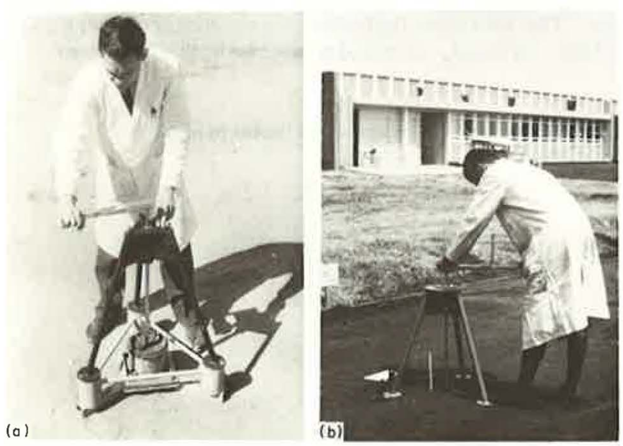


Figure 4. Dry density, vane shear strength, and fluid content relationships (section 6).

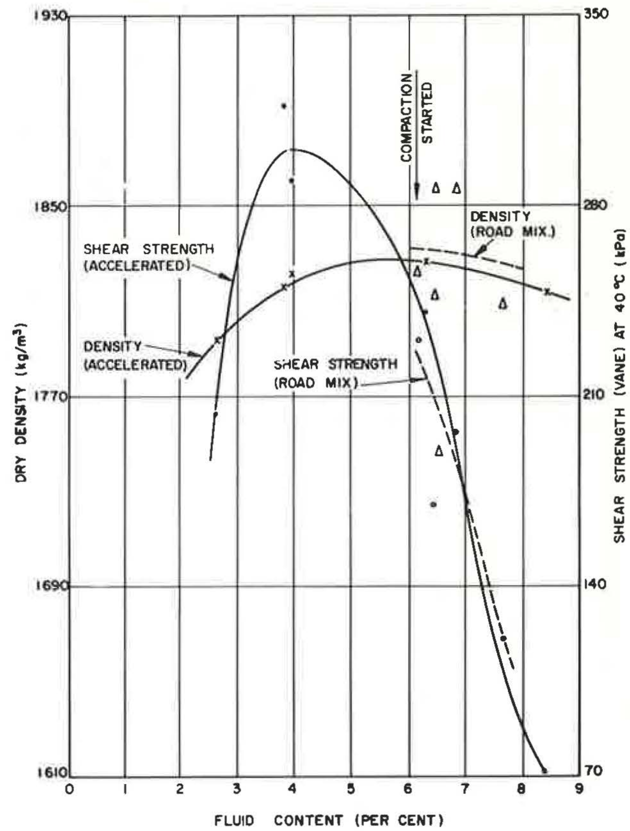


Table 1. Vane shear strength of experimental bituminous-stabilized-sand bases.

Stabilizer	Section No.	Binder Content (%)	Filler Content (%)	Vane Shear Strength (KPa) at 40°C After: Compaction	1 Year	2 Years
MX cutback bitumen	12	4.0	0	80	230	305
	5	4.0	15	125	346	415
	1	6.0	0	60	185	224
	13	6.0	0	60	178	255
	2	6.0	15	64	242	315
	6	6.0	15	90	286	405
Cutback bitumen [rapid-to-medium cure (250)]	14	4.0	0	68	240	318
	7	4.0	15	230	373	495
	15	6.0	0	54	210	305
	8	6.0	15	175	255	390
Cationic bitumen emulsion	16	4.5	0	50	170	235
	9	4.5	15	168	280	345
	17	6.5	0	25	180	242
	10	6.5	15	130	280	314
Cutback tar (30 to 35°C Evt)	18	6.0	0	44	150	226
	11	6.0	15	98	380	555

placed at depths of 0.75, 150, and 300 mm below the surface of section 6. Maximum and minimum road surface temperatures on a hot summer day in February 1966 were 70 and 24°C. On the same day, the maximum and minimum temperatures were 52 and 34°C at a depth of 150 mm.

FIELD STUDIES

Setting Up of Bituminous-Stabilized Sand Mixtures

To study the setting-up pattern of the bituminous-stabilized-sand mixtures, shear strength measurements were made on the sections with the vane shear apparatus (Figure 3b). These measurements were carried out at intervals after completion of the work, up to the time of laying the asphalt-concrete surfacings (1967), and all vane shear strengths were converted to values at 40°C (standard temperature).

The vane shear strengths of the mixtures for various periods of time are given in Table 1. All of the mixtures gained significantly in shear strength with time. This gain in strength was fairly linear over the 2-year period. On the average, the mixtures containing sand only increased in shear strength by a factor of approximately 5.3 and those with sand and calcrete filler by a factor of approximately 3.6, i.e., annual increases in shear strength of 225 and 125 percent respectively.

The change in binder content did not have a significant effect on either the shear strengths of the various mixtures or their rate of gain in strength with time. However, the addition of 15 percent powder calcrete to the sand had a significant effect on the shear strengths of the mixtures; mixtures with calcrete filler had shear strengths from 1.1 to 3 and about 1.5 times those of the sand-only mixtures after compaction and after the 2-year period respectively.

Structural Tests

Various structural tests were carried out on a selected number of these experimental sections toward the beginning of 1966; those results were reported by Gregg and others (13) and will therefore not be covered in this paper.

Deformation Measurements

The precise level observations for the 8-year service period have shown that, significantly, the deformation that occurred during the 6-year period after the laying of the 30-mm-thick asphalt-concrete surfacing was very much less than the deformation that occurred during the preceding 2-year period, particularly in those sections that deformed excessively (>10 mm), viz., sections 16 and 18.

To investigate whether there was any relationship between the laboratory, vane shear strengths of the mixtures at the time of construction and the subsequent permanent deformation (the average maximum rut depth between the inner and outer wheel tracks) measured after an 8-year in-service period, these results were plotted as shown in Figure 5. These data gave a reasonable envelope of laboratory, vane shear strength versus rut depth. The data also indicated which mixtures were satisfactory, critical, or unsatisfactory with respect to

Figure 5. Relation between vane shear strength and deformation under traffic after 8-year service of bituminous-stabilized-sand bases.

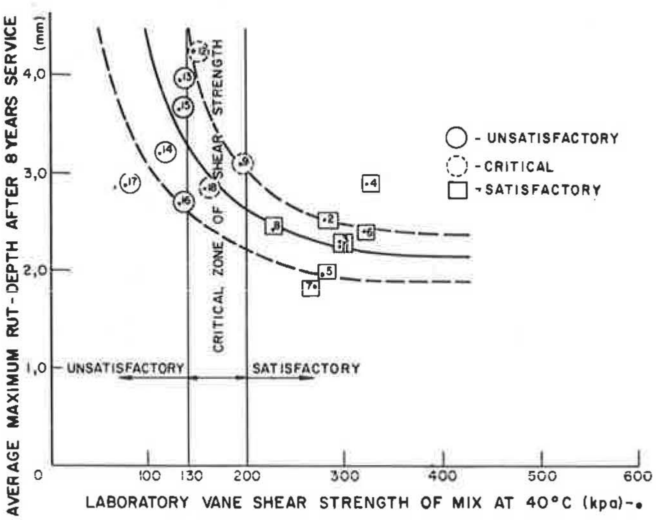


Figure 6. Crack indexes of sections.

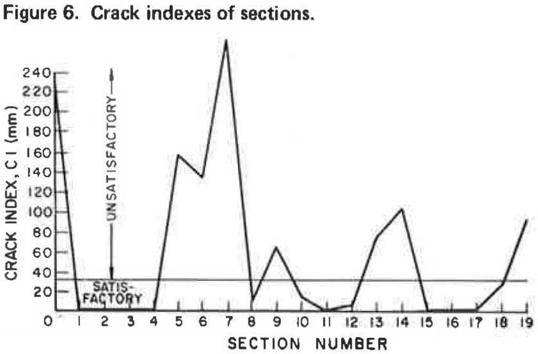


Figure 7. Extrapolation of measured rut depths to higher axle-load regimes.

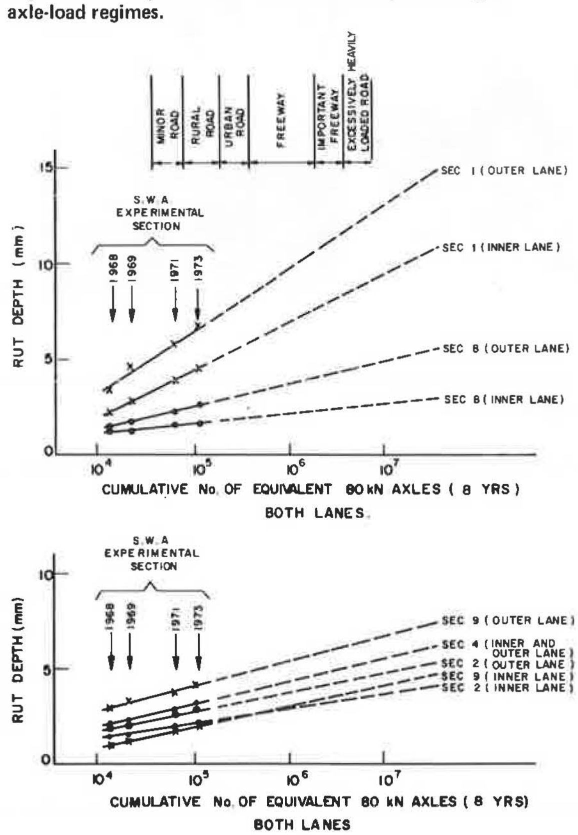
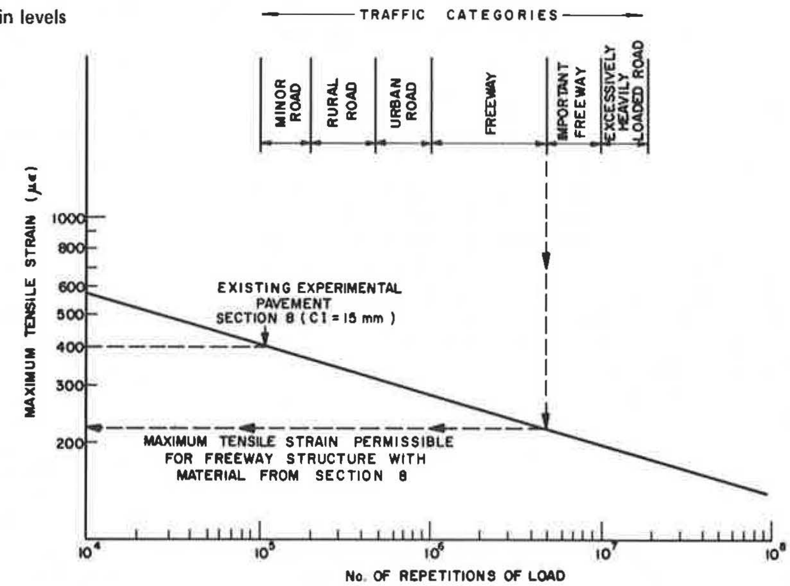


Figure 8. Prediction of maximum permissible tensile strain levels for different traffic categories.



permanent deformation: Those with laboratory, vane shear strengths between 130 and 200 kPa were regarded as critical, those above 200 kPa as satisfactory, and those below 130 kPa as unsatisfactory. The major external factors affecting permanent deformation on a particular bituminous material are traffic, rate of loading, and temperature, and it is not a simple matter to extrapolate the results from a given condition to a new situation if one or more of these major factors is different. However, techniques have recently become available for this purpose if the material properties are well characterized. These techniques were applied to this experiment in an attempt to predict the deformation likely to occur in similar materials under the heavier traffic conditions of a rural freeway; they are discussed later.

Cracking of Sections

A visual survey of the sections was made in 1973 (after 8 years service) to assess the extent of cracking on the different sections. A crack index (CI) (14) was developed for this purpose where

$$CI = \sum_{\substack{\text{all} \\ \text{patterns}}} (\text{percentage of area covered by cracks})(\text{average width of cracks in mm}) \quad (1)$$

The parameters of the spacing and the length of the cracks are not included in this definition. In the case of block and chicken-mesh cracking the area affected can usually be determined. However, a difficulty arises in the case of longitudinal and transverse cracks: When only a few such cracks appear, engineering judgment must be used to assess their effect on the pavement structure.

The CI gives an indication of the severity and area covered by cracks, regardless of the pattern. The CIs for the various sections, including the control sections (0 and 19) on which a lime-treated calcrete base was used, are shown in Figure 6.

It is interesting to compare the results shown in Figures 5 and 6 and to note the opposite trends in performance of some sections with respect to deformation and cracking; e.g., section 12 has the highest deformation and a very low CI and section 7 has a low deformation and a very high CI. This type of behavior is, of course, expected and points to the problem facing the highway engineer in selecting a suitable material that will give good performance in both deformation and crack resistance. Sections 2, 3, 4, 8, 10, and 11 performed acceptably and may be used with confidence for a pavement carrying similar traffic under a similar climatic environment.

CONCLUSIONS FROM EXPERIMENTAL PAVEMENT

In terms of the objectives of the experiment, it can be stated that

1. It is feasible to construct in situ bituminous-stabilized-sand bases that will give satisfactory performance for a highway pavement by using the following binders: (a) MX special cutback bitumen, (b) cutback bitumen manufactured from a 40 to 50 penetration bitumen [rapid to medium cure (250)], (c) cationic emulsion, and (d) cutback tar (30 to 35°C Evt grade), [if the sand is mechanically stabilized by the addition of 15 percent powder calcrete (calcareous filler)];
2. The two 76.2-mm-thick bituminous-stabilized-sand sections (3 and 4) gave excellent performances and stand out as the most successful sections laid, from the points of view of both economy and performance;
3. The performance of the section containing a blend of calcareous filler and sand laid on a sand-clay subbase (section 2) was acceptable, and for similar traffic conditions, this type of structure is worth serious consideration because it would be economically advantageous; and
4. All of the bituminous-stabilized-sand bases increased in shear strength with time (for the 2-year period that measurements were made).

EXTRAPOLATION OF PERFORMANCE RESULTS TO HEAVIER TRAFFIC CONDITIONS

The design and particularly the construction of existing pavements have already provided a wealth of experience and information. The extrapolation of analyses of existing material to new pavements should not be neglected simply because the test methods used are considered inadequate to characterize the pavement by current standards. In general, extrapolation has not been widely used, primarily because new and more advanced methods of testing and evaluating materials are continually being developed.

An alternative approach is to use the results of in situ tests to characterize existing pavements as well as possible. These data can then be extrapolated to other conditions on the clear understanding that the extrapolations are guidelines with which engineers can supplement their existing knowledge. In this context, existing experimental and prototype test pavements are normally the best to use, because a number of variations are usually incorporated in the experiment and a well-defined program of testing is carried out. The experimental sections described in this paper are an example of this type. One of the main aims of these experimental sections was the determination of the behavior of bituminous-stabilized-sand bases under the environmental and traffic conditions pertaining to this road.

Pavement Deformation

Precise level determinations of permanent deformation were made for each of the experimental sections of the road. These measurements were originally made in 1967 to establish a datum line and then repeated in 1968, 1969, 1971, and 1973. The maximum rut depths in the inner and outer lanes were determined from the measurements. Semilog plots of rut depth versus cumulative number of equivalent 80-kN axle loads for some of the experimental sections are shown in Figure 7.

The amount of heavy traffic on these sections was small, about 0.1 million equivalent heavy axles over the 8-year test period. However, the few heavy axles borne by the section during this period were usually loaded more than the legal limit.

In Figure 7, the rut depth has been extrapolated to a higher number of 80-kN axles by using a linear-log relationship (14). From this relationship between rut depth and number of equivalent 80-kN axles, it appears that the base mixtures used would give reasonably acceptable values of rut depth even under freeway traffic conditions (i.e., 1 million to 10 million equivalent axles).

The conditions under which the extrapolation will remain valid are

1. That no major variations in the environmental conditions occur, other than those that have already occurred over the previous 8 years,
2. That the behavior of the bituminous materials does not go into the shear failure zone, and
3. That the deformation behavior of the bituminous materials used is not as dependent on their stress history as is that of, e.g., unbound granular materials.

The first condition is perhaps the most unpredictable. There are also other unpredictable factors, e.g., cracking of the surfacing and base, which could allow ingress of surface water to the subbase and result in excessive deformation.

Three zones can be identified in the deformation behavior of bituminous-based mixtures to repetitions of load. In zone 1, there is initial deformation, but no further increase; in zone 2, the deformation increases at a constant rate; and in zone 3, excessive shear begins to take place.

The conditions under which zone 3-type deformation will occur can be studied by investigating the cohesion (c) and the angle of internal friction (ϕ) of the material under the appropriate test conditions.

A study of this nature was carried out by using an extreme case of dual wheel loading with double the legal axle load (2 times 40 kN). Under these conditions, the base material should have a c-value greater than 35 kPa and a ϕ-value greater than 27° (at 40°C) to prevent zone 3 deformation. An analysis of the stress states of the 150-mm-thick bituminous layer showed that, except for a small area under the wheel loads at the 150-mm depth, the material remained in the satisfactory zones of deformation. This condition is not considered important from a practical point of view because of the extreme loading conditions chosen.

Fracture (Fatigue Cracking)

It is also desirable to estimate the cracking that can be expected if a similar base mixture is used in another pavement with different loading conditions. At present, the degree of cracking is known for specific sections of the SWA experiment after 8 years of trafficking. Section 8, for example, has a CI of 15 mm, which means that 15 percent of the area has cracks 1-mm wide, after 100 000 load repetitions. This degree of cracking is acceptable even for freeways. However, to achieve this level where there is a greater volume of equivalent 80-kN axles, the maximum tensile strain at the bottom of the base layer must be reduced. The amount by which it must be reduced can be determined from the fatigue behavior of the material, as shown in Figure 8. From knowledge of the tensile strain computed for the conditions at the pavement section (in the case of section 8, 400 microstrains) and of the traffic, a point on the strain-versus-load-repetitions (ϵ-N) line can be established. The slope of the (ϵ-N) line representing the material used for the base was taken to obey the formula given by Brown and Pell (15). This relationship can be determined either by measurement or by the average values used (14, 15). In Figure 8, the average value for the slope of the line was used. In the case of section 8, the maximum permissible strain level is approximately 210 microstrains.

A reduction in strain at the bottom of the base can be achieved either by increasing the modulus of the subbase (i.e., by increasing the CBR or by stabilization) or by increasing the thickness of the base layer. Structural analysis using elastic theory is an effective method of determining the upgrading of the pavement that is necessary to avoid fatigue cracking of the base material.

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Ponding an Expansive Clay Cut: Evaluations and Zones of Activity

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The use of ponding water on a clay subgrade of high swelling potential to cause soil heaves before pavement placement was successful on an expressway project outside San Antonio, Texas. Expansive soils are a worldwide problem and cause over $2 billion damages/year in the United States. Their effectiveness of controlling the clay was measured, and the depth of the movements was determined. Observations began in 1968 and continued through 1976, both inside and outside the ponded area. The elevation rods were set at depths of 0.6 to 5.8 m (2 to 19 ft), and the moisture measurements were taken in the same zones. The ponding generally resulted in an upward movement of the elevation rods. The maximum movement was that of the shallower set rods in all areas. It now exceeds 0.12 m (0.42 ft) in the area where the predicted vertical rise is 0.15 m (0.5 ft). The moisture variations were greatest at depths up to 3 m (10 ft), where the rods exhibited maximum movement. This was the zone of activity. Pavements in the ponded areas have shown less distress and less major cracking and have required less major maintenance work than those in the nonponded areas. The relation of rainfall measurements to rod movements is not definitive. A trend may be developing that shows upward movement to follow rainfall after prolonged dry periods. Ponding does seem to help curb the destructive movements of expansive clays.

Expansive soils are an international problem. They are also expensive problems, costing the United States more than $2.2 billion in 1973 (1). They have been studied extensively (2, 3, 4, 5, 6, 7). In Texas, expansive clays extend in a corridor from the northern borders with Oklahoma and Arkansas almost to the Rio Grande River and Mexico in the south. They usually lie along the Wichita Falls, Dallas, Fort Worth, Austin, San Antonio line. The clays present fewer problems in the eastern areas of the state, where higher rainfall rates tend to keep subsoil moistures uniformly higher. In the areas west of San Antonio and Austin, the Balcones escarpment and the upland area contain more limestone and less expansive clay.

In 1968, the Texas Highway Department, now the State Department of Highways and Public Transportation, began planning improvements to US-90 west of San Antonio. This highway lays over an expansive clay area, and multiple control measures were considered. Ponding and the installation of a lime-treated moisture seal and an underdrain were chosen. The department, in cooperation with the Federal Highway Administration and the Center for Highway Research at the University of Texas, also decided to measure the effectiveness of these techniques, and to find the depths at which the movements of these clays and the moisture changes take place. The work was initially reported as part of the center's series on expansive clays (8).

SITE AND ITS GEOLOGY

US-90 is a transcontinental route. It passes through the San Antonio area where the movement of expansive clays has long been observed. This highway project, where the ponding was used, begins at I-410 on the southwest