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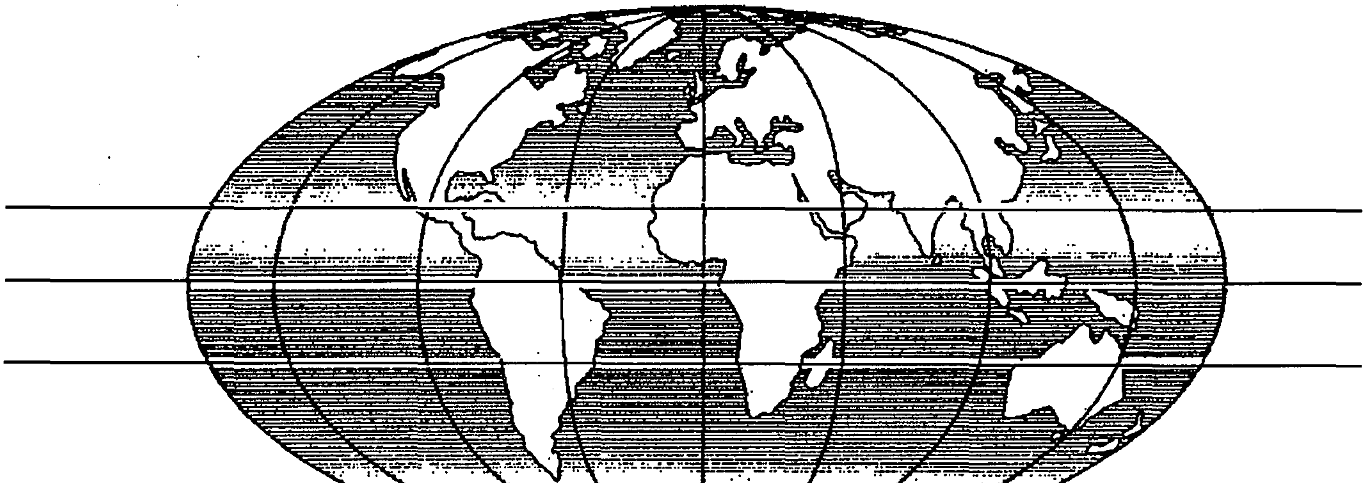

Overseas Centre
Transport Research Laboratory
Crowthorne Berkshire United Kingdom

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Experimental use of cinder gravels on roads in Ethiopia

D.Newill & R.Robinson
Transport and Road Research Laboratory, Crowthorne, Berkshire, UK
Kassaye Aklilu
Ethiopian Transport Construction Authority, Addis Ababa

SUMMARY: A full scale experiment has been carried out in Ethiopia to examine the performance of volcanic cinder gravels as the surfacing material for unpaved roads and as the road base under bituminous surfaced roads.

Compaction trials were carried out to determine the type of plant to be used and an experimental road comprising 20 different sections was then constructed. Six sections were left unsurfaced and were monitored for 28 months during which they carried approximately 140,000 vehicles. A bitumen surface was provided for the remaining 14 sections and these carried 150-200 vpd for 7½ years giving a total of 440,000 esa in one direction. Monitoring was carried out by taking quantitative measurements of the performance of the road pavement throughout this period. As a result of the study, recommendations are made for the use of cinders in both paved and unpaved roads.

For unpaved roads, recommendations are made for a particle size distribution which provides a road surface that is resistant to corrugations. Improved performance can be obtained by mechanically stabilising cinders with plastic fines.

For paved roads, it is concluded that the types of materials used in this experiment are all capable of carrying in excess of 400,000 esa when sealed with a surface dressing and designed according to Road Note 31. Roadmixed asphalt is not a suitable surfacing for cinder gravels. In addition to the cinders, other materials also performed satisfactorily including a dry bound macadam, an agglomerate and a tuff. Cinders are easier to compact when they are mechanically stabilised with 10 per cent of volcanic ash soil.

1 INTRODUCTION

The Joint Road Research Project of the Ethiopian Transport Construction Authority and the Transport and Road Research Laboratory (UK) has been studying various low-grade materials to see if effective specifications can be devised for their use on lightly-trafficked roads.

The work began with an investigation of volcanic cinder gravel which, although widespread in Ethiopia, had only occasionally been used for road construction. The reason for its limited use was mainly because these gravels tend to be deficient in fine material and do not conform to the generally accepted grading specifications. In addition, they were reportedly difficult to compact.

The study, aimed at a full examination of cinder gravels, comprised the following stages:

(i) A field survey to locate and identify cinder gravels and to obtain samples for laboratory testing; the survey also included the examination of aerial photographs and photo-mosaics and the preparation of a map showing the distribution of cinder deposits throughout Ethiopia.

(ii) A laboratory investigation to determine their physical and engineering properties.

(iii) An examination of existing cinder gravel roads to compare the performance of the materials under trafficking with the results of the laboratory investigations.

(iv) Compaction trials to determine the most suitable plant for practical use.

(v) A full-scale experiment to examine the behaviour of different cinder gravels under controlled conditions in relation to traffic and climate.

The first three stages of the study were discussed in an earlier paper (Newill and Kassaye 1980) and the final two stages are described here.

Reference was also made to other studies of cinder gravels and principally those carried out in the USA (Hendrickson and Lund).

In the compaction trials four cinders from different sources were examined with and without plastic soil fines added and with and without water added.

The full-scale experiment consisted of 20 sections of road built on a standard sub-base and subgrade. Each section had a road base of different types of volcanic material with the exception of one crushed stone base used for control. Eleven sections were surfaced with a double surface dressing, three were surfaced with 50 mm of roadmixed asphalt and the remaining six sections were unsurfaced. The unsurfaced sections were monitored for two years after which time they were surface dressed. After one year, several of the unsurfaced sections were reconstructed. The surfaced sections were monitored for 7½ years. During this time, one of the road mixed sections was surfaced dressed, but no other pavement maintenance was carried out.

2 DEFINITION OF VOLCANIC CINDER

Volcanic cinders are pyroclastic materials associated with recent volcanic activity. They occur in characteristically straight-sided cone-shaped hills which frequently have large concave depressions in their tops or sides where mixtures of solids and gases were released during the formation of the cone. Cinders vary in colour, often within the same cone and may be red, brown, grey or black. The cinder particles also vary in size from large irregularly shaped lumps 0.5 m in diameter to sand and silt sizes. In some cones, however, particles may be more uniform with the largest size not exceeding 30 mm in diameter. Other characteristic features of cinders are their light weight, their rough vesicular surface and their high porosity. Usually they are weak enough to be crushed under the heel. An advantage as a road construction material is the relative ease with which they can be dug from the quarry; a mechanical shovel or hand tools are usually adequate for their extraction although, occasionally, a bulldozer may be required to open up a working face.

3 PRELIMINARY INVESTIGATIONS

The main conclusions from the preliminary investigation of cinder gravels which covered a field survey, a laboratory study and an examination of a cinder gravel road, are given below (Newill and Kassaye 1980):-

(i) Cinder gravels are more widespread in Ethiopia than was originally believed; this showed the value of using aerial photographs in survey work and enabled a preliminary map to be prepared giving the distribution of cinder cones.

(ii) In order to obtain representative material from a cinder cone, it is important that samples are taken from below the weathered zone which can extend to a depth of two metres.

(iii) Although 'as dug' cinder gravels do not meet the recommended grading requirements for road base materials, the laboratory investigation revealed that, because of the weak nature of the aggregate particles, breakdown under compaction occurred with an improvement in both grading and strength properties.

(iv) In the laboratory investigation, the cinder gravels were not affected by changes in moisture and even complete immersion in water only reduced their strength slightly.

(v) The addition of locally available plastic volcanic ash soil, to make up for the deficiency of fine material in the grading, improved the mechanical stability of cinder gravels and indicated that this could be a valuable construction practice. However, unlike the natural cinders the mixed materials lost some of their strength when they were saturated with water.

(vi) The gravel road study confirmed that an improvement in the grading and the strength of cinder gravels occurred under normal road conditions even when trafficking was used as the means of compaction.

The results from the preliminary investigations indicated that cinders could provide useful road construction materials especially for gravel roads. However, it was necessary to carry out further work under known conditions of traffic and climate in bituminous surfaced roads, as well as in gravel roads, before limits could be recommended for their various uses. It was therefore decided to construct pilot scale compaction trials and then a full-scale road experiment to examine these aspects further.

4 COMPACTION TRIALS

4.1 Organisation of the trials

Because doubts had been expressed about the ability to compact cinder gravels in normal road construction practice, a series of pilot-scale compaction trials were under-taken before carrying out full-scale road experiments. Earlier laboratory tests had shown that, for typical cinder gravels, repeated compaction caused the breakdown of cinder particles to give an improved grading and an increase in density. An improvement in grading and better compaction could also be obtained by mechanical stabilisation with the volcanic ash soils which are normally found close to cinder deposits. It was also shown that, unless subjected to repeated compaction, the densities obtained were not affected by changes in moisture content. For these reasons, the compaction trials were designed to examine the following conditions:-

(i) crushed and uncrushed materials.
(ii) with and without fines added.
(iii) with and without water added.

Four different cinder gravels were used in the trials. Of these, two, one from Mojo and one from Bekojo, were classed as typical cinders, but the strengths of their cinder particles were different. Another, from Nazret, had relatively uniform sized cinder particles, whereas the fourth, from km 130 on the Awash Malkassa to Assela construction project, was unusually a well-graded material with a higher proportion of fine-sized particles. Description and properties of the materials are shown in Table 1 and Fig 1. The volcanic ash soil used for mechanical stabilisation was a clayey silt with a plasticity index between 8 and 12.

Table 1. Cinders used in compaction trials

Cinder gravel - source	Modified aggregate impact value - per cent	Description
Mojo	140	Typical black cinder
Bekojo	93	Typical red cinder
Nazret	90	Single-sized cinder
Km 130 Awash-Assela Road	59	Well-graded cinder

Different types of compaction plant were used in the trials. They comprised a Galeon Roll-o-Static 10 tonne smooth-wheeled roller, a Bomag BW 200 7 tonne vibrating roller and a Hyster C530A pneumatic-tyred 10 tonne roller. They were used singly and in different combinations

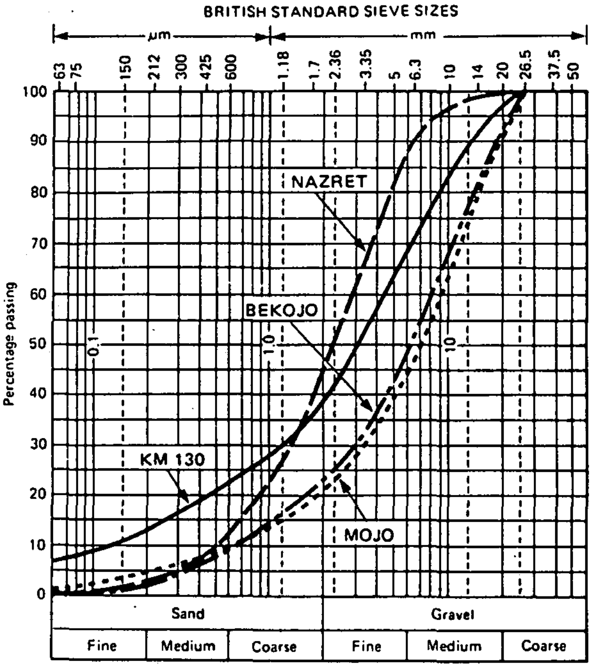


Fig.1 Particle size distributions of cinders used in compaction trials

to determine the most effective way of compacting the cinders.

The site for the trials was at the Mojo cinder quarry on a cleared area of the quarry floor. The other materials were imported and stock-piled at the site. For the comparison between crushed and uncrushed cinders sufficient quantities of crushed material were prepared by spreading thin layers of 100 mm of cinder on a specially constructed concrete slab and compacting with the vibrating roller. The individual pilot-scale compaction trials were carried out on the materials laid in loose layers 200 mm thick in sections 15 m long and 3 m wide. After compaction, the densities were measured using the sand replacement density test and the means of three tests for each trial are those given in Table 2 where they can be compared with the maximum densities obtained in the standard laboratory compaction tests (BSI 1975).

4.2 Results of the compaction trials

The compaction trials showed that:-

(i) Of the four cinders used, the Mojo and Bekojo cinders behaved similarly and

Table 2. Compaction trials on cinder gravels with different treatments

Type of cinder	Type of compaction	Cinder – as dug		Cinder + water		Cinder + fines + water	
		Density Mg/m^3					
		Uncrushed	Crushed	Uncrushed	Crushed	Uncrushed	Crushed
MOJO	Vibratory roller, 12 passes	0.99	1.22	–	–	–	–
	Pneumatic tyred, 2 passes + smooth-wheeled, 8 passes	1.09	1.12	1.09	1.17	1.07	1.26
	Pneumatic tyred, 2 passes + smooth wheeled, 16 passes	1.12	1.27	1.12	1.30	1.11	1.26
	Laboratory 2.5 kg rammer	–	–	1.03	1.25	1.11	
	compaction 4.5 kg rammer			1.30	1.63	1.30	
BEKOJO	Vibratory roller, 12 passes	1.09	1.15	–	1.23	–	–
	PTR, 2 passes + smooth-wheeled, 8 passes	1.12	1.31	1.26	1.36	1.12	1.30
	PTR, 2 passes + smooth-wheeled, 16 passes	1.17	1.35	1.35	1.39	1.17	1.33
	Laboratory 2.5 kg rammer	–	–	1.22	1.38	–	–
	compaction 4.5 kg rammer			1.44	1.73		
Km 130	Vibratory roller, 12 passes	–	–	1.25	1.28	–	–
	PTR,2 passes + smooth-wheeled, 8 passes	–	–	1.28	1.35	–	–
	PTR, 2 passes + smooth-wheeled, 16 passes			1.35	1.36		
	Laboratory 2.5 kg rammer	–	–	1.43	–	–	–
	compaction 4.5 kg rammer			1.49			

the other two behaved very differently:

(ii) The relatively uniform-sized Nazret cinder could not be compacted satisfactorily with any of the rollers in the crushed or as-dug condition, with or without water added. Although compaction could be achieved with the mechanically stabilised mixture the Nazret cinder was regarded as unsuitable for consideration as a road base material and therefore no test results are reported.

(iii) The well-graded km 130 cinder was little changed by the preliminary crushing and had sufficient fines to give a tight well-knit finish in the compacted layer. It was not necessary to add further fines for mechanical stabilisation and only the compaction results with water added are reported.

(iv) For the Mojo and Bekojo cinders in their natural condition, it was necessary to start compaction with the pneumatic tyred roller working from the outside of the section towards the inside in order to prevent the materials being displaced sideways. With the vibrating roller, which was lighter than the smooth-wheeled roller, the first two passes were made without vibration.

(v) For the Mojo and Bekojo cinders, higher densities were obtained with the crushed material than with the uncrushed. Densities of the as-dug dry materials were not noticeably changed either by the addition of water or the addition of fines. However, the addition of fines made compaction easier and produced a tightly-bound surface which could not easily be scuffed or damaged by brushing with a broom. The addition of water also gave a better surface finish especially as the material dried.

(vi) In the compaction trials, the densities achieved with the combination of the pneumatic-tyred and smooth-wheeled roller were generally higher than with the vibrating roller. This was probably because of the greater mass and higher energy input of the roller combination. It is worth noting that the compaction plant used in the trials was that being used in the construction of the Awash Melkassa to Assela road. It was lighter than that often used in road works and it is likely that better results could be obtained with heavier plant.

(vii) The highest densities achieved in the trials were generally close to those obtained in the standard 2.5 kg rammer laboratory test. It should be noted, however, that for typical cinders like Mojo and Bekojo there was a large difference in the maximum densities between the 2.5 kg and the 4.5 kg rammer tests. This means that normal specifications for compaction of road base materials, which are usually related to the heavier laboratory compaction test, are unlikely to be achieved. Clearly, this is because greater breakdown of particles occurs in the laboratory test which is conducted in a confined mould. An alternative specification would therefore be required for cinder gravels which would have to consider the relation between density and breakdown of particles. It is recommended that, for typical cinder gravels used in road construction, compaction trials should first be carried out to establish the method of compaction and the target density to be achieved. If a 'method' type specification is adopted, this would also help to reduce the number of difficult in-situ density tests that have to be made.

It was concluded, therefore, that typical naturally-occurring cinder gravels can be compacted in practice, but care is needed in selecting the most suitable plant. For the full-scale experiments described in this paper, it was recommended that 2 passes of a pneumatic-tyred roller followed by 12 passes of a smooth-wheeled roller should be used.

5 FULL-SCALE EXPERIMENT

5.1 Objectives of the experiment

A site for the full-scale experiment was offered by the Ethiopian Transport Construction Authority on the Awash Melkassa-Assela road. This 80 km long gravel road, consisting of an old three metre wide telford base which had subsequently been widened to six metres, was being upgraded to bitumen surfaced standard by complete reconstruction on the old alignment. The whole length of road was traversed and subgrade investigations were carried out at two possible experimental sites. The one eventually chosen was six kilometres south of the town of Dhera where the main camp of the construction project was situated.

The objective of the experiment was to compare the performance of a variety of cinder gravels under the same conditions of subgrade, sub-base, climate and traffic to assess the most effective way that they could be used as road building materials. To achieve this, the cinders were laid as a gravel wearing course in addition to being used as base material under a surface dressing and under a 50 mm roadmix surfacing. The cinders were laid as 'pit-run' material and were also mechanically stabilised with a volcanic ash soil of a clayey silt texture which was the natural subgrade material in the area of the experiment. On the gravel sections, both compacted and uncompacted cinder gravels were laid. In two of the sections constructed with a surface dressing, the 'as dug' cinders were initially passed through a crushing plant to produce an improved grading before compaction was carried out.

The roadmix surfacing was used because it was thought that bases constructed with 'as dug' cinder gravels might not have sufficient cohesion to withstand deformation by trafficking if only a relatively thin surface dressing was used. Duplicate sections were constructed, therefore, to compare the performance of cinder gravel bases with two types of surfacing.

Subsidiary experiments examined the performance of other volcanic materials.

5.2 Pavement design

The soil at the experimental site was a uniform fine volcanic ash with a plasticity index between 8 and 15. This gave a subgrade with a soaked CBR of 7 per cent. To obtain uniform subgrade conditions over the whole of the experiment, it was decided to remove the existing telford base, and this was done with a bulldozer and a grader.

A preliminary traffic count estimated that, of a total ADT of about 200, 80 to 100 commercial vehicles per day were using the road in both directions, and an axle-load survey already carried out by the Joint Research Project on other roads indicated that the number of equivalent standard axles per commercial vehicle for Ethiopia was 2.33. Because of the economic situation in the country, a traffic growth rate of zero was assumed. Thus, for a 15 year design life, it was predicted that the road would carry:

$\frac{100}{2} \times 365 \times 15 \times 2.33 = 0.64$ million equivalent standard axles.

The basic pavement design of the experiment was calculated using Road Note 31 (Transport and Road Research Laboratory 1977). The cumulative axle loading coupled with a 'soaked' subgrade CBR of 7 per cent gave a design thickness of 150 mm of base with 150 mm of sub-base. The thickness of the experimental construction was kept uniform throughout, irrespective of the different kinds of material being used for each section.

The design thickness of the experimental sections contrasted with the main contract which was 200 mm of sub-base laid in two layers with 150 mm of crushed rock base.

5.3 Layout of experimental sections

The cross-sectional profiles were kept the same as those on the main contract with a sub-base width of 10 metres covered by a 6 metre base and 2 metre wide shoulders (Fig 2). The base width was increased to 8 metres on the unsurfaced sections and the shoulders reduced to one metre.

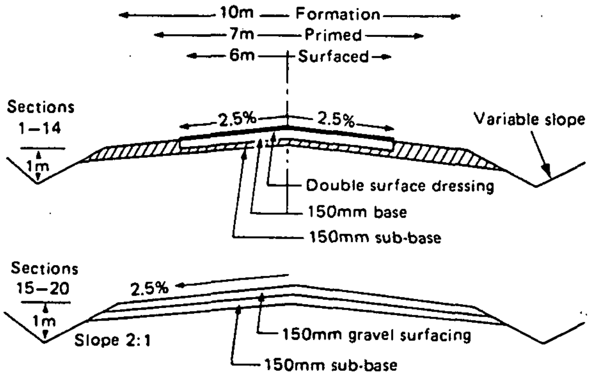


Fig.2 Cross-section profiles of the experimental sections

The experiment was divided into three parts: eleven 60 metre sections were surfaced with a double surface dressing using single sized chippings (BSI 1971); three sections, one 60 metres and two 120 metres in length, were surfaced with ETCA standard roadmix asphalt; six 100 metre sections were left unsurfaced as a gravel road. With traffic levels of the order of 200 vehicles per day, the experiment provided the opportunity for accelerated testing of unsurfaced materials.

The major part of the experiment was to compare three different types of cinder representing a wide range of particle strengths: Mojo, Bekojo and km 130 (see Table 1). The cinder from km 130 was also used as the common sub-base for the experiment as it differed from any other cinder sampled in being better graded and having a much higher percentage of fines. The layout of the experimental sections is shown in Table 3.

The compaction trials indicated that it might prove difficult to produce a tightly-knit surface on some of the sections that were subsequently to be primed and surfaced. Fines were therefore added to most of these cinder sections to help provide the necessary cohesion. Both clay fines, mixed during laying, and quarry dust, vibrated into the surface after laying, were used. One section was laid without the addition of fine material.

Two other materials which did not meet the ETCA specifications for road bases were also included among the sections to be surface dressed. These were a volcanic tuff and a volcanic agglomerate. The former had been used as sub-base on the Addis Ababa-Nazret road and the latter was being used as the sub-base material on the main contract. To demonstrate a technique of construction for use in arid areas, a section of dry-bound macadam was included and laid in two 75 mm layers. This used a single-sized (50-60 mm) aggregate with quarry fines vibrated into the voids. A control section of crushed rock, as used for project base, was also included.

Three of the cinder sections were also repeated under the roadmix surfacing.

Of the six sections constructed with gravel surfacings, four were laid in the same way as the base in the bitumen-surfaced sections. The other two sections comprised, material laid 'as dug' with compaction being left to subsequent trafficking which was the form of construction used on the low-cost road schemes inspected during the first stage of the project (Newill and Kassaye 1980).

5.4 Materials properties

Particle size distribution charts for the sub-base used in the experiment are shown in Fig 3 and charts for the materials used for road base under paved surfacings are shown in Figs 4-6. Gradings of the material used for gravelling the unsurfaced test sections are shown in Fig 7.

Measurements of in-situ density and moisture content of the road base materials were recorded at the time of construction, as were in-situ subgrade CBR values.

Table 3. Experimental sections

Section	Length (metres)	Surfacing	Material	Mean dry density (Mg/m³)	Moisture content %	Per cent compaction (BS heavy)
Subgrade	–	–	Volcanic ash	1.49	15.2	103
Sub-base	–	–	Km 130 cinder	1.49	13.5	94
1	60	Double surface dressing	Km 130 cinder	1.53	14.4	97
2	60		Km 137 crushed stone (Rhyolite used as base material on main contract)	1.99	7.2	97
3	60		Dry-bound macadam	–	–	–
4	60		Sodere agglomerate	1.90	8.9	102
5	60		Mojo cinder + fines	1.49	9.1	103
6	60		Bekojo cinder + fines	1.43	10.4	101
7	60		Bekojo + quarry fines	1.41	10.2	100
8	60		Crushed Bekojo + fines	1.59	14.7	113
9	60		Crushed Mojo + fines	1.50	16.4	104
10	60		Mojo cinder	1.34	14.6	103
11	60		Nazret tuff	1.39	20.8	93
12	60	Road mix	Bekojo cinder + fines	1.45	11.6	103
13	120		Modjo cinder + fines	1.45	13.8	101
14	120		Km 130 cinder	1.48	16.8	94
15	100	Unsurfaced	Km 130 cinder	1.51	14.8	96
16	100		Bekojo cinder + fines	1.49	11.5	106
17	100		Bekojo cinder (not compacted)	–	–	–
18	100		Mojo cinder (not compacted)	–	–	–
19	100		Mojo cinder + fines	1.45	12.4	101
20	100		Sodere agglomerate	1.91	10.6	97

Table 4. Results of traffic counts

Date of count (Time since construction)	Estimated ADT	Percentage of buses and trucks
6 months	210	63
28 months	156	50
5½ years	173	54
6½ years	230	51

Table 5. Results of axle load surveys

Date of survey (Time since construction)	Mean equivalence factor per vehicle	
	Towards Assela	Towards Nazret
6 months	1.3	2.5
28 months	1.2	2.9
6½ years	2.2	3.6

These are recorded in Table 3. No measurements were made on Section 3 which was of dry-bound macadam construction. Sections 17 and 19 were originally constructed without compaction so the initial density measurements were deferred for three months until compaction by traffic had taken place.

5.5 Maintenance and reconstruction of sections

One year after construction, three of the six gravel sections Nos 17, 18 and 19 were reconstructed. On two other sections, Nos 15, and 16, the upper 50 to 100 mm of the material was loosened with a grader and respread, then watered, shaped and compacted. New material was provided for Sections 17-19 from the same quarries (Bekojo and Mojo) that were used previously. Again, 10 per cent of fines from adjacent to the experimental

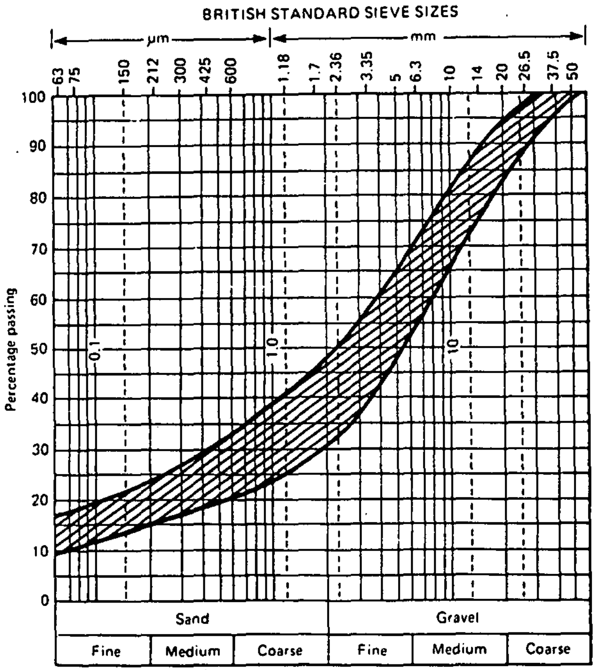


Fig.3 Grading envelope of sub-base material

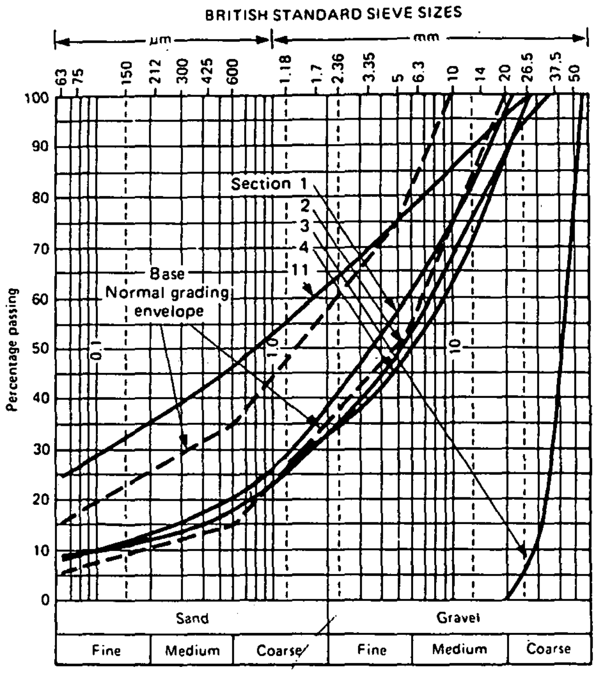
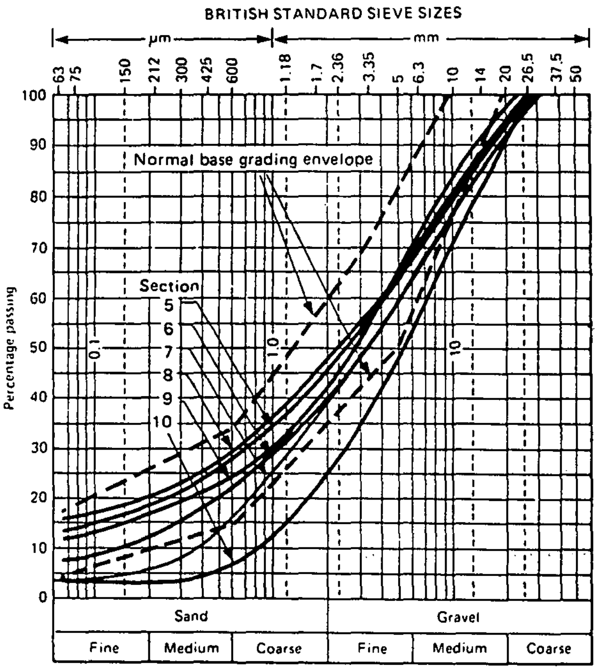


Fig.5 Gradings of other base materials in surface-dressed sections



Fg.4 Gradings of cinder gravels in surface dressed sections

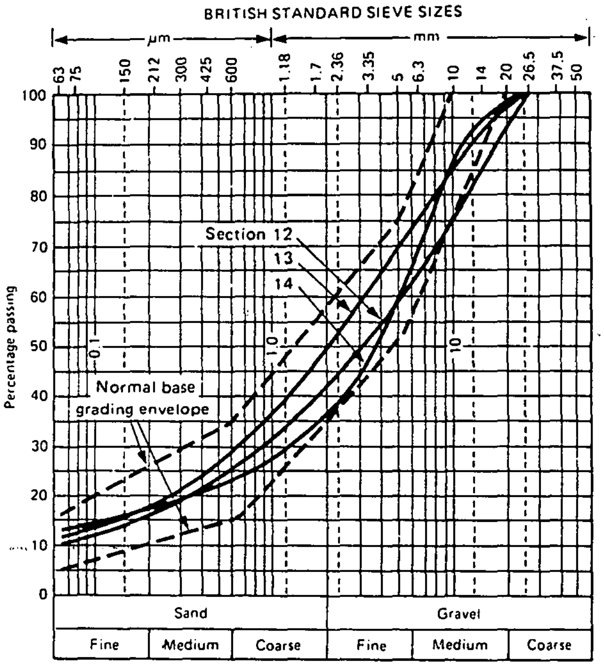


Fig.6 Gradings of bases under road-mix sections

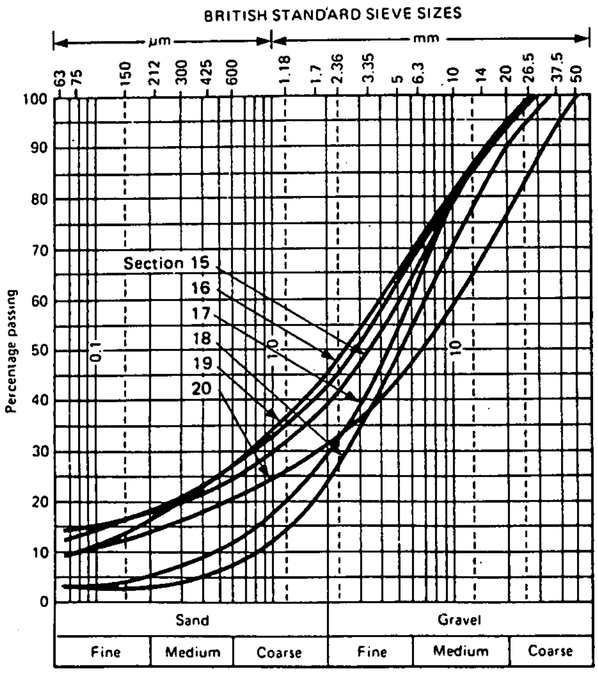


Fig.7 Gradings of gravel test sections

construction area were added to the cinder of Section 19. These materials were watered, graded and rolled, thus providing a comparison with the previous construction where the compaction was carried out by trafficking on Sections 17 and 18. Section 20 was not reconstructed or maintained at this stage. During the first year of trafficking, maintenance grading was carried out twice: once on Sections 15-19 and once on Sections 17-19 only. No maintenance at all was carried out during the second phase of the experiment. After 28 months, monitoring of the gravel sections ceased. At this time, residual gravel was bladed off the sub-base which was then primed and sealed with a single surface dressing.

At the time of the monitoring 28 months after construction, the surfacing of Section 13 was badly cracked and it was necessary to take some action to prevent failure occurring during the next rainy season. As plant was on site to surface dress the gravel sections, the opportunity was taken to apply a single surface dressing to one half section to seal the cracks. This enabled future comparison to be made with the remaining half of the section which had not been resurfaced. Other than this, no pavement maintenance was carried out on any of the surfaced sections during the 7½ years in which they were monitored.

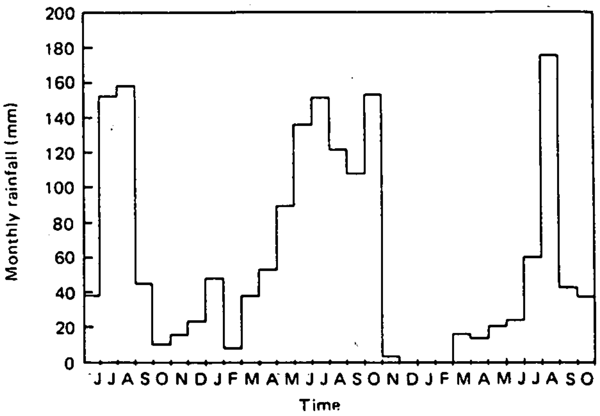


Fig.8 Monthly rainfall during first two years of experiment

6 CLIMATE

Rainfall records were obtained from an automatic rainfall intensity gauge installed in Dhera village approximately 6 km from the experimental site. Back-up readings were obtained from a daily rainfall gauge operated by the Central Arussi Development Unit (CADU) at the same site.

A histogram showing the monthly rainfall pattern during the first two years of the experiment is shown in Fig 8.

It is known from national climatological data (Mesfin 1967) that the mean temperature at the site is approximately 19°C and that this varies little throughout the year. The annual mean rainfall for the area is approximately 700 mm with the wettest months from May to September.

7 TRAFFIC SURVEYS AND AXLE LOADING

During the course of the experiment, four traffic surveys were carried out. These comprised manual classified counts of vehicles for seven consecutive days. On three of these surveys, axle loads were also measured using a portable weighbridge located close to the site of the experiment.

A summary of the surveys is given in Tables 4 and 5.

The reduction in flow between the 6 and 28 month surveys reflects the removal of a large amount of construction traffic from the road following the completion of the Awash-Assela project 11 months after construction of the experimental sections. It is clear from Table 5, that the Nazret traffic lane was the most heavily laden, and the detailed results from the survey show that many trucks travelling in this direction were loaded with agricultural produce, whereas there was a higher percentage of empty vehicles travelling in the other direction.

In the absence of axle load data for the period between 28 months and 6½ years, the mean equivalence factor for intervening years has been taken as the mean of these two values. Thus, values of 1.7 in the Assela direction and 3.3 in the Nazret direction have been assumed. On this basis, the cumulative axle loading in the heavier laden direction was determined to be 440,000 esa over the 7½ year life of the project.

8 MONITORING OF PERFORMANCE

8.1 Density and moisture content

In situ densities of the base were measured using the sand-replacement test with a 4 inch cone. Samples were weighed in the field then returned to the laboratory where moisture contents were determined. On the paved sections, three measurements were taken spread equally along the length of the road and arranged to cover the centre line and two metres on either side. On these sections, it was necessary to remove the surfacing in order to take the measurements. On the gravel sections, five equally spaced measurements were made in a similar way. Here, results were corrected for the irregularity of the surface.

8.2 Particle size distribution

Grading samples were collected adjacent to each density hole. The samples for each section were combined and then analysed by wet sieve analysis. No gradings were carried out on the dry-bound macadam base of Section 3. With this exception, the paved sections were sampled on four occasions: immediately after construction and after 1 year, 2 years and 6 years. The gravel sections (15-20) were sampled at more frequent intervals during their life.

8.3 Subgrade strength

In situ California Bearing Ratios of the subgrade were made in holes dug through the pavement layers using a rig mounted on the back of a truck. One measurement per section was made and those were taken on the centre line and left and right hand lanes varying along the road. Readings were taken one year after construction and again a year and a half later and after 6 years. The moisture content of the subgrade at the points where CBR tests were carried out were also measured.

In addition to the CBR measurements, a Clegg Impact Hammer (Clegg 1976) was used to determine subgrade strength during the second round of measurements. Pavement strength was also assessed using a dynamic cone penetrometer (Kleyn and Savage 1982) during the final round of monitoring.

8.4 Cross-section levelling and rutting

The surface of all the sections were levelled at regular intervals. Readings were taken on a grid of 25 centimetres across the road and 10 metres along the road. Paved sections were marked with paint at 10 metre intervals along the centre line to act as a datum for the measurements and 15 centimetre nails were knocked into the gravel sections at 30 metre intervals along the centre line between which a measuring tape could be stretched. Elevations were tied to bench marks established approximately 30 metres either side of the road at the start and finish of each test section.

Throughout the period of the experiment, elevations obtained were used to plot profiles so that the deformation of the surface could be monitored. Using these profiles, the formation of ruts was measured under a simulated two metre straightedge. During the later stages of the experiment on the paved sections, any rutting in the wheelpaths was measured directly under a 2 m straightedge. The elevations obtained at each survey from the gravel sections were read directly from field sheets into a computer program which determined the loss of surfacing material between each survey.

8.5 Corrugations

The formation of corrugations on the gravel sections was monitored both before and after reconstruction. Initially, corrugations were measured by spanning adjacent crests with a straightedge and measuring the depth of the trough with a steel tape but, after six months, a new method was introduced and elevations of crests and troughs were found using an engineer's level and staff. In both cases, measurements were made on the middle 30 metres of the section, traverses being carried out on the centre line and at one metre on either side.

8.6 Roughness

An assessment was made of the roughness (riding quality) of the gravel sections using a vehicle-mounted bump integrator, measurements were corrected for variation of the vehicle characteristics by checking readings on a two kilometre long calibration strip. However, it was not possible to compare readings with those of a standard instrument, hence the values obtained are not absolute.

Measurements were made during the period following reconstruction to the time the gravel sections were paved.

8.7 Deflection

Deflection measurements were made on the paved sections of the experiment using deflection beams and the standard TRRL method (Smith and Jones 1980). Points where deflections were to be measured were painted on the road and all readings were taken within 150 mm of these. Measurements were made in both wheel tracks and in both traffic lanes at 5 metre intervals along the road in the centre 40 metres of each test section. Temperature calibrations were carried out and readings were corrected as necessary for road temperature.

8.8 Cracking

Measurements of cracking were made by placing a one metre square on the road surface on the centre line and at two metres on either side at a point 27.5 metres from the start of each section. Any cracks were marked with chalk and the area was photographed from vertically above. The resulting photographs were then projected on a screen at full size enabling the total length of cracking to be measured directly.

9 PERFORMANCE OF THE GRAVEL TEST SECTIONS

9.1 Monitoring

The gravel test sections were monitored for a period of 28 months. Total traffic in this period was approximately 140,000 vehicles (70,000 in each lane) which was divided into two phases. The first phase lasted for 11 months following initial construction, after which Sections 15 and 16 were reshaped and recompacted, and Sections 17 and 19 were regravelled and compacted. During the second phase, lasting for 17 months, no maintenance was carried out. Section 20 received no maintenance throughout the whole period of the experiment.

The performance of the gravel test sections was assessed by measurements of deformation, by loss of material and by changes in materials properties. Deformation is indicated principally by the formation of ruts and corrugations, as well as general loss of shape and an increase in roughness.

9.2 Deformation

9.2.1 Rutting

The relationship between rut depth and traffic is shown in Figs 9-13. During the first phase, the range of mean rut depths for the different sections was from 22 mm for Section 20 to 44 mm for Section 19, and this agreed fairly closely with measurements made in the second phase. It is important to note, however, that for all of the sections there was considerable variation in the rut depths of individual wheel-tracks. In general, the greatest rut depths occurred in the Assela-bound lane, which surprisingly was not the lane carrying the heavier traffic, although vehicles were climbing a slight gradient in this direction.
The maximum mean rut depth measured in any wheel-track was 114 mm (Section 19, Assela lane, outside wheel-track) and it is clear that all of the surfacing material was lost from this wheel-track. On site, this was observed by the change of colour of the gravel as the sub-base showed through. Under non-experimental conditions it is clear that maintenance by motor-grader would be required before this condition was reached.

Lack of compaction during construction was apparently not detrimental to the materials' ability to withstand the formation of ruts.

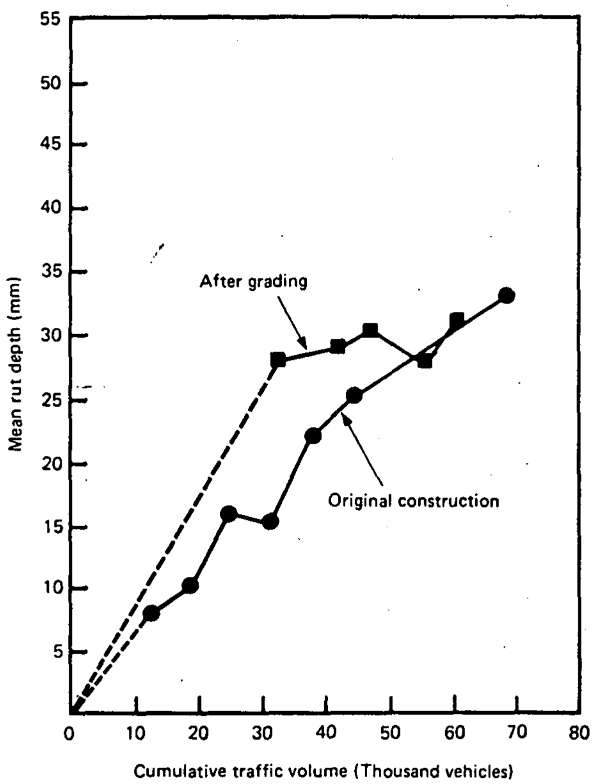


Fig.9 Rutting of Section 15

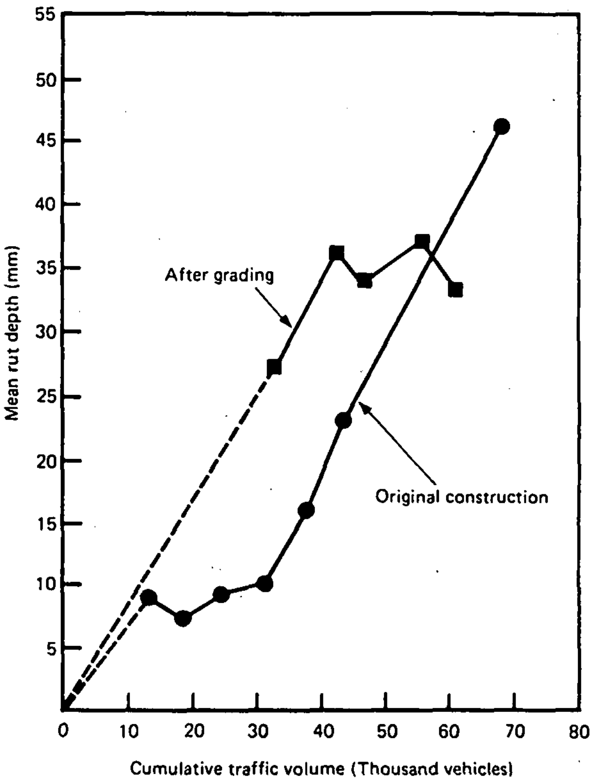


Fig.10 Rutting of Section 16

9.2.2 Corrugations

These formed on all sections except Section 20, although the degree of corrugation varied considerably. In all cases, corrugations were more severe on the Nazret-bound lane. Thus, the road tended to rut in the Assela-bound lane where there was a slight gradient, but where axle loads are small, and the road tended to corrugate in the other lane which has a slight downhill gradient and a higher axle loading. The reasons for this are likely to be that corrugations are dependent on vehicle speed (Heath and Robinson 1980) which would probably be higher on the downhill gradient, and that the abrasive action of vehicles climbing the gradient could wear ruts in the road. Because of the short length of the test sections and the relatively low level of traffic flow, it was not practical to measure vehicle speeds over the experimental sections in order to confirm this. It is thought that axle loading contributes to both rutting and the formation of corrugations and it is difficult to relate the differences in the performance in the two directions to the different

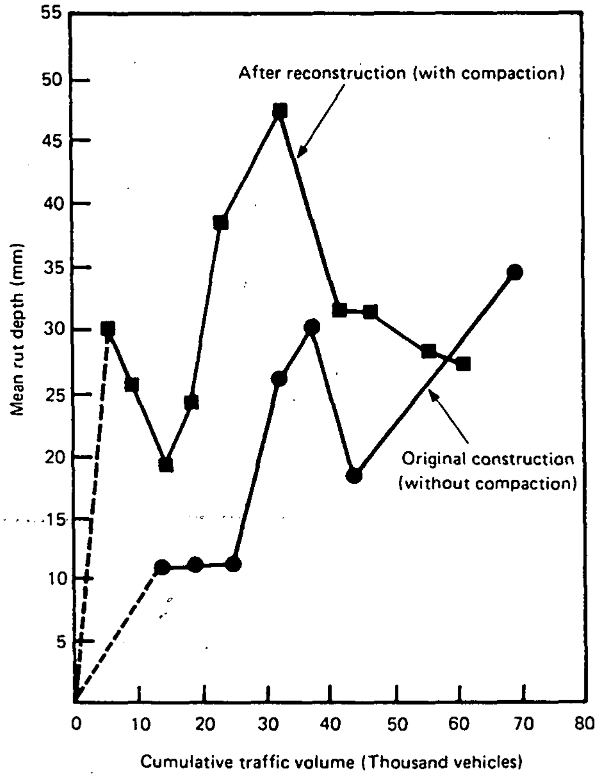


Fig. 11 Rutting of Section 17

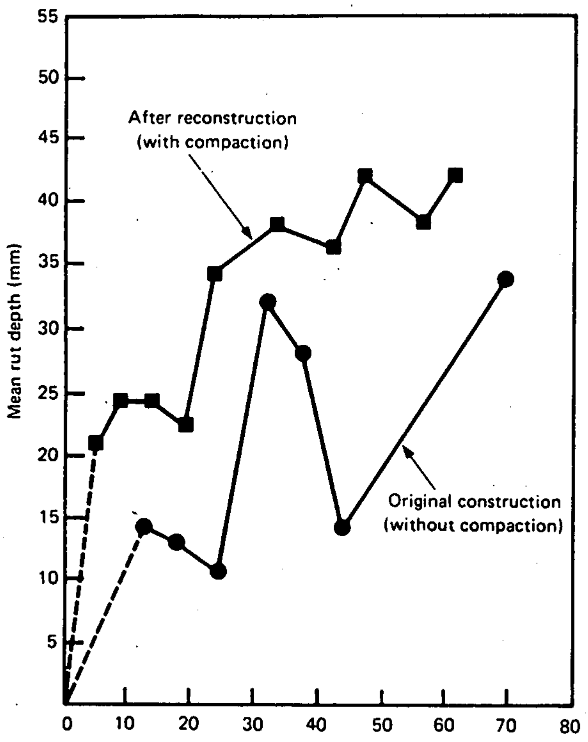


Fig. 12 Rutting of Section 18

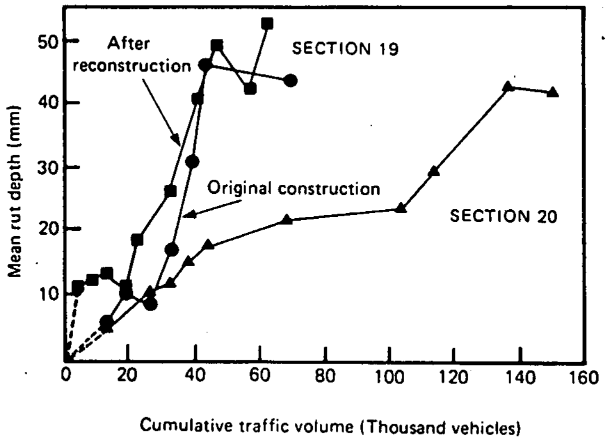


Fig.13 Rutting of Sections 19 and 20

mean equivalence factors.

Sections 17 and 18 both formed severe corrugations within a few weeks of the initial construction. This severity increased through the dry season but then reduced with the onset of the rains. During this period, minor corrugations formed on Sections 15 and 16, but no corrugations formed on Section 20. Although corrugations did form on Section 19, these were much less severe than on Sections 17 and 18. The resistance to corrugation of the various sections was a major factor when designing the reconstruction after 11 months. Sections 15 and 16 which had performed relatively well were then reshaped, watered and rolled without the addition of any new material, whereas Sections 17 and 18 were completely reconstructed using new material from the same quarries as the original, but this time with compaction at optimum moisture content to try and improve the subsequent performance. Section 19 was completely reconstructed to the same specification as the original to provide a control section. Section 20 which had performed very well was left untouched.

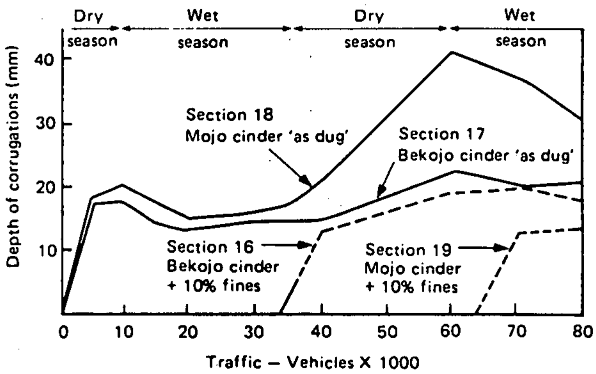


Fig.14 Depth of corrugations of gravel surfaced sections (Nazret Lane)

After reconstruction, Sections 17 and 18 again formed corrugations very quickly, but these never reached the severity which the uncompacted material had achieved. Again there was a tendency for the severity of the corrugations to cycle with the wet and dry seasons and an overall tendency for the severity of the corrugations to reduce with time as the surfacing material was gradually worn away. Sections 16 and 19 did not exhibit corrugations until six months after reshaping and reconstruction respectively and these were of a very minor nature. Section 20 continued not to corrugate and, following reshaping, no corrugations appeared on Section 15 either. The corrugation measurements made during this second phase are shown in Fig 14.

From these results, it is clear that the untreated cinders in Sections 17 and 18 are prone to the formation of corrugations whereas the materials in the other sections are relatively resistant to their formation. The grading envelopes of these two groups of materials at the time of construction are plotted in Fig 15. It is clear

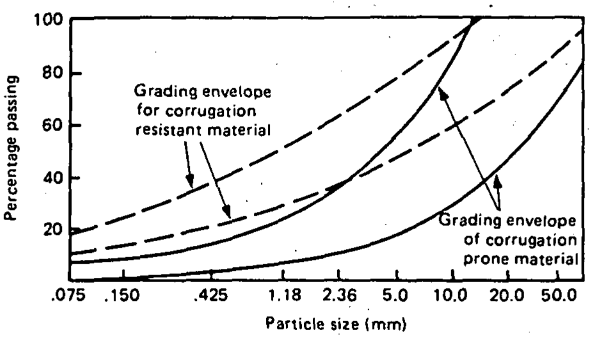


Fig. 15 **Grading envelopes of corrugation resistant and prone materials**

from this that the corrugation resistance of these cinder gravels is related to their fines content. Materials with a fines content of 10 per cent or less are prone to corrugations, and corrugation resistant materials have a fines content of more than 10 per cent. The volcanic ash soil added to the cinder had a plasticity index between 8 and 15 and the cohesion this provided contributed to the improved performance as a gravel wearing course. It is interesting to note that the grading envelope for corrugation resistant material was wider than that normally recommended (TRRL ORN 2) but the plasticity index was similar to the preferred limits.

9.2.3 Roughness

The progression of roughness is shown in Figs 16-18 and it can be seen that these readings correlate well with the corrugation observations. The rapid formation of corrugations in the Nazret-bound lane of Sections 17 and 18 after the reconstruction can be clearly seen. Sections 18 and 19 start off relatively smooth, and Section 16 exhibits characteristics midway between the two. As observed on site, the general level of roughness on Section 20 was considerably higher than the other sections. This is because of the amount of oversize material in this gravel. The riding quality of all sections deteriorated with time, but again the seasonal trend is observed with conditions improving during the wet seasons. The reason that all of the readings fell at the final measurement is not understood, but may be connected with a calibration problem experienced at that time.

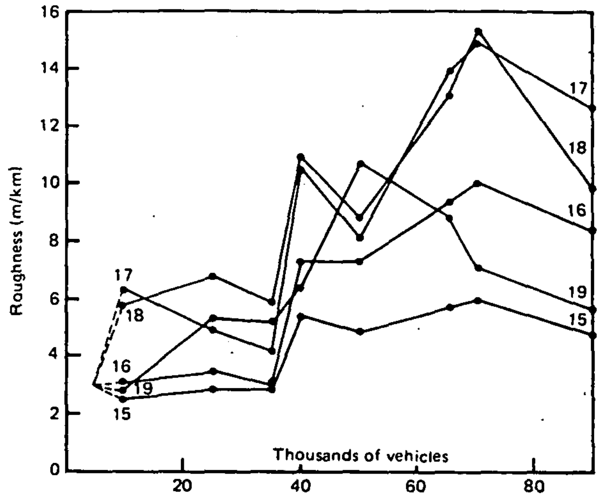


Fig.16 **Roughness progression for Assela-Bound Lane**

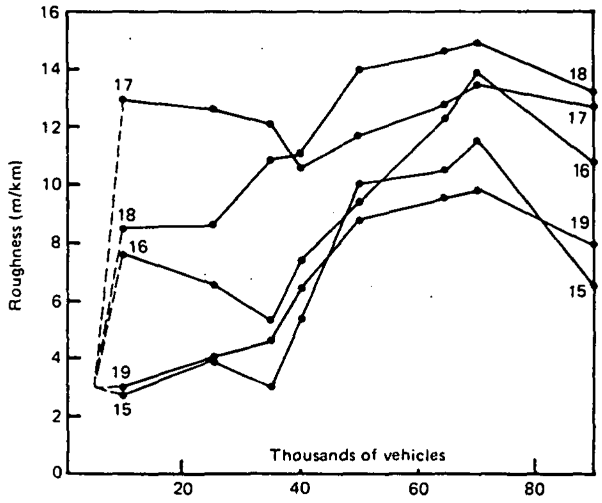


Fig.17 **Roughness progression for Nazret-Bound Lane**

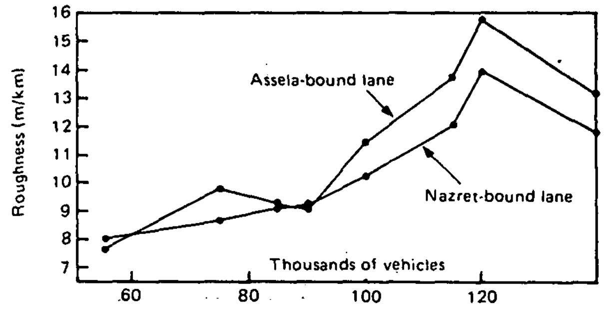


Fig.18 **Roughness progression for section 20**

Roughness values in excess of about 10 m/km would normally be considered to be uncomfortable from a ride point of view and, in the Nazret direction, all sections exceeded or came close to this value eventually.

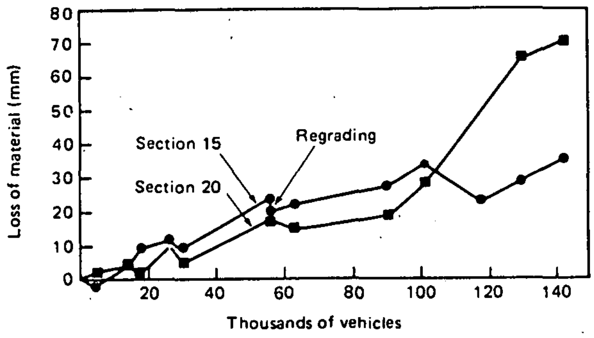


Fig.19 Gravel loss on sections 15 and 20

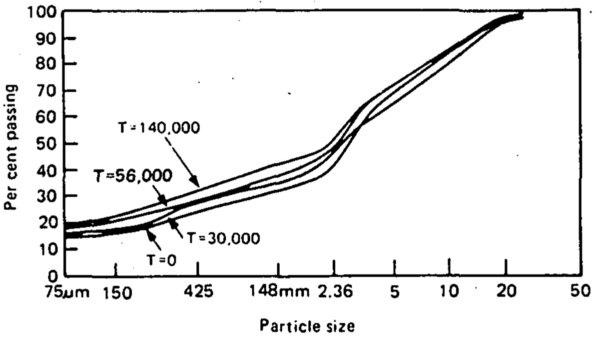


Fig.22 Change in grading of section 15

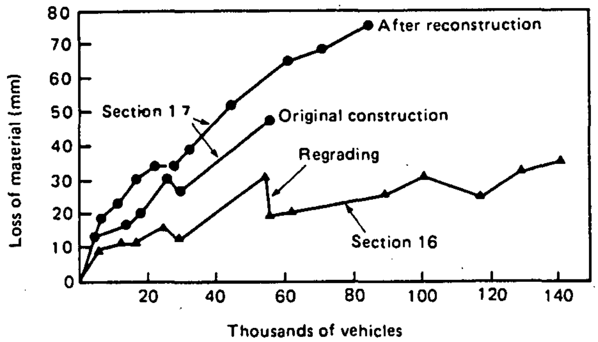


Fig. 20 Gravel loss on sections 16 and 17

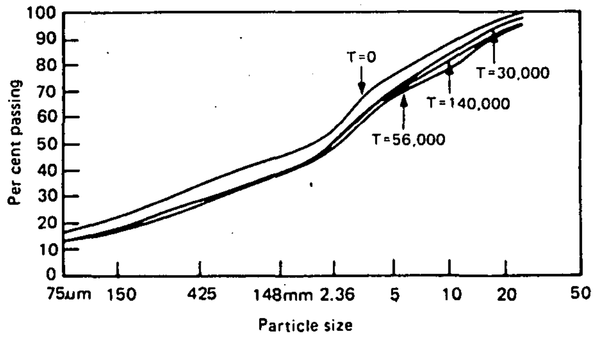


Fig.23 Change in grading of section 16

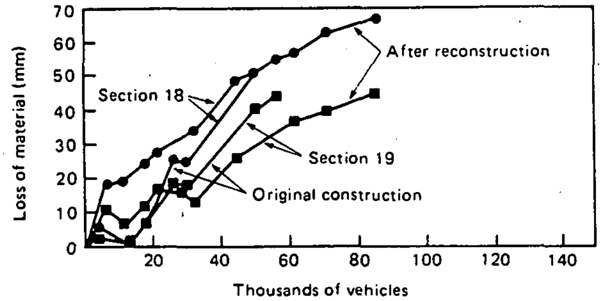


Fig. 21 Gravel loss on sections 18 and 19

9.3 Loss of material

The relation between gravel and total traffic volume is shown in Figs 19-21 for the six sections. These show that the three sections that were completely reconstructed all exhibited similar rates of loss before and after reconstruction, with the loss by Sections 17 and 18 being slightly higher than for Section 19. The effect of maintenance grading on Sections 15 and 16 is clearly shown, the reduction in loss being due to surfacing material being brought back from the shoulders on to the carriageway.

For the first 90,000 vehicle passes it was possible to compare the six sections directly. Up to this point Section 20 performed best having lost only 20 mm of material. This performance was closely followed by that of Sections 15 and 16 which each lost about 25 mm of material. The other three sections each lost approximately twice as much material, Sections 17 and 18 losing around 70 mm and Section 19 losing around 45 mm.

The performance of Sections 15 and 16 remained good up to 140,000 vehicle passes by which time these sections had still only lost 35 mm of gravel. However, the performance of Section 20 rapidly deteriorated after 90,000 vehicles such that, by the time 140,000 vehicles had passed, the loss of material had more than trebled to 70 mm. However, measurements of rutting indicated that the verge-side rut in the Assela-bound lane had moved outside the normal width of the carriageway and it was most probable that this 'lost' material was actually deposited on the shoulder just beyond the extreme point of the gravel loss measurements. Nevertheless, it would be normally extraordinary for a gravel road to carry this volume of traffic without even being graded.

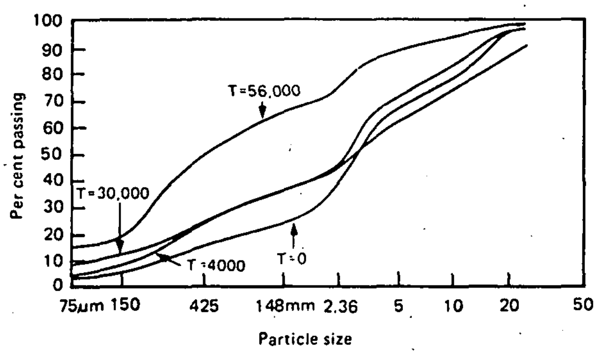


Fig.24 Change in grading of section 17 (Original construction)

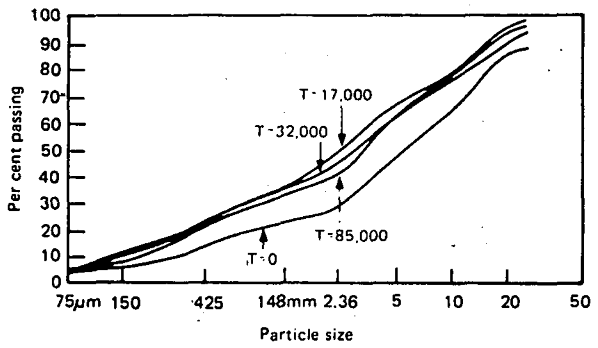


Fig.25 Change in grading of section 17 after reconstruction

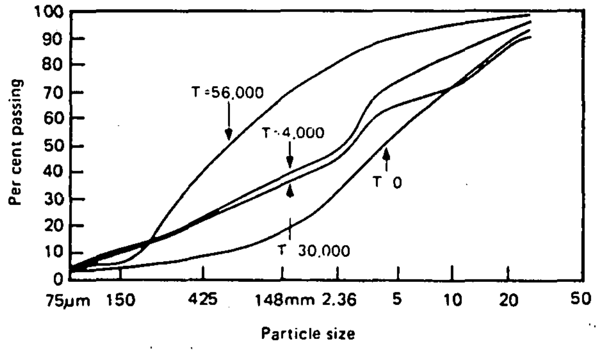


Fig.26 Change in grading of section 18 (Original constructior

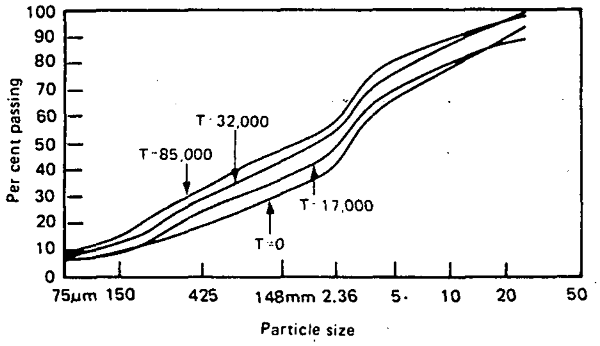


Fig.27 Change in grading of section 18 after reconstruction

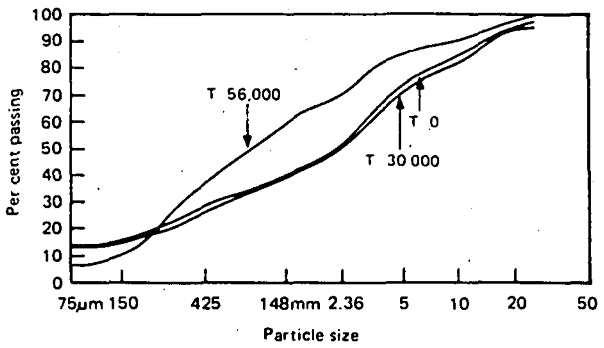


Fig.28 Change in grading of section 19 (Original construction

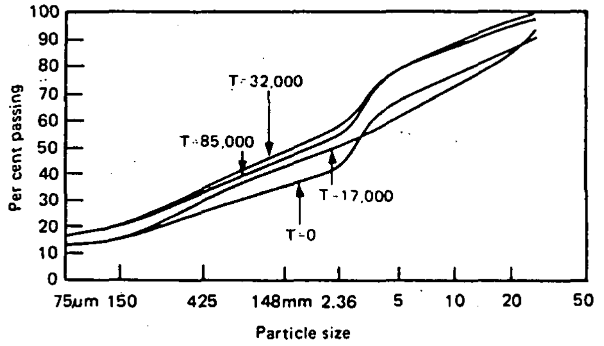


Fig.29 Change in grading of section 19 after reconstruction

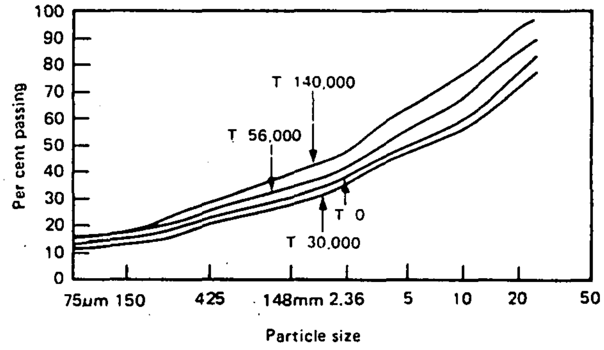


Fig.30 Change in grading of section 20

9.4 Materials properties

Particle-size distributions and compacted dry densities of the gravel sections were measured at intervals during the course of the experiment and are shown in Figs 22-30 and Table 6. For the untreated cinders in Sections 17 and 18, considerable changes of particle-size distribution were caused by trafficking during the first phase when the two sections were laid without compaction. When the two sections were reconstructed with compaction, the changes in

Table 6. Density and moisture content of unpaved sections.

	Density (Mg/m^3) at months after construction						
Section	0	3	7	13	16	20	30
15	1.51	-	1.46**	1.46	-	-	1.54
16	1.49	-	1.54**	1.47	-	-	1.47
17	-	1.48**	1.56**	1.28*	1.45	1.55	-
18	-	1.46**	1.52**	1.24*	1.40	1.52	-
19	1.45	-	1.43**	1.39**	1.48	1.49	1.54
20	1.91	-	1.83**	1.89	-	-	1.94

Subgrade at construction = 1.49
Sub-base at construction = 1.49

	Moisture content, per cent, at months after construction						
	0	3	7	13	16	20	30
15	15	-	9	12	-	-	11
16	12	-	2	6	-	-	3
17	-	9	1	13*	10	7	2
18	-	12	2	16*	11	4	2
19	12	-	4	16*	8	6	5
20	11	-	4	12	-	-	4

Subgrade at construction = 15
Sub-base at construction = 14

*After reconstruction
**Measurements made on rough surface with no correction made for the irregularity

particle-size distribution were less, but trafficking did lead to similar marked increases in density as in the first phase. The properties of the materials in the other sections, which had higher 'fines' contents, changed little during the experiment, although Section 19, the Mojo cinder with fines added, did change more than the others.

9.5 Conclusions from gravel surfaced sections

The results of the gravel surfaced experimental sections showed that:-

(i) After 140,000 vehicle passes and no maintenance, the Sodere agglomerate in Section 20 had performed well. However, during this time, as trafficking increased, the level of surface roughness and gravel loss became higher because abrasion of the fine soil matrix led to large oversize material (> 37 mm) being exposed and eventually lost from the surface. Screening or crushing the material to remove or break down the large-sized stone would be expected to improve the quality.

(ii) In the main part of the experiment to examine cinder gravels it was clear that by comparing the modified cinders in Sections 16 and 19 with the untreated cinders in Sections 17 and 18 much better performance was obtained by mechanical stabilisation. This was reflected by lower rates of gravel loss and roughness and by the development of corrugations. It was notable that the naturally occurring cinder gravel which contained sufficient fines and was used in Section 15 performed as well as the mechanically stabilised materials. However, such well-graded cinder gravels are rare and it was regarded as an atypical material.

(iii) There appeared to be little difference in the performance of the untreated cinder gravels (Sections 17 and 18) when comparisons were made between the materials laid with and without compaction. This was because traffic induced compaction accompanied by the breakdown of weak cinder particles increased the density to the same level in both cases. The fines produced in this way, however, were non-plastic and

Table 7. Deterioration of premix surfaced sections.

Section No.	Measurement	Years after construction							
		0.5	1.5	2.5	3.5	4.5	5.5	6.5	7.5
12	Cracking (m/m²)	0	0	4.8	9.1	-	10.4	11.0	-
	Rutting (mm)	5	3	2	4	3	-	4	-
	Deflection (mm)	0.91	0.97	0.94	1.12	1.16	1.45	1.78	1.71
13	Cracking	0	0	1.3	9.6 (1.0)	-	8.3 (1.4)	12.7 (3.6)	
	Rutting	3	3	4	5	5	-	5	
	Deflection	1.06	1.23	1.38	1.47 (1.36)	1.37 (1.18)	1.45 (1.29)	1.64 (1.33)	1.49 (1.46)
14	Cracking	0	0	0	0.7	-	4.6	15.9	-
	Rutting	1	1	2	2	2	-	4	-
	Deflection	0.79	-	0.78	1.03	1.02	1.18	1.23	1.30
Axle loading ESA x 1000 (Nazret direction)		30	80	117	183	239	296	360	440

Note: Values in brackets are measurements made on the half of Section 13 that was sealed with surface dressing

did not bind the material as well as the plastic volcanic ash soil. It is recommended therefore that in order to obtain good performance from cinder gravels as gravel wearing courses they should be stabilised with volcanic ash.

(iv) In the mechanically stabilised sections of the experiment, approximately 10 per cent of the ash soil was dumped and spread by a front-end loader on top of the previously spread layer of cinder gravel. Mixing was carried out by motor grader and it is believed that this method of processing could readily be used in normal construction work.

10 PERFORMANCE OF THE PAVED TEST SECTIONS

10.1 Monitoring

The paved sections were monitored for a period of 7½ years. During this time, 440,000 cumulative equivalent standard axles were carried in the heavier trafficked lane towards Nazret. The rate of deterioration was studied without maintenance being carried out, except for Section 13 where, after 28 months, the first half of the section was treated with a single surface dressing to seal cracks. This enabled a determination of the effectiveness of the seal as cracks continued to develop on the other part of the section.

The performance of the sections was assessed by surface measurements of rutting, cracking and deflection at approximately yearly intervals. In addition, particle size distribution, density and moisture content of the base materials and strength, in terms of CBR, and moisture content of the subgrade were measured three times during the experiment. The main difference between those sections that were treated with a double surface dressing and those treated with a premix surfacing are discussed below.

10.2 Rutting and cracking

For the surface dressed sections (Nos 1-11), there was no increase in rutting during the monitoring period and the maximum value recorded was 4 mm. For the premix sections (Nos 12-14), the rut depths were similar, but there was some indication that slight increases were occurring as trafficking increased (see Table 7). Of section Nos 1-11, the only cracking that occurred was on Section 11 where a longitudinal crack developed in the sixth year close to the edge of the pavement in part of the Nazret lane. As there were no other signs of failure, it was believed that this was caused by shrinkage of the plastic volcanic tuff in the road base. Cracking began to occur in Section 12 of the premix sections in the third year and this quickly increased to a level regarded as unacceptable (TRRL Overseas Unit 1981). Section 13 also started to crack in the third year and the level of this had increased so much by the following year that the first half of the section was treated with a single coat of surface dressing. The effect of the surface dressing initially sealed the cracks but they were becoming extensive again three years later. The onset of cracking was much later in Section 16 but, by the sixth year, these had also reached an unacceptable level. Examination of the premix surfacing in cracked areas showed clearly that the cracking had started from the top of the layer and had progressed downwards.

10.3 Deflection

The transient deflections of the paved sections are shown in Figs 31 and 32 and those for the premix sections are also shown in Table 7. For the surfaced dressed sections, deflection measurements, which are recorded as the means of all values within a section at each survey, either reduced or stayed constant with increasing traffic during the first 6½ years of the experiment. A general increase occurred in the last series of measurements but, apart from Section 11, no other signs of surface deterioration were evident. The behaviour of the premix sections was quite different, with Section 13 showing increases from early in the life of the experiment, and Sections 12 and 14 showing progressive increases from the third year onwards. The increases in deflection correlated well with increases in cracking. When surface dressing treatment was applied to part of Section 13 to seal cracks, this coincided with a reduction in the pavement deflection for two years. This was followed by further increases as cracks began to appear again.

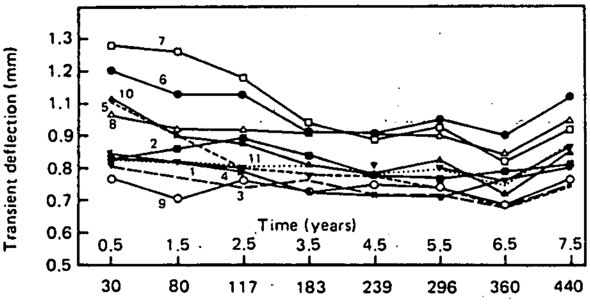


Fig.31 Transient deflections of surfaced dressed sections (Nos. 1–11)

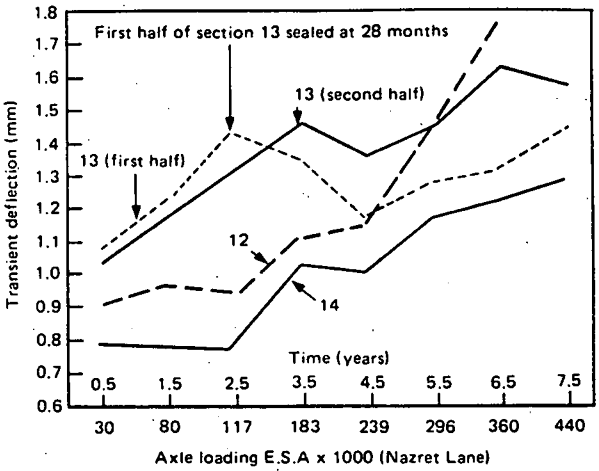


Fig.32 Transient deflections of premix surfaced sections (Nos. 12-14)

10.4 Materials properties

The measurements of materials properties in the base, sub-base and subgrade made at intervals during the experiment showed that the only marked changes that occurred were in the moisture contents and strengths of the pavement layers in the premix sections. However, it was not until the sixth year that increases in moisture content and reductions in strength were detected which was four years after cracks first appeared in the surfacing. This supports the observation that cracks in the surfacing started from the top of the layer and took some years before they penetrated the whole layer.

In situ CBR values of the different base materials, interpreted from the use of the Clegg Impact Hammer, ranged from 68 per cent to 140 per cent, with the highest values for the control section of the crushed stone base (Section 2) and the Sodere agglomerate (Section 4). Subgrade strengths for the volcanic ash soil varied from 13 per cent to 38 per cent at the time

Table 8. In situ densities and moisture contents of the base of paved sections

Section No.	At construction		After 1 year		After 2½ years	
	Density Mg/m³	Moisture content %	Density Mg/m³	Moisture content %	Density Mg/m³	Moisture content %
1	1.53	14	1.43	13	1.47	11
2	1.99	7	1.91	4	2.05	4
3	–	–	–	–	–	–
4	1.90	9	1.71	6	1.89	4
5	1.49	9	1.32	7	1.48	5
6	1.43	10	1.35	6	1.50	5
7	1.41	10	1.46	2	1.46	1
8	1.59	15	1.45	6	1.47	5
9	1.50	16	1.60	8	1.57	7
10	1.34	15	1.34	4	–	2
11	1.39	21	1.34	11	1.55	11
12	1.45	12	1.58	5	1.38	6
13	1.45	14	1.36	9	1.36	9
14	1.48	16	1.47	14	1.44	11

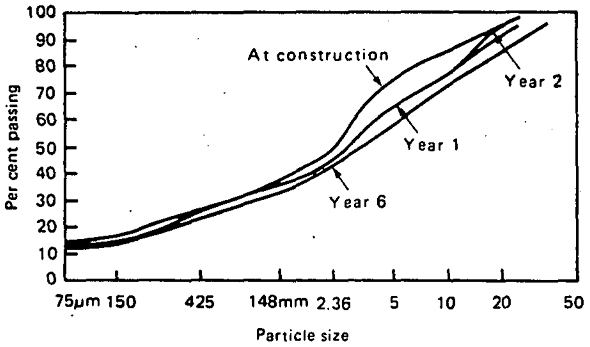


Fig.33 Change in grading of section 6 (Bekojo cinder + fines)

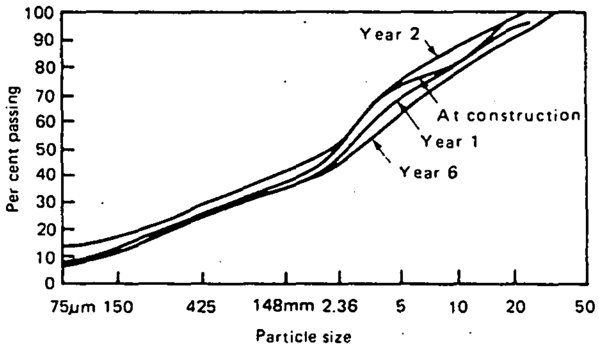


Fig.34 Change in grading of section 9 (Crushed Mojo cinder + fines)

of construction. By the sixth year, the strengths of Sections 12 and 13 were 11 per cent. No appreciable changes occurred in the particle size distributions or the densities of the base materials. As examples, the particle size distributions of Sections 6 and 9 measured at four different times are shown in Figs 33 and 34 and densities and moisture contents are shown in Table 8.

10.5 Conclusions from the paved test sections

(i) The satisfactory performance of the surface dressed sections showed that cinder gravels, whether untreated or mechanically stabilised with volcanic ash soil, were suitable for use in road bases up to a level of 440,000 esa. The use of quarry fines (less than 5 mm maximum size) vibrated into the surface of the compacted cinder gravel also provided a satisfactory base material.

(ii) The mechanically stabilised cinder gravels and the km 130 material which was naturally well graded were easier to compact than untreated cinders.

(iii) The dry bound macadam, the Sodere agglomerate and the volcanic tuff also performed satisfactorily during the experiment although the volcanic tuff did show some cracking.

(iv) The roadmix surfacing was unsatisfactory. It is believed that the material was too porous and that either changes in the properties of the bituminous binder, insufficient binder or inadequate mixing was the main reason that caused the surface to crack. It is possible that the application of a surface dressing to the roadmix immediately after construction could have prevented the cracking. However, with the considerably better performance of the surface dressed Sections 1-11, the use of roadmixed asphalt is clearly uneconomic with or without a further seal.

11 RECOMMENDATIONS

(i) Cinder gravels typical of those used in the full-scale road trials and with particle size distributions similar to those shown in Fig 1 can be used for road bases in lightly trafficked surface dressed roads. Such gravels have now been tested for carrying a traffic loading of 440,000 esa, which is close to the design for 500,000 esa included in Road Note 31.

(ii) Other materials used in the road base of the surface dressed sections performed satisfactorily. These included dry bound macadam, which is a construction process without the use of water, Sodere agglomerate and a volcanic tuff.

(iii) For gravel surfaced roads, mechanically stabilised cinder gravels perform much better than as-dug cinders which lack plastic binder even after the breakdown of cinder particles by trafficking. Recommended gradings for materials which are more resistant to corrugations are shown in Fig 15.

(iv) Mechanically stabilised cinder gravels using 10 per cent of locally available volcanic ash soil are easier to compact and process during construction than as-dug cinders. For as-dug cinders, it is necessary to start compaction with a pneumatic tyred roller before using either a smooth wheeled or vibrating roller. There was little breakdown of cinder particles during compaction with the type of plant used.

(v) Roadmix is not a satisfactory method of providing a bituminous surface for cinder gravels.

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