TRANSPORT RESEARCH LABORATORY

Enabling the use of secondary aggregates and binders in pavement foundations

Prepared for British Cement Association, British Steel, Buxton Lime Industries, Bardon Aggregates/ECC International, National Power, PowerGen, Tarmac HBM (UK) and the Department of the Environment, Transport and the Regions, Science and Technology Policy Division

V M Atkinson, B C Chaddock and A R Dawson

TRL REPORT 408

First Published 1999
ISSN 0968-4107

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CONTENTS

Executive Summary

This report describes the secondary materials used during the research, the laboratory testing of mixtures including secondary aggregates and binders and a full-scale trial of six sub-bases constructed using these mixtures, carried out in TRL's Pavement Test Facility (PTF). The secondary materials investigated were china clay sand, blastfurnace slag, steel slag, pulverised fuel ash, gypsum and cement kiln dust.

Mixtures of the secondary materials, sometimes combined with conventional aggregates and binders, were selected for laboratory testing. The selection was based on known successful performance, suggestions from sponsors and best estimates of novel, successful mixtures. Thirteen mixtures were selected which divided into three families, according to the major component of each mixture. These families were:

i Fine aggregate - Pulverised fuel ash or Gypsum.

ii Medium aggregate - China clay sand.

iii Coarse aggregate - Crushed granite Type 1 or Air-cooled blastfurnace slag.

Each mixture was tested in the laboratory for compactability, compressive strength, indirect tensile strength and stiffness and durability. Based on the results of these tests, six mixtures were selected for further testing and for use as sub-base in full scale trials:

Mixture 3: Pulverised fuel ash, gypsum and quicklime.

Mixture 4: Pulverised fuel ash, cement kiln dust and granulated blastfurnace slag.

Mixture 7: China clay sand, cement kiln dust and ordinary Portland cement.

Mixture 8: China clay sand, pulverised fuel ash, quicklime and sodium carbonate.

Mixture 11: Crushed granite, pulverised fuel ash, quicklime and gypsum.

Mixture 13: Air-cooled blastfurnace slag, granulated blastfurnace slag and basic oxygen steel (BOS) slag.

Further laboratory testing examined the effect of small changes in moisture content and constituent proportions on the structural properties of each mixture. The performance of most mixtures was improved by mixing them slightly wet of optimum moisture content, allowing more water for hydration of the binder. A low curing temperature reduced the strength and stiffness of some mixtures significantly. Depending on the material, it could be laid from 3 to over 24 hours after mixing, with little loss of strength and stiffness.

The variant of each mixture used in the full scale trials was chosen to ensure a range of strength and stiffness within each family. The two materials with pulverised fuel ash as the main component were produced in a continuous flow, forced action mixer. The other materials were mixed in a batch paddle mixer except for the all slag mixture, which was mixed in an asphalt plant.

The sub-base materials were each laid in a wedge with the thickness varying along the direction of trafficking. One material from each family and a control of unbound, Type 1 granular material, were laid to thicknesses thought appropriate for trafficking immediately after laying. The remaining materials and a CBM1 control were laid to a lesser thickness appropriate to trafficking commencing seven days after laying. All sub-bases were laid on a uniform clay subgrade, of approximate CBR 3 per cent. The performance of the resulting foundations, under wheel loads representative of construction traffic, was assessed by measuring deformation in the wheelpath, ruts and the foundation surface modulus obtained from Falling Weight Deflectometer (FWD) testing.

One of the foundations designed to be trafficked immediately after laying hydraulically bound sub-base material was successfully trafficked twelve hours after construction. Difficulties in early trafficking of the others may have been due to low compacted density (mixture 3) and high moisture content (mixture 13).

In general the surface modulus of the foundations increased with increase in the thickness of sub-base. Where sub-base material remained undamaged by trafficking, the surface modulus of the foundations also increased over the period of trafficking.

All foundations incorporating secondary aggregates and binders performed better than that with unbound, Type 1 sub-base, some performing better than the CBM1 control.

This trial has shown that it is possible to mix, lay and traffic satisfactorily as sub-base, selected materials comprising mixtures of secondary aggregates and binders. It has also provided guidance on the required thickness of these sub-bases for satisfactory performance on a low strength subgrade.

Based on the laboratory and full scale trials, a procedure for the design of sub-bases comprising secondary materials has been produced. Use of these materials results in environmental benefits, and the stronger foundations produced can lead to thinner upper layers of the pavement and/or to pavements with a longer life.

1 Introduction

Enabling the use of secondary aggregates and binders in pavement foundations has many potential benefits and the determination of material performance data and a generic materials specification will provide the framework for successful use of these materials in road construction. The use of secondary materials has the environmental benefit of decreasing the requirement for extraction of primary aggregates and the disposal of secondary materials to landfill, thus assisting in meeting government recycling targets. The use of hydraulically bound mixtures of secondary materials could also produce stiffer and stronger foundations than those produced with conventional unbound, granular Type 1 material used as sub-base, which may result in the possibility of thinning the upper pavement layers and/or increasing the life of the pavements. The purpose of this research was therefore to investigate the use of secondary aggregates and binders in pavement foundations and to examine the performance of these foundations under construction traffic.

The laboratory and full-scale testing was carried out by the research partners, the Transport Research Laboratory and the Department of Civil Engineering of the University of Nottingham, as part of the LINK Transport Infrastructure and Operations Programme. The research was supported by the British Cement Association, British Steel, Buxton Lime Industries, Bardon Aggregates/ECC International, National Power, PowerGen, Tarmac HBM (UK) and the Department of the Environment, Transport and the Regions.

The principal objectives of the research were to:

i establish appropriate laboratory tests to identify candidate materials and mixtures for use in the pavement foundation;

ii test some of the mixtures in full-scale trials, to evaluate their structural equivalence to conventional materials;

iii develop a generic specification to enable secondary aggregates and binders to be used in the pavement foundation.

The materials examined were those which the project sponsors considered to be most appropriate. This report gives a brief description of each of these materials and presents the results of initial laboratory tests, carried out on thirteen mixtures of the secondary materials, sometimes combined with conventional aggregates and binders. Six mixtures were selected for further testing and the report presents the results of tests on these mixtures, to ascertain the effect of small changes in moisture content and constituent proportions on the structural properties, together with the effect of curing temperature and the workability time.

The report then describes full scale trials carried out in TRL's Pavement Test Facility (PTF) using the six selected mixtures as sub-base, together with control sub-bases of unbound granular, Type 1 material and cement bound material, CBM1. The sub-base materials were laid in a wedge on pre-shaped subgrade, the thickness of each sub-base varying along the direction of trafficking. The performance of the resulting foundations, under wheel loads representative of construction traffic, was assessed by measuring deformation in the wheelpath, ruts and the surface modulus of the foundation obtained from Falling Weight Deflectometer (FWD) testing. The trials showed that good performance can be achieved with sub-bases constructed using secondary aggregates and binders.

Based on the laboratory and full-scale trials, a procedure for the design of sub-bases comprising secondary materials has been produced. The benefits of constructing foundations incorporating secondary materials are discussed.

2 Secondary materials tested

There are many resources, resulting from industrial processes, which can be used to replace primary materials. Those secondary materials tested in this research project, for use in pavement foundations, comprised both aggregates and binders. The materials were:

- China clay sand.
- Air-cooled blastfurnace slag.
- Air-cooled basic oxygen steel (BOS) slag.
- Granulated blastfurnace slag.
- Ground granulated blastfurnace slag.
- Pulverised fuel ash.
- Gypsum.
- Cement kiln dust.

A brief description of the materials follows; further details are given by Nunes (1997).

2.1 China clay sand

China clay sand is the main waste generated when kaolin, or china clay, is extracted from rock by application of high pressure jets of water. Extraction of 1 tonne of china clay generates 9 tonnes of waste, including 3.7 tonnes of coarse sand (OECD, 1977). An estimated 1.5 million tonnes of china clay sand is used in the construction industry each year, in block making, structural concrete and building mortar (Whitbread, Marsay and Tunnel, 1991). This is a small proportion of the available china clay sand, because of the difficulties of transportation from Cornwall and Devon, where the china clay pits are located. China clay sand includes a small percentage of mica.

2.2 Blastfurnace slag

Blastfurnace slag is composed primarily of oxides of calcium and magnesium, and is a co-product of the iron manufacturing industry. The blastfurnace is continuously fed with iron ore, coke and fluxing stone. The blastfurnace slag is formed as the iron oxide charge is chemically reduced and melted and the non-ferrous components combine. In most current blastfurnace practices, the slag is separated from the iron during casting by use of a system of dams and weirs.

To form air-cooled slag the liquid slag is poured into a pit and left to cool slowly in the open air. Water is sprayed onto the solidified but still hot slag, to develop cracks and accelerate the cooling. When crushed and screened according to normal quarry procedures the resulting material is very similar to natural rock in texture and gradation.

For the production of granulated slag the liquid slag is poured into a high pressure water jet which fragments and quenches the molten slag. The high speed of cooling causes the slag particles to solidify as glassy granules of diameter between 0 and 3mm. Granulated blastfurnace slag has a chemical composition similar to cement and possesses hydraulic properties when combined with a basic activator such as lime (SETRA-LCPC, 1979).

Approximately 5 million tonnes of blastfurnace slag currently exists in stockpiles. All the currently produced blastfurnace slag is used in construction, mainly as an aggregate for road construction, as a lightweight aggregate in general building, and in blended cements. The material is produced by British Steel integrated steelworks in Teesside, Scunthorpe, Llanwern and Port Talbot.

2.3 Steel slags

BOS slag is the co-product resulting from the manufacture of steel from pig iron and scrap using oxygen. The BOS slag is predominantly composed of lime, silica and iron oxide. After extraction of free metal the steel slag is screened and stockpiled for weathering to provide a non-expansive aggregate. Currently approximately one million tonnes of steel slag is held in stockpiles undergoing weathering. The annual tonnage arising in the UK is two million tonnes. BOS slag is produced in the four British Steel integrated steelworks mentioned in paragraph 2.2. In Sheerness, South Wales and the Sheffield/Rotherham area where electric furnaces are in operation, Electric Arc Furnace (EAF) slag is produced.

2.4 Pulverised fuel ash

Pulverised fuel ash (pfa) is a by-product of coal burning power stations. There are some 200 million tonnes of ash stockpiled (Whitbread, Marsay and Tunnel, 1991). Ashes are most readily available in South Yorkshire, Nottinghamshire and Derbyshire. Pfa is used in many construction applications such as manufacturing lightweight blocks, as partial replacement of cement and as bulk fill material. The main components of pfa are silicon dioxide, aluminium oxide and iron oxide. Pfa is a pozzolan and becomes bound in the presence of water and calcium hydroxide.

2.5 Gypsum

Flue gas desulphurisation gypsum is generated in power stations burning lignite or sulphur contaminated coals. Gas scrubber systems used to remove the sulphur from the flue gases generate the by-product, gypsum. One of its main uses is for the production of building plaster. It has been used in sub-base and roadbase construction, stabilised with cement and/or pfa.

2.6 Cement kiln dust

During the production of Portland cement the gases generated carry very fine and light particles from the kilns. This material is extracted for environmental reasons resulting in cement kiln dust. Some cement kiln dust is re-used in the production of cement but the material having a high alkali content, not suitable for re-use, is available as secondary binder. This material has also been used in embankments and as filler in bituminous materials. Weathered cement kiln dust will have reduced alkali and sulphate contents when compared with recently produced kiln dust but, due to the hydration of the free lime and cement, its cementitious properties will also be much diminished.

3 Laboratory testing: Phase 1

The potential number of mixtures of secondary aggregates and binders is large, with natural and secondary aggregates being mixed with conventional or secondary binders and activators, in varying combinations and proportions. The selection of mixtures for laboratory testing was based on known successful performance, in the UK and in other European Countries, suggestions from sponsors and best estimates of other, novel, successful mixtures. Thirteen mixtures were selected to represent the large number of possible mixtures. They divided into three families, according to the major component of each mixture. These were:

i Fine aggregate - Pulverised fuel ash or Gypsum.

ii Medium aggregate - China clay sand.

iii Coarse aggregate - Type 1 granular material or Air-cooled blastfurnace slag.

The mixture proportions were chosen on the basis of achieving both short and long term structural capacity, while taking into account practical proportions of constituents for successful mixing. The mixtures are summarised in Table 1, which includes the results of the compactibility testing.

The initial laboratory testing regime was determined to characterise the mixtures in terms of strength, stiffness and fatigue behaviour. The durability of the mixtures was also investigated. Testing was carried out on specimens aged 7, 28, 90 and 360 days, though not all tests were carried out at all ages. The results reported are generally an average of test results on three specimens, for each test and at each time of testing.

3.1 Compactibility tests

To determine the optimum moisture content and dry density, compactibility tests were performed. Tests were carried out in accordance with BS 5835 (1980) and BS 1924 (1990), except that the number of portions taken for each level of initial moisture content was generally reduced to one. Also, the amount of material compacted was chosen to give similar sized compacted specimens for all mixtures, rather than taking a standard mass of material which, because of the wide range of densities of the secondary materials, would lead to specimens of different materials being different sizes.

The optimum moisture content for compaction (OMC) and the maximum dry density (MDD) obtained are presented in Table 1 for the 13 mixtures tested. In some

Table 1 Selected mixtures, with optimum moisture content and maximum dry density

Mixture	*Aggregate/binder*	*Proportions*	*OMC (%)*	*MDD (Mg/m³)*
1	FA/C	FA 93%, C 7%	19.1	1.43
2	FA/CdC	FA 90%, Cd 7%, C 3%	19.4	1.51
3	FA/GyL	FA 91%, Gy 5%, L 4%	21.2	1.46
4	FA/CdGs	FA 70%, Cd 20%, Gs 10%	17.9	1.58
5	GY/FaC	GY 82%, Fa 10%, C 8%	14.1	1.36
6	CC/C	CC 93%, C 7%	9.8	2.01
7	CC/CdC	CC 90%, Cd 7%, C 3%	9.3	2.03
8	CC/FaL+	CC 80%, Fa 15.5%, L 4%, + 0.5%	9.2	1.95
9	CC/GgsLGy	CC 92.5%, Ggs 6%, L 1%, Gy 0.5%	9.2	1.96
10	T1/C	T1 96%, C 4%	6.5	2.14
11	T1/FaLGy	T1 92%, Fa 6%, L 1.5%, Gy 0.5%	6.5	2.29
12	BS/GsL	BS 84%, Gs 15%, L 1%	7.0	2.08
13	BS/GsSs	BS 70%, Gs 20%, Ss 10%	8.0	2.18

Bulk materials: FA = pfa, GY = gypsum, CC = china clay sand, T1 = granite Type 1, BS = air-cooled blastfurnace slag

Binders/activators: C = cement, L = quicklime, Fa = pfa, Gs = granulated blastfurnace slag, Ggs = ground granulated blastfurnace slag, Cd = cement kiln dust, Gy = FGD gypsum, Ss = steel slag fines, + = sodium carbonate

cases drainage during compaction prevented an OMC being measured and the value quoted is the moisture content at the MDD obtained.

3.2 Compressive strength of cubes

The compressive strength was determined on cubic specimens after different curing periods, according to BS 1924 (1990), except that most specimens were left in the mould longer than the specified one day because of the slower rate of strength development, compared with conventional materials. Specimens were compacted at OMC and then sealed and stored at 20°C until the end of the curing period. The results of the compressive strength tests on cubes are presented in Figure 1.

3.3 Repeated load indirect tensile tests

The Nottingham Asphalt Tester (NAT) was used, in indirect tensile (IT) mode, to assess the strength and stiffness of the test mixtures. This equipment is reasonably established for testing bituminous materials but some modifications were required for testing stabilised materials. The equipment and modifications are described in detail by Nunes (1997). Basically, a linear vertical load is applied diametrically to a cylindrical specimen, generating both vertical compressive strain and horizontal tensile stress. Specimens are compacted in the standard rig for the compactibility test, using a specially built compaction mould, which supports an inner mould of thin plastic. The same energy of compaction is used as during the compaction test. After compaction, specimens are stored for curing in a similar manner to the cubes. The results of the indirect tensile strength tests are presented in Figure 2.

Strength tests were carried out at 28, 90 and 360 days, and were performed using monotonic loading at a constant rate of 20kPa/s until failure occurred. The vertically applied loading and the vertical deformation are recorded throughout the test.

For the stiffness modulus test, the maximum load to be applied was limited to approximately 60% of the failure value, ascertained from the strength test, to be sure that the mixture was performing in the elastic range. In the stiffness test, five to six increasing stress levels are used and, at each level, the load is applied repeatedly for 15 cycles. The vertical load and horizontal deformation are recorded throughout the test. To quantify the stiffness modulus, a constant Poisson's ratio of 0.15 was assumed, based on the value measured on pfa mixtures. The results of the indirect tensile stiffness tests are presented in Figure 3.

The results of the IT strength and stiffness tests indicate that the relationship between the strength of a mixture and the stiffness of the mixture varies according to the size of the main aggregate used in the mixture; fine, medium or coarse.

3.4 Durability

The durability test for cement bound materials in the Specification for Highway Works (DOT, 1993) specifies comparing compressive strength of specimens immersed for 7 days after curing in the standard manner for 7 days, with that of specimens cured in the standard manner for 14 days. Due to the slow development of strength of some of the mixtures of secondary aggregates and binders, this durability test was adapted for the materials used in the project. The procedure adopted was:

- Three specimens were cured at 20°C for 21 days and then immersed in water, at the same temperature, for a further 7 days.
- All specimens were tested for indirect tensile stiffness modulus at 28 days.
- Two of the specimens were tested for indirect tensile strength at 28 days.
- The remaining specimen was immersed until 90 days, then tested for stiffness and strength.

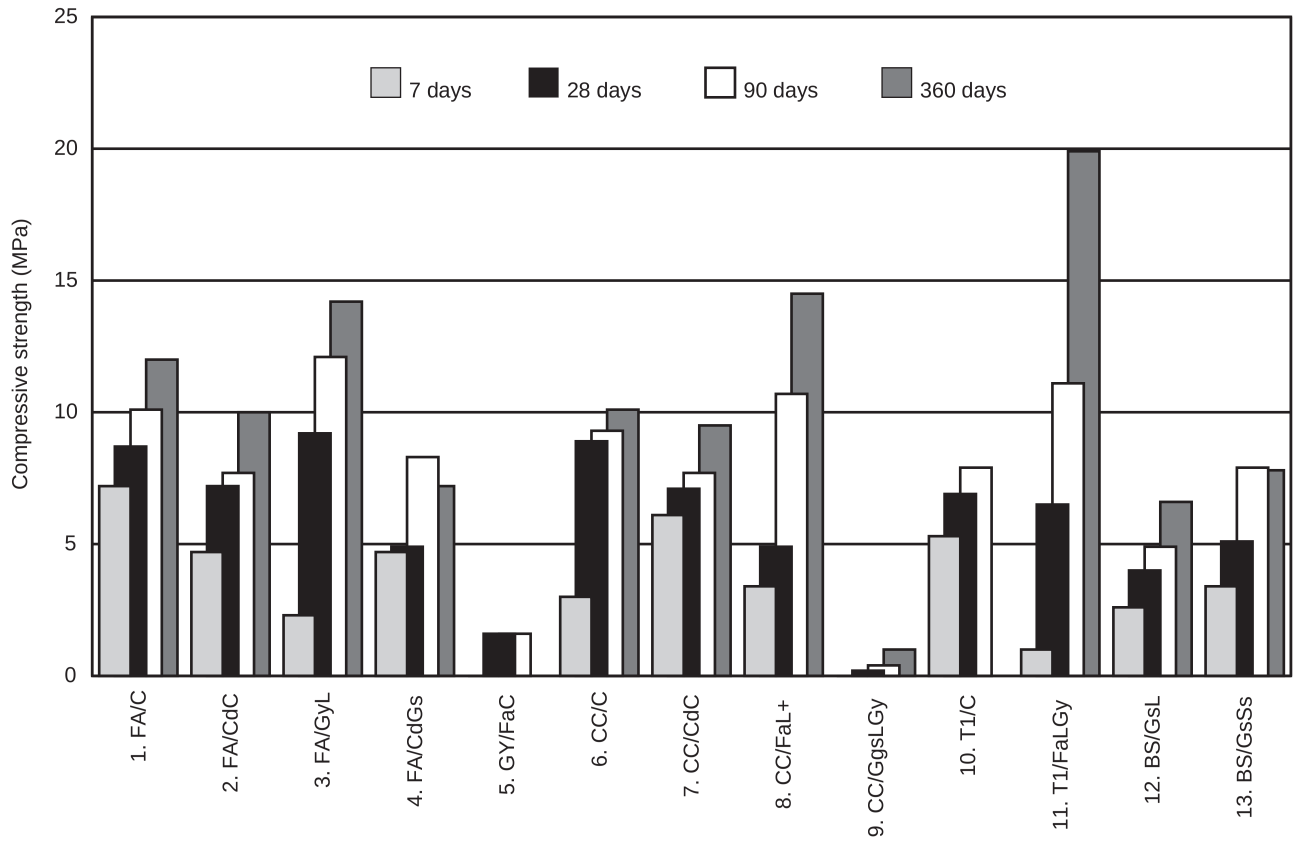

Figure 1 Cube compressive strength

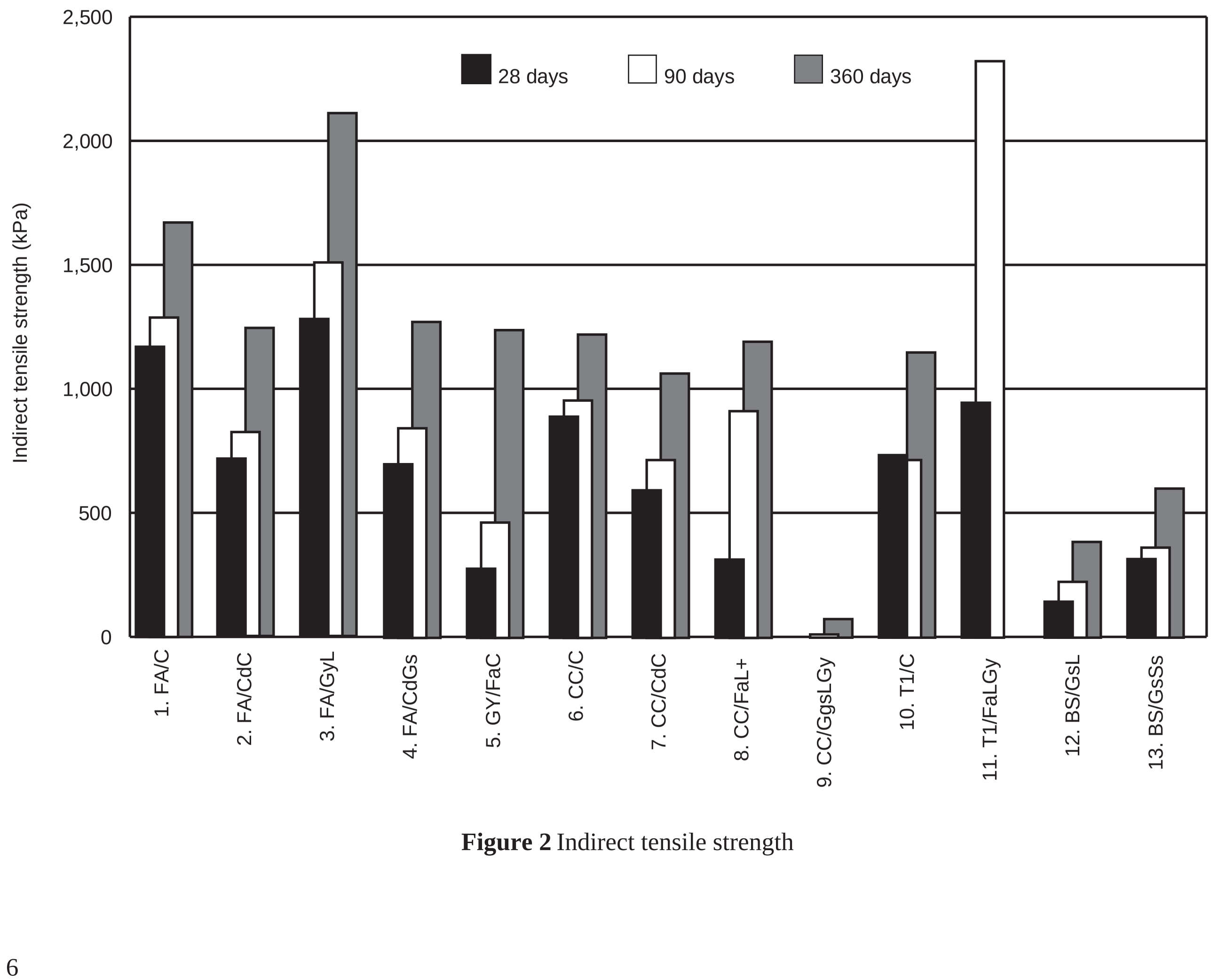

Figure 2 Indirect tensile strength

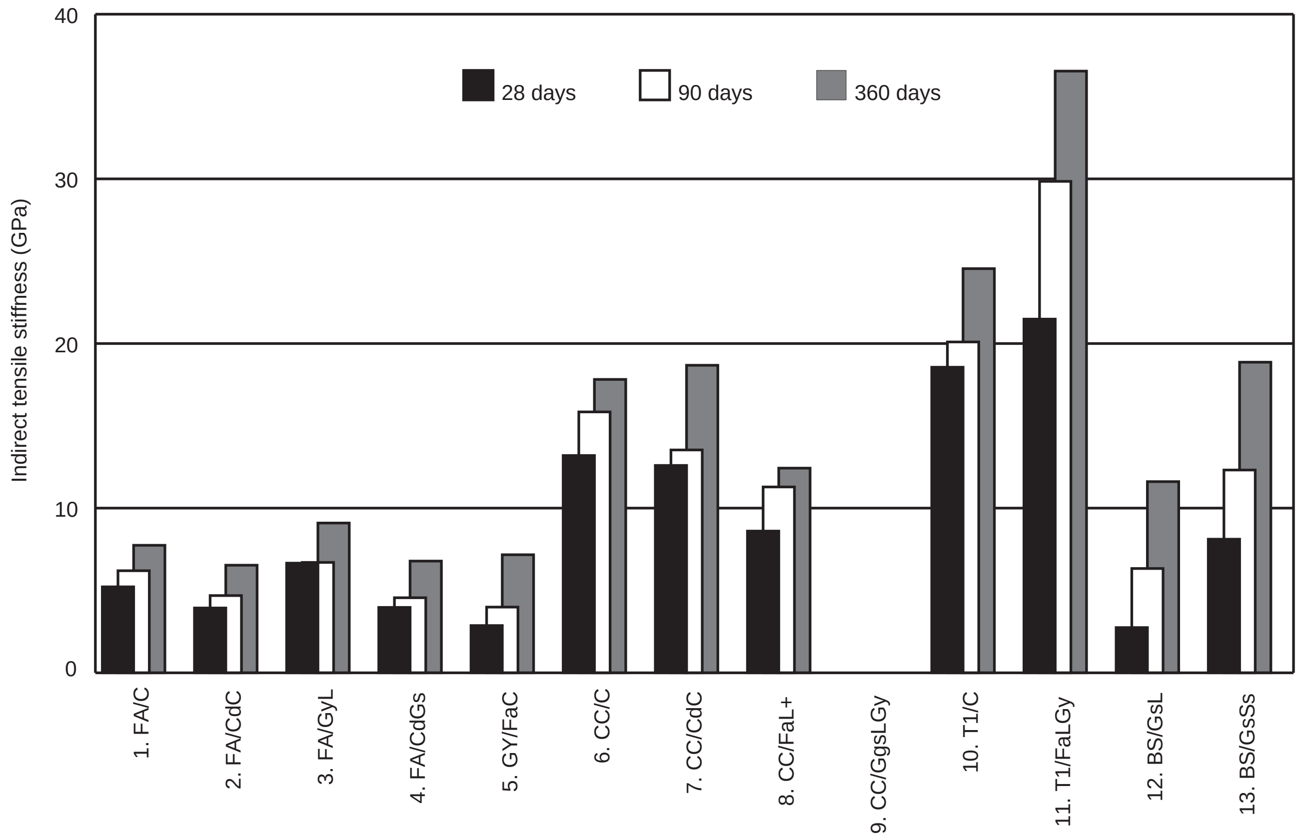

Figure 3 Indirect tensile stiffness

These specimens were compared with specimens cured in the normal manner at a constant temperature of 20°C. The results of the durability testing are presented in Table 2. Immersion during durability testing had the greatest effect on the performance of mixture 5 (GY/FaC).

4 Laboratory testing: Phase 2

Six of the original thirteen mixtures were selected for further testing and for use, as sub-base, in full-scale trials.

4.1 Selection of mixtures for further testing

In the Phase 1 laboratory tests all the mixtures performed well enough to indicate that they could be used as road construction materials, apart from mixture 9 (CC/GgsLGy). It is not clear why this mixture was initially unsuccessful, as similar mixtures were known to have performed well elsewhere. Further testing was carried out on a mixture with increased moisture and binder content, resulting in a 90 day IT strength of 2,280kPa and a 90 day stiffness of 31,161MPa, making it one of the strongest and stiffest mixtures. This successful composition of mixture 9 was obtained by increasing the ground granulated blastfurnace slag content to 10 percent and the quicklime content to 2.5 percent, with a compensating reduction in the amount of china clay sand. The moisture content was increased from 9.2 to 11.2 per cent.

Table 2 Durability

		Ratio - immersed / control (%)			
		Ratio of strength at:		*Ratio of stiffness at:*	
Mixture	*Aggregate/ binder*	*28 days*	*90 days*	*28 days*	*90 days*
1	FA/C	82	83	90	81
2	FA/CdC	71	103	102	102
3	FA/GyL	96	92	102	106
4	FA/CdGs	88	93	109	131
5	GY/FaC	63	63	89	75
6	CC/C	90	86	106	82
7	CC/CdC	102	107	102	106
8	CC/FaL+	151	99	117	107
9	CC/GgsLGy				
10	T1/C	109	142	114	99
11	T1/FaLGy	91	95	88	108
12	BS/GsL	122	119	114	145
13	BS/GsSs		87		89

Bulk materials: FA = pfa, GY = gypsum, CC = china clay sand, T1 = granite Type 1, BS = air-cooled blastfurnace slag.

Binders/activators: C = cement, L = quicklime, Fa = pfa, Gs = granulated blastfurnace slag, Ggs = ground granulated blastfurnace slag, Cd = cement kiln dust, Gy = FGD gypsum, Ss = steel slag fines, + = sodium carbonate

The selection of mixtures for Phase 2 testing was based on choosing mixtures with a range of properties encompassing the complete range of properties for the family to which they belong. The mixtures not chosen are considered to be equally adequate for use in pavement construction with the possible exception of mixture 5 (GY/FaC), as there was some concern over its durability. Two mixtures from each family were selected for further testing;

Fine aggregate:	Mixtures 3 and 4
Medium aggregate:	Mixtures 7 and 8
Coarse aggregate:	Mixtures 11 and 13

The objectives of the Phase 2 testing were:

- To determine the effects of small changes in moisture content and mixture proportions on structural properties.
- To determine the effect of low curing temperatures on the rate of development of the structural properties.
- To determine the time period during which the mixtures may be mixed, transported and laid.

To meet these objectives, up to four variants of each mixture, in terms of the proportions of the mixture constituents, were tested. These are denoted by a subscript added to the original mixture code. It was assumed that small variations in mixture proportioning would not greatly affect the OMC or MDD.

Observation of some of the broken specimens from the Phase 1 testing showed what appeared to be discrete particles of quicklime. This was thought to indicate that full hydration of binder had not taken place. As these specimens had been compacted at OMC it was assumed that the optimum moisture content for density is less than the optimum moisture content for hydration. For this reason the mixtures in Phase 2 were generally compacted just wet of conventional optimum. Mixture 11 was not treated in this way as it was found that the laboratory testing unusually gave evidence of segregation. Mixtures wetter than OMC are denoted by the subscript, w.

On the basis of the Phase 1 testing, the proportions of the mixture components were adjusted in order to add or remove strength and/or stiffness, to replace an expensive component with one which would provide similar performance but at a reduced cost or to change the rate of strength and/or stiffness development. The rationale was necessarily subjective but based on experience with similar materials.

4.2 Effect of changes in consituent proportions

The effect of increases in the moisture content and changes in the proportions of the mixture constituents was examined by indirect tensile strength and stiffness testing at 28 and 90 days. The variant of each mixture tested, in terms of proportions by mass, and the moisture content (mc) at which specimens were prepared is given in Table 3, together with the results of the IT tests.

4.2.1 Effect of mixing at a higher moisture content

Mixing at a higher moisture content increased the strength and stiffness of all mixtures except mixture 4, tending to confirm the assumption that a higher moisture content increases binder hydration. The decrease in compacted density, due to the wetter mixture, is greatest for mixture 4, and it may be that this offsets the hydration benefits, resulting in a decrease in strength for this mixture.

4.2.2 Effect of changed composition

Figures 4 and 5 illustrate the effect of changed composition on the strength and stiffness.

Mixtures 3, 4 and 13 were not very sensitive to changes in mixture proportions, in terms of both strength and stiffness. For mixture 3, adding less binder reduced strength and stiffness slightly, and adding more gypsum but less quicklime reduced strength but did not effect stiffness. For mixture 4, the stiffness was slightly increased and the strength slightly reduced by adding more granulated slag and less cement kiln dust (4_{aw}), and by adding more pulverised fuel ash and less cement kiln dust (4_{bw}). The wetter mixture 4 variants appeared spongy indicating positive pore pressure. Mixture 13 was little affected by the changes in proportion of the different slags.

The strength and stiffness of mixture 7 varied with the amount of binder; 7_{aw}, with more binder, was stronger and stiffer than 7_w, while 7_{bw}, with less binder, was weaker and less stiff. Mixture 7_{cw} appeared to behave much as 7_w.

The strength and stiffness of mixture 8 were expected to develop more rapidly with increased amounts of sodium carbonate but 8_{aw} showed no evidence that this happened. Adding more pfa and quicklime, 8_{bw}, made for a fast setting, stronger and stiffer mixture. Adding less binder, 8_{cw}, caused a corresponding reduction in strength but little change in stiffness.

Mixture 11 was weakened by adding less binder, as in 11_a and 11_c, the mixture showing some segregation and voids because of the deficiency of fines. The effect of adding a small amount extra of gypsum and quicklime, 11_b, was modest and there was little evidence that faster strength development was achieved.

4.3 Low temperature curing

In order to assess the effect of constructing at different seasons of the year, low temperature curing was investigated on one mixture from each family. It was expected that the rate of hydration would be reduced at low temperatures. Curing was carried out at 10°C and specimens were tested for indirect tensile strength and stiffness at 28 and 90 days. The results are compared to those of specimens cured at 20°C in Table 4.

Mixtures 4_w and 7_{aw} were little affected by the low temperature curing, while mixture 11_c was affected significantly, even after 90 days. The low relative 28 day strength of mixtures 3_{aw}, 8_{aw} and 13_{aw} could, perhaps, be compensated for by laying thicker material in colder conditions, and is largely recovered by 90 days.

In general binders based on pfa and quicklime cure more slowly at low temperature, while cement kiln dust with cement is little affected by low temperature curing.

4.4 Workability testing

Workability testing was carried out on one mixture from each family, to give an indication of the time which could

Table 3 Strength, stiffness and density for mixture variations

Constituents	Mixture code	Proportions (% by mass)	mc (%)	IT Strength (kPa) 28 days	IT Strength (kPa) 90 days	IT Stiffness (MPa) 28 days	IT Stiffness (MPa) 90 days	Dry density (Mg/m^3)
FA/GyL:	3	91,5,4	21.2	1,278	1,506	6,662	6,706	1.462
	3_{w}	91,5,4	22.2	1,603	1,829	8,901	9,040	1.426
	3_{aw}	93,4,3	22.2	1,342	1,470	7,621	7,122	1.407
	3_{bw}	90,7,3	22.2		1,458		8,789	1.431
FA/CdGs:	4	70,20,10	17.9	700	845	3,974	4,563	1.575
	4_{w}	70,20,10	19.9	577	763	4,170	4,522	1.508
	4_{aw}	70,10,20	19.9		611		5,325	1.750
	4_{bw}	80,10,10	19.9		593		5,692	1.575
CC/CdC:	7	90,7,3	9.3	595	717	12,594	13,534	2.030
	7_{w}	90,7,3	10.3	768	1,487	15,950		2.067
	7_{aw}	87,9,4	10.3	1,358	379	19,040	22,507	2.071
	7_{bw}	93,5,2	10.3		920		9,792	1.999
	7_{cw}	89,9,2	10.3				16,499	2.103
CC/FaL+:	8	80,15.5,4,0.5	9.2	316	914	8,609	11,290	1.950
	8_{w}	80,15.5,4,0.5	11.2	1,206	2,028	15,930	19,524	1.979
	8_{aw}	80,15.5,3.5,1	11.2	1,472	1,903	16,833	23,256	2.009
	8_{bw}	74,20.5,5,0.5	11.2	1,858	1,239	19,035	17,862	1.974
	8_{cw}	85,11.5,3,0.5	11.2	797		16,043		2.045
T1/FaLGy:	11	92,6,1.5,0.5 9	6.5	947	2,324	21,483	29,850	2.290
	11_{a}	4.67,4,1,0.33	6.5	353	1,059	10,879	20,290	2.195
	11_{b}	91,6,2,1	6.5	1,015	2,486	23,708	30,146	2.220
	11_{c}	93.33,5,1.26, 0.41	6.5	502	1,599	17,253	27,299	2.136
BS/GsSs:	13	70,20,10	8.0	317	363	8,115	12,316	2.178
	13_{w}	70,20,10	9.0	480		16,192	15,911	2.167
	13_{aw}	60,30,10	9.3*	431	560	14,404	15,832	2.052
	13_{bw}	79,14,7	9.0		604		14,495	2.404
	13_{cw}	60,20,20	9.0		562			2.191

Bulk materials: FA = pfa, CC = china clay sand, T1 = granite Type 1, BS = air-cooled blastfurnace slag

Binders/activators: C = cement, L = quicklime, Fa = pfa, Gs = granulated blastfurnace slag, Cd = cement kiln dust, Gy = FGD gypsum, Ss = steel slag fines, + = sodium carbonate

* *Compactibility re-tested due to change in aggregate proportions, MDD 2.0Mg/m3*

be allowed between mixing and completing compaction of sub-bases constructed from the trial mixtures. Material was sampled from the same wet source mixture, stored in air tight bags, and then compacted at 2, 4, 7 and 24 hours after mixing. After curing at 20°C specimens were tested at 28 days for indirect tensile strength and stiffness. This enabled the workability time of the hydraulically bound mixture, during which it is possible to use and compact the mixture with little deterioration of its long-term mechanical properties, to be estimated.

The time allowed between mixing and compaction for standard cement bound materials is two hours. The workability time is therefore taken as being the time between mixing and compaction which causes no more than a 10 per cent drop, in either strength or in stiffness, when compared to that of material compacted two hours after mixing. Table 5 gives the workability times estimated from this approach. These times also indicate whether materials need to be mixed on site or can be transported from a mixing plant elsewhere.

Experience elsewhere in Europe suggests that the workability times of materials such as mixtures 8 and 11, both inert aggregate bound with pfa and quicklime, are, in practice, longer than those indicated in Table 5.

5 Selection of mixtures for full scale trials

As in the selection of mixtures for Phase 2 testing, the variant of each Phase 2 mixture used in the full-scale trials was chosen to ensure that the range of strength and stiffnesses exhibited by each family was covered. Where two mixtures had similar performance, the cheaper variant was selected.

Information was required on the performance of sub-bases when trafficked immediately after construction and when trafficked after a curing period of seven days was allowed to permit the sub-base to gain strength by cementation. One material from each aggregate family was selected to be trafficked in each manner to provide a wide range of information to guide the use of these materials in different road construction situations. Selection for immediate trafficking or trafficking after seven days curing (delayed trafficking), was based on the results of the workability tests.

To assess the performance of the sub-bases constructed using secondary aggregates and binders it was necessary to

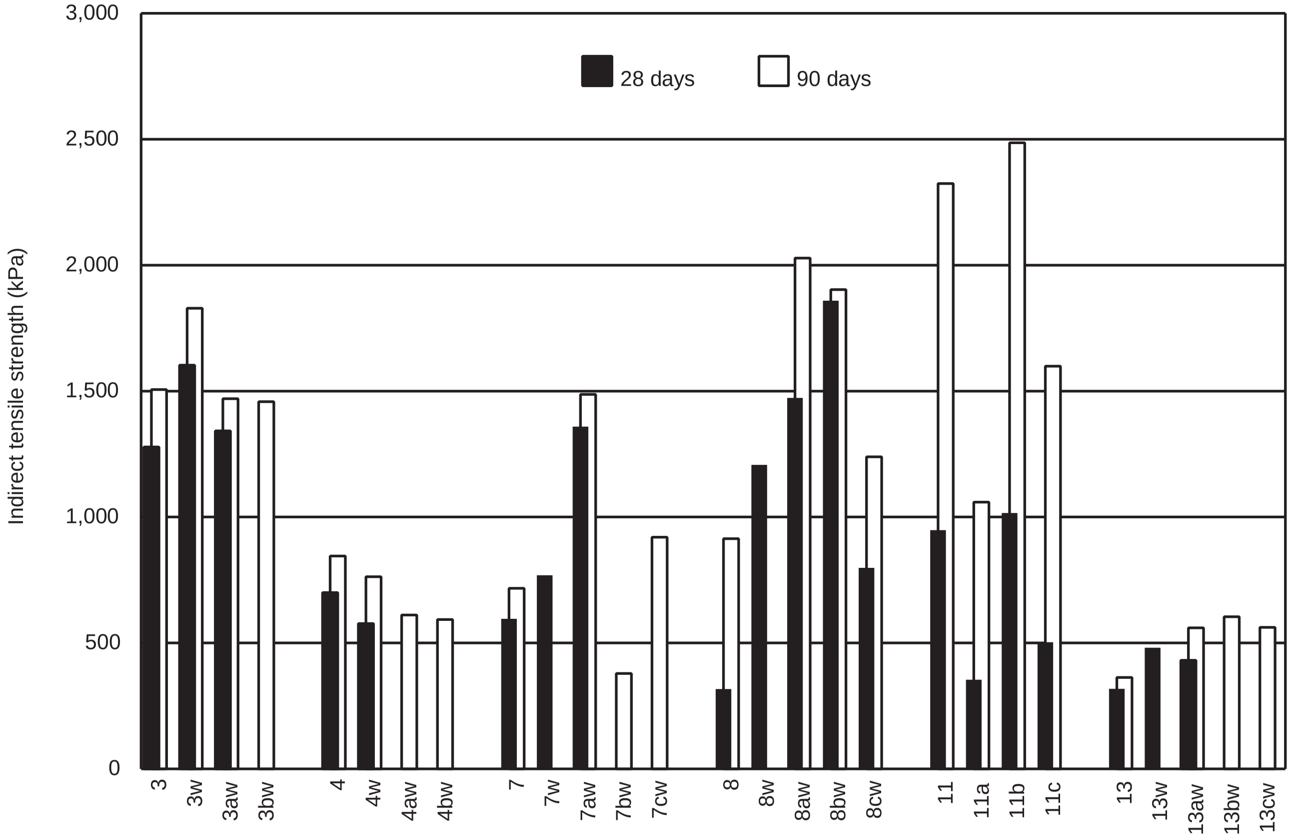

Figure 4 Effect on strength of changed composition

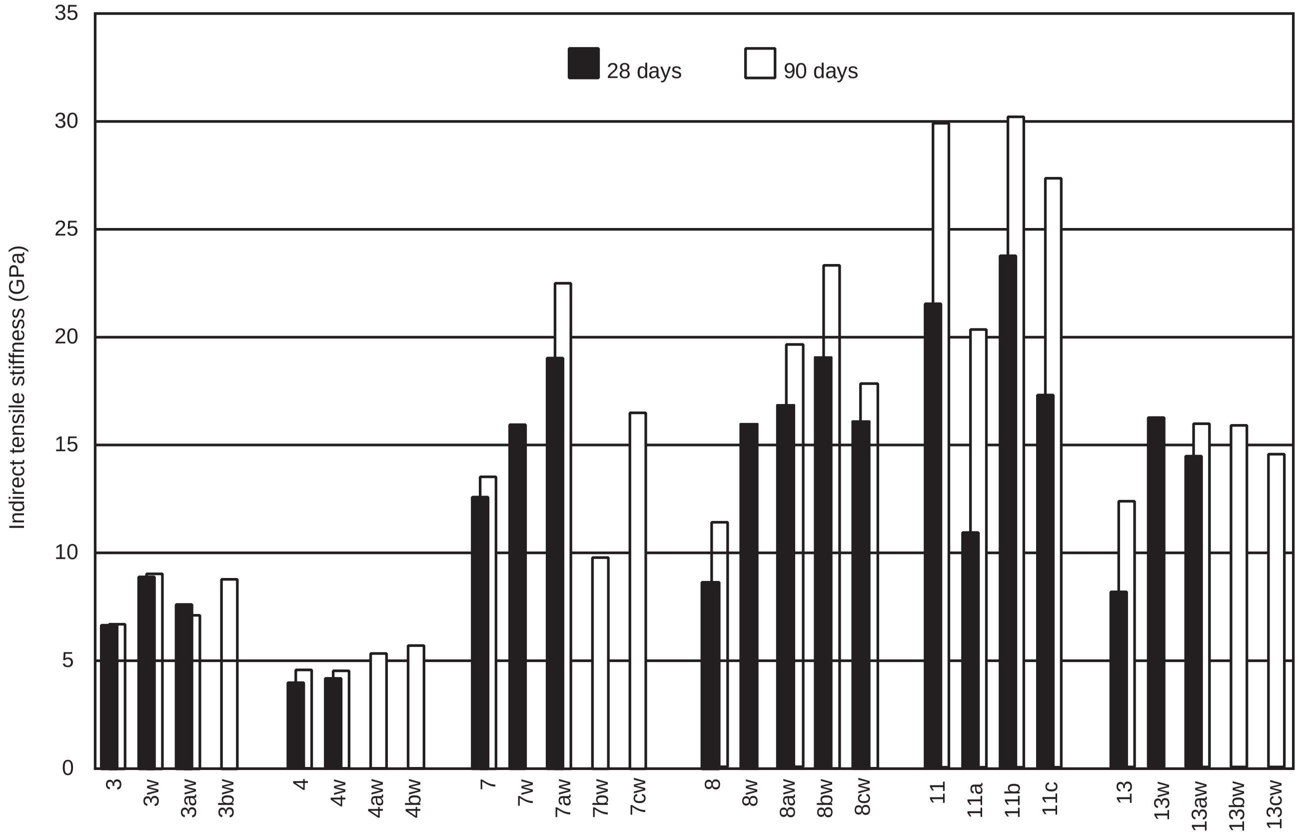

Figure 5 Effect on stiffness of changed composition

Table 4 Ratio of indirect tensile results, 10°C curing/20°C curing

	Ratio of strength (10°C/20°C)		Ratio of stiffness (10°C/20°C)	
Mixture	*28 days*	*90 days*	*28 days*	*90 days*
3_{aw}, FA/GyL	0.64	0.95	0.88	0.96
4_{w}, FA/CdGs	0.96	0.77	1.02	0.95
7_{aw}, CC/CdC	0.97	0.97	0.99	0.95
8_{aw}, CC/FaL+	0.36	0.89	0.71	0.89
11_{c}, T1/FaLGy	0.34	0.55	0.44	0.81
13_{aw}, BS/GsSs	0.78	1.00	0.46	0.81

Bulk materials: FA = pfa, CC = china clay sand, T1 = granite Type 1, BS = air-cooled blastfurnace slag

Binders/activators: C = cement, L = quicklime, Fa = pfa, Gs = granulated blastfurnace slag, Cd = cement kiln dust, Gy = FGD gypsum, Ss = steel slag fines, + = sodium carbonate

Table 5 Estimated workability times based on indirect tensile testing

Mixture	*Workability time (hours)*
3_{aw}	24
4_{w}	4
7_{aw}	5
8_{aw}	8.5
11_{c}	3
13_{aw}	>24

construct sub-bases of conventional materials against which to compare their performance. Two control sub-bases were constructed, one from unbound granular, Type 1 material and one from cement bound material, CBM1 (MCHW 1 Series 800 and 1000).

The materials were combined in the trial mixtures as summarised in Table 6, which also indicates which mixtures were chosen for immediate trafficking and which for delayed trafficking. The target proportions of the component materials are based on their dry weights. A mix design for the CBM1 sub-base material established that 95 per cent sand to 5 per cent cement should form a material of the appropriate strength when mixed at a moisture content of 9.8 per cent.

6 Full scale trials in the PTF

Trial and control sub-bases were constructed and tested at TRL's Pavement Test Facility, which allows road foundations and pavements to be trafficked by a moving wheel under controlled conditions of speed and wheel load.

The purpose of the PTF trials was to obtain information on the large scale mixing and laying of sub-bases comprising secondary aggregates and binders, and to assess the performance of the foundations resulting from the use of these materials as sub-base. A further aim was to provide information on the thickness of sub-base necessary to limit to acceptable levels the amount of rutting and/or cracking in the sub-base material, as this would be detrimental to the subsequent performance of a road pavement. Each trial sub-base was therefore laid as a wedge of varying thickness along the trial section. The wedge was achieved by shaping the subgrade. A uniform subgrade was required on which to construct the trial sub-bases and the subgrade design CBR was 2.5 percent, a low value being chosen to facilitate foundation failure.

For ease of reference the mixture codes are also used for the foundations and sub-bases constructed with those mixtures.

6.1 Sub-base design

The wedge dimensions of the sub-bases were chosen so that the thinner end was expected to fail under trafficking while the thicker end was expected to be at least appropriate for the subgrade strength and envisaged construction traffic. The foundation in this region was expected to support the loading without excessive deterioration and to provide information on the contribution a thicker sub-base could make to the pavement structure. As the stiffness modulus of the trial materials increases with their age, sub-bases to be trafficked immediately after construction were built thicker than those designed to cure before carrying traffic. Wedge dimensions were also chosen to give an overlap between the thickness of sub-base for immediate trafficking and the thickness for delayed trafficking. The chosen dimensions were:

Table 6 Trial materials

Family	*Mixture*	*Components*	*Proportions (%)*	*Trafficking*	*Moisture content (%)*
Pulverised	3_{aw}	FA/GyL	93,4,3	Immediate	22.2
fuel ash	4_{bw}	FA/CdGs	80,10,10	Delayed	19.9
China clay	7_{cw}	CC/CdC	89,9,2	Delayed	10.3
sand	8_{cw}	CC/FaL+	85,11.5,3,0.5	Immediate	11.2
Coarse	11	T1/FaLGy	92,6,1.5,0.5	Delayed	6.5
aggregate	13_{w}	Bs/GsSs	70,20,10	Immediate	9.0
Controls	T1	T1	100	Immediate	As supplied
	CBM1	CC/C	95,5	Delayed	9.8

Bulk materials: FA = pfa, CC = china clay sand, T1 = granite Type 1, BS = air-cooled blastfurnace slag

Binders/activators: C = cement, L = quicklime, Fa = pfa, Gs = granulated blastfurnace slag, Cd = cement kiln dust, Gy = FGD gypsum, Ss = steel slag fines, + = sodium carbonate

Immediate trafficking	Sub-bases 3_{aw}, 8_{cw}, 13_{w}, Type 1:	300 to 550mm
Delayed trafficking	Sub-bases 4_{bw}, 7_{cw}, CBM1:	150 to 350mm
	Sub-base 11:	250 to 375mm

6.2 Subgrade

The subgrade for the trial foundations was London Clay, from two sources with comparable plasticity characteristics. According to Casagrande's extended soil classification system the soil is described as Type CH, a clay of high plasticity. The clay was rotovated, wetted, rotovated and compacted in layers to provide a uniform subgrade.

The plasticity characteristics of the two sources of London clay are given in Table 7.

Table 7 Plasticity characteristics of subgrade

	Source 1	*Source 2*
Liquid limit (%)	72	62
Plastic limit (%)	26	22
Plasticity Index (%)	46	40

During the process of conditioning the clay, the MEXE cone penetrometer, described by Black (1979), was fitted with the large cone and used to test clay at a variety of moisture contents. 'Undisturbed' specimens were then recovered by extracting in-situ clay with the CBR mould, and their CBR value measured in the laboratory. This enabled an approximate relationship between cone penetrometer index (CI) and CBR to be established:

$$\text{CBR (\%)} = 2.64\text{Log(CI)} - 1.86 \qquad (1)$$

where CI is the average cone index reading over a depth of 75mm.

The subgrade was graded so that the sub-base in each trial foundation would form a wedge of the dimensions given in Section 6.1. MEXE cone penetrometer measurements were then made along each prepared bay, and used to calculate the average CBR values at the top of the subgrade. Table 8 shows that there was very good uniformity across the bays.

At the end of the experimental programme the sub-bases were cored and MEXE cone penetrometer measurements were made through each core hole. The average CBR values, given in Table 8, show a slight increase in strength, probably due to consolidation. It should be noticed that the CBR values obtained after trafficking was completed were based on a small number of measurements.

6.3 Production and construction of sub-bases

The sources of the component materials are listed in Appendix A, Table A1. Sub-base materials were mixed in four locations, transported to the PTF and laid in pre-prepared bays.

6.3.1 Production of sub-base mixtures

Mixtures 3_{aw} and 4_{bw}, with pulverised fuel ash as the largest component, were produced in a continuous flow, forced action mixer by Wrekin Construction Ltd. at Dudley, West Midlands.

Mixture 3_{aw} comprised 93 per cent pfa, 4 per cent gypsum and 3 per cent quicklime. The pfa and gypsum were placed in separate aggregate hoppers while the quicklime, which was delivered dry in a tanker, was blown into a silo. To achieve the target proportions, the in-situ moisture contents of the materials were measured. The pfa and gypsum were initially loaded into hoppers and blended 50:50 by dry weight. This mixture was placed in the hopper which had previously contained the gypsum and blended 8:89 with pfa, the required proportion of quicklime being simultaneously blown in from the silo. The equipment was calibrated by determining the rates of supply of the component materials for specific settings, by selecting each hopper in turn and weighing the quantity of material passing along the conveyor in a given time. Water was added at a preselected rate, taking account of the in-situ moisture content of the materials, to raise the moisture content of the mixture. Moisture contents of the mixed material were measured using a microwave oven and the water supply adjusted to more closely achieve the target value.

Mixture 4_{bw} comprised pfa, cement kiln dust and granulated slag. The pfa and granulated slag were placed in separate hoppers and the 'dry' cement kiln dust in the silo. These components were blended together, with added water, in a single process, with calibration as described previously for mixture 3_{aw}. The cement kiln dust did not flow freely, apparently because it was not completely dry. The final proportions of the pfa, cement kiln dust and granulated slag were approximately 81 per cent, 9 per cent and 10 per cent, slightly different from the target values. Both mixtures appeared to be uniform and compaction in moulds by vibrating hammer gave no evidence of excessive water.

Mixture 13_{w} comprised blastfurnace slag, granulated slag and steel slag fines and was manufactured in an asphalt plant at Llanwern Works, Gwent by Cambrian Stone Ltd., a joint venture company owned by Tarmac Heavy Building Materials (UK) Ltd. and British Steel plc. Mixtures of the same proportions of components as given in Table 6 were prepared in the laboratory over a range of moisture contents at and above optimum. These mixtures

Table 8 Average subgrade CBR for trial sections (%)

	3_{aw}	*8_{cw}*	*13_{w}*	*Type 1*	*11*	*4_{bw}*	*CBM1*	*7_{cw}*
Before sub-base construction	3.0	2.9	2.9	3.0	3.0	3.0	3.0	3.0
After trafficking completed	3.2	3.1	3.3	3.2	3.3	3.6	3.4	3.7

were formed into cones and observed for loss of water and fines, to select the appropriate mixture moisture content. Prior to production, the steel slag aggregates were screened to provide a steel slag fine aggregate. The steel slag fine aggregate, granulated slag and air-cooled blastfurnace slag fine aggregate were loaded into separate ground feed hoppers and fed as a predetermined blend into the fine aggregate bin on the asphalt plant. The blastfurnace slag coarse aggregate single sizes were fed over the asphalt plant screens and collected in the individual asphalt plant bins. Aggregates from the bins were then proportioned by weight in the aggregate weigh hopper and batched into the pugmill mixer with additional water and a small proportion of filler. The additional water was introduced from a piped supply calibrated in kg per second.

The *CBM1* control sub-base comprised sand and cement and was produced by RMC plc., Eversley, Hampshire. China clay sand from Lee Moor Quarry, Plymouth in Devon was mixed with cement and water in a forced action pan mixer in five batches. When mixed at the design moisture content and compacted in a mould by a vibrating hammer the material was found to be unstable. Its moisture content was therefore reduced, by less than one per cent of dry weight, to facilitate compaction whilst leaving sufficient water for hydration of the cement.

Mixtures 7_{cw}, 8_{cw} *and 11,* (china clay sand with cement kiln dust and cement; china clay sand with pfa, lime and sodium carbonate; and Type 1 crushed granite with pfa, lime and gypsum) were produced in a small batch, paddle mixer at TRL, in 0.75 tonne batches.

Of the component materials the quicklime, cement and sodium carbonate were delivered dry. The cement kiln dust was at a sufficiently low moisture content and used in sufficiently small quantities to be considered dry. The bulk materials of china clay sand (mixtures 7_{cw} and 8_{cw}) and Type 1 crushed rock (mixture 11) were placed in a hopper by a loading shovel and a quantity weighed for each batch. The other components were pre-weighed for each batch and sealed in polythene bags. The calculated quantity of each component material took into account its in-situ moisture content. The component materials were initially well mixed together prior to adding the required amount of water to obtain the target moisture content. The small amount of sodium carbonate in mixture 8_{cw} was sprinkled onto the other components during the initial mixing, to ensure an even distribution throughout the mixture. The gypsum in mixture 11 was also added in this manner.

For each of the three mixtures a specimen from the first batch was compacted in a mould with a vibrating hammer to examine the stability. The moisture contents of all three mixtures were consequently reduced to improve their stability during construction. However, these changes were limited to a maximum of one per cent of dry weight so as to limit the reduction in material strength due to insufficient water for complete hydration.

6.3.2 Construction

Shuttering, lined with compressible building board, was used to build a separate bay in which to construct each sub-base. The materials produced by off site mixers were initially tipped on wetted hard standing and covered by sheets to prevent drying, prior to being transported into the PTF. The materials produced by the batch mixer at TRL were transported immediately into the PTF following production.

Each mixture was transported into the PTF by dumper and tipped, spread and compacted into the appropriate bay. Sub-bases were constructed in two or three layers, depending upon the required thickness. The surfaces of the lower layers of the bound mixtures were loosened before spreading an overlying layer, to help bonding. A Bomag 80/60D reversing plate was used to compact the sub-bases. The number of passes adopted was the same as that specified for the same thickness of Type 1 sub-base (MCHW1, Series 800). When the layer varied in thickness along the bay, the compaction required for the thicker part of the layer was applied over the complete layer.

The pfa based sub-bases, 3_{aw} and 4_{bw}, were compacted without difficulty, although it was later found that their densities were rather low, especially in the case of 3_{aw}. The top layer of each of these sub-bases was laid above the target level and cut back to the appropriate finished level after completion of compaction. The china clay sand mixtures, 7_{cw}, 8_{cw} and CBM1, proved difficult to compact. Their high moisture contents resulted in an unstable mixture and held back the vibrating plate, which often had to be pushed along manually. Only about two thirds of the designated passes could be applied to the upper layer of the CBM1 control material. The all slag material, mixture 13_w, was also found to be unstable during compaction and this was considered to be attributable to the target moisture content being above optimum. Only half the number of specified passes could be applied to the upper layer of this material.

The difficulties experienced during compaction may well have been due to the use of the vibrating plate, which was necessary because of the restricted access in the Pavement Test Facility. In the field more suitable compaction plant would be used, such as a rubber tyred roller.

The unbound granular Type 1 sub-base control and mixture 11, composed primarily of this well graded crushed rock, were constructed easily.

For the china clay sand and coarse aggregate mixtures, final compaction with a light weight pedestrian roller was necessary to achieve a reasonably level surface finish.

All layers of most sub-bases were completely constructed within the estimated workability times of the bottom layer (Table 6). For mixture 8_{cw}, although each layer was constructed within the workability time of the material contained in that layer, the upper layer was completed about three hours after the expiry of the workability time of the material in the bottom layer. For the CBM1 the permitted time between mixing and compaction is normally 2 hours, which was exceeded by 1¼ hours.

The CBM1 sub-base was shown to be adequately compacted as the average in-situ bulk wet density, measured by a nuclear density gauge, was at least 95 per cent of the average bulk wet density of specimens compacted to refusal in cube moulds.

The TRL core scanner was used to determine the in-situ dry densities of cores cut from the trial sections on completion of the trials. These densities were measured between 50 and 80 days after construction of the trial sections. The dry densities of the laboratory specimens, however, were deduced from the wet bulk densities of compacted specimens and the moisture content of material surplus to the specimen preparation and were therefore representative of the laboratory specimens in their early life. As the dry density of stabilised materials increases with their age (MCHW 2, Clause NG 1003), unless hydration causes expansion, the in-situ and laboratory densities given in Table 9 are not exactly comparable. Despite this difference, these results show that the in-situ densities are broadly similar for all mixtures except 3_{aw} whose in-situ dry density was 14 per cent lower than that determined for the laboratory specimens.

Table 9 Dry densities of stabilised materials

	Dry densities (Mg/m3) of:	
Mixture	*Cores*	*Laboratory specimens*
3_{aw}	1.210	1.407
4_{bw}	1.497	1.575
7_{cw}	2.078	2.103
8_{cw}	2.078	2.045
11	2.279	2.290
13_{w}	2.234	2.167
CBM1	2.027	1.972

The compacted thicknesses of the sub-bases along the trial sections are shown in Figure 6 and can be seen to agree closely with the target values.

6.4 Laboratory assessment

Mixture 3_{aw} was assessed for immediate trafficability. Also strength and stiffness tests were carried out on specimens made from samples of the sub-base mixtures, taken during construction of the trial foundations, and compacted in moulds. Following the trafficking cores were cut from the trial sub-bases and their strength and stiffness was measured in the laboratory to compare with the target values determined in Phase 2.

6.4.1 Immediate trafficability

As a guide to the immediate trafficability of the pfa based mixture 3_{aw}, a laboratory CBR test without surcharge was conducted. The average CBR value was 23 per cent. Values of at least 25 per cent and preferably 40 per cent are normally required. Although marginal, the measured value, when taken with an earlier result of 34 per cent on mixture 4_{bw}, supported the aim of attempting trafficking 3_{aw} immediately after construction.

The laboratory CBR test was performed on a sample of the material compacted in the mould. The density of the CBR specimen was, therefore, not necessarily the same density as the material compacted as sub-base, which subsequently failed to support immediate trafficking. Elsewhere, similar mixtures have been successfully trafficked immediately after laying, although these have

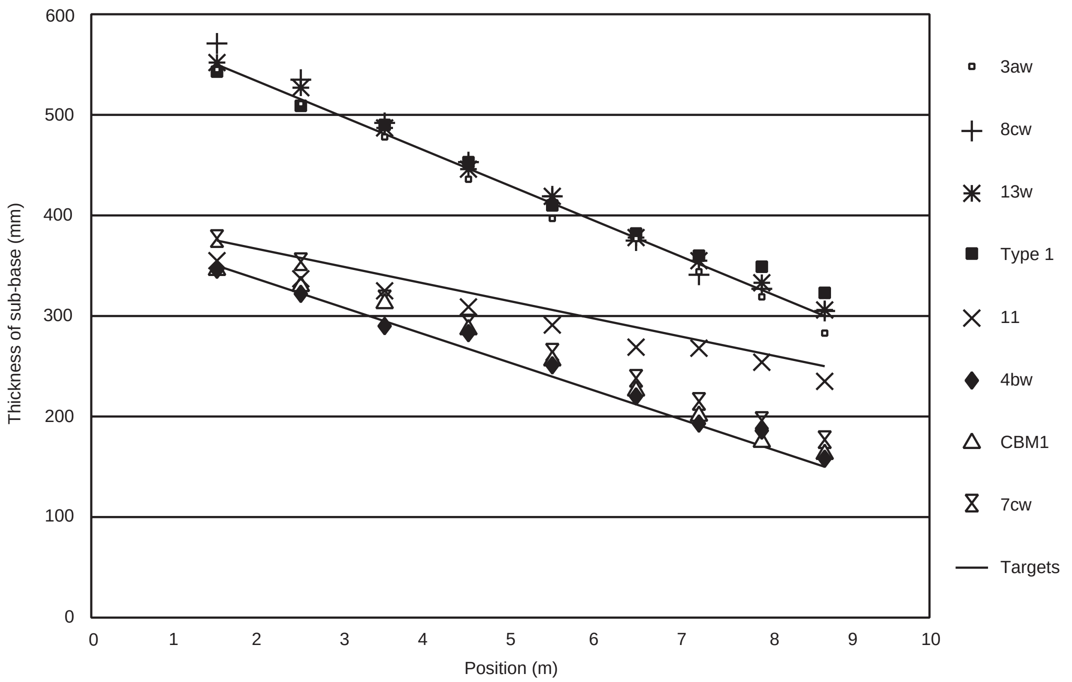

Figure 6 Thickness of sub-base along trial sections

been laid on material of higher bearing capacity than that of the PTF subgrade (Thijs, 1984).

To assess trafficability, CBR specimens could be compacted in the laboratory at the measured in-situ density, or an in-situ CBR test could be carried out. Alternatively, FWD testing could be performed. Foundation surface modulus values calculated from FWD testing appear to give a better indication of trafficability than CBR tests (section 6.6.1).

6.4.2 Moulded and cored specimens

The indirect tensile strength and stiffness of specimens, compacted in moulds, of material produced for the PTF trials, are given in Table 10, together with the target values obtained from the Phase 2 laboratory testing of these mixtures. Where cores were taken from the trial sub-bases, after trafficking was completed, the results of strength and stiffness tests on the cored material are also given.

The indirect tensile strength values of moulded specimens of the material laid as sub-base, were within approximately 20 per cent of target values for specimens of mixtures 4_{bw}, 7_{cw}, 11 and 13_{w}. The average strength of the trial moulded specimens of mixture 3_{aw} was approximately 40 per cent lower than the target value. However, the average strength of trial moulded specimens of mixture 8_{cw} was approximately 90 per cent higher than the target. Although the agreement between the measured properties of the trial mixtures and their target values is mixed, the objective of achieving a wide range in the strengths and stiffnesses of these mixtures was achieved.

The average compressive cube strength at 7 days of the CBM1 control sub-base was 6.7MPa, showing the material to comply with the requirements given in MCDHW1 (Series 1000) of minimum strength 4.5MPa.

6.5 Loading

Loading was applied to the trial foundations by a dual wheel travelling at 20kph. The trafficking was canalised, with the wheel tracks superimposed without lateral distribution of the wheel across the foundation, to simulate the worst possible conditions that may occur in practice.

HD 25 (DMRB 7.2.2) recommends that sub-bases should be designed to support construction traffic loading of 1000 standard axles. In the full-scale trials traffic loading exceeded this value. Loading was initially applied at a wheel load of 40kN (equivalent to 1 standard axle) and then increased to 56.4kN (equivalent to an axle load of 112.8 kN, or a mass of 11.5 tonnes; the maximum load permissible from 1999). The number of passes applied was calculated assuming a fourth power law, which indicates that, in terms of pavement wear, one pass of an 11.5 tonne axle load is equivalent to approximately four passes of a

Table 10 Average strengths and stiffnesses of sub-bases

		Strength (kPa) at:			*Stiffness (MPa) at:*	
Material	*Source*	*7 days*	*28 days*	*90 days*	*28 days*	*90 days*
3_{aw}, FA/GyL	Target	-	1,341	1,470	7,621	7,122
	Trial	398	755	-	4,314	-
	Cores	-	414*	-	2,053*	-
4_{bw}, FA/CdGs	Target	-	491+	593	5,037+	5,692
	Trial	354	427	-	3,798	-
	Cores	-	-	213*	-	1,876*
7_{cw}, CC/CdC	Target	-	763+	920	15,316+	16,499
	Trial	522	781	-	10,458	-
	Cores	-	-	835	-	13,750
8_{cw}, CC/FaL+	Target	-	797	1,239	16,043	17,862
	Trial	776	1,516	-	16,215	-
	Cores	-	1,233*	-	13,850*	-
11, T1/FaLGy	Target	-	947	2,324	21,483	29,850
	Trial	169	1179	-	13,994	-
	Cores	-	-	1,558	-	27,983
13_{w}, Bs/GsSs	Target	-	480	562+	16,192	26,361+
	Trial	320	564	-	6,685	-
	Cores	-	380*	-	3,788*	-
CBM1, CC/C	Target	-	789	843	12,450	14,402
	Trial	488	722	-	10,072	-
	Cores	-	-	469	-	15,303

* *Value adjusted to designated age*
+ *Value predicted from best fit curves*

Bulk materials: FA = pfa, CC = china clay sand, T1 = granite Type 1, BS = air-cooled blastfurnace slag

Binders/activators: C = cement, L = quicklime, Fa = pfa, Gs = granulated blastfurnace slag, Cd = cement kiln dust, Gy = FGD gypsum, Ss = steel slag fines, + = sodium carbonate standard axle. Using this calculation each sub-base received at least 5,000 standard axles of traffic loading. It is uncertain whether this power law holds for the type of structures tested here but, as the actual number of passes of the loaded wheel exceeded 1,000, the applied traffic was considered to adequately test the performance as sub-base.

If the foundations remained without deterioration, after application of 5,000 standard axles, the loading was increased to 95kN to induce cracking, for a maximum of 1000 passes. If the sub-base cracked under this loading, it was then trafficked by a further 1000 standard axles so that the behaviour of the cracked material could be observed.

Each trial sub-base was trafficked in approximately the same manner, the first 5000 standard axles being completed within three weeks of commencement of trafficking.

6.6 Performance of trial and control sub-bases

The performance of the trial sub-bases was assessed by measuring foundation deformation relative to a fixed datum, rutting (peak to trough), cracking and surface modulus calculated from FWD measurements of deflection.

At any position along the trial sections the reported deformation is the average of that measured in both wheel tracks of the dual wheel. The deformation of the foundation in the untrafficked area was also measured to identify any heave or subsidence of the whole foundation.

The FWD was used to carry out dynamic plate tests on the completed foundations immediately before the first load was applied, seven days after laying the sub-base (on all sub-bases except 3_{aw}, where the ruts were too great to allow seating of the plate) and 30 to 35 days after laying the sub-base. If the 95kN load was applied, then the FWD tests were repeated to check whether the foundation had been weakened, possibly by micro-cracking in the sub-base or weakening of the subgrade.

The FWD was fitted with the 300mm diameter plate. On each occasion measurements were taken over a range of applied stresses. To give a common reference for each foundation the surface modulus is reported immediately before trafficking, at seven days and after approximately one month, calculated from the deflections resulting from an applied stress of 200kPa. The surface modulus reported on completion of all trafficking, including the 95kN load, is calculated from the deflections resulting from an applied stress of 400kPa. The surface modulus of a foundation indicates the structural contribution of the whole foundation to subsequent pavement construction, and is calculated from the equation:

$$\text{Surface modulus (MPa)} = (1/d) \times 1.571qr(1-n^2) \qquad (2)$$

where d = deflection under centre of loading plate (microns)
q = surface stress (Pa)
r = radius of loaded plate (m)
n = Poisson's ratio (taken as 0.25)

6.6.1 Immediate trafficking

Three of the foundations with bound sub-bases, 3_{aw}, 8_{cw} and 13_{w}, were scheduled for trafficking immediately after laying.

The first mixture laid as sub-base was 3_{aw} and, based on the results of the laboratory CBR test, a 40kN wheel load was applied to the foundation immediately after laying. However, the foundation flexed and rutted markedly under the loading, probably partly as a consequence of the low material density achieved by compaction. Trafficking was halted after 6 passes to allow the sub-base to gain strength. A further 14 passes of the loaded wheel were applied two days later but the foundation was still flexing and rutting noticeably. The deformation after the first 20 passes was almost 30mm at the thin end of the trial section. Figure 7 shows that the surface modulus of the foundation, measured immediately before trafficking, was less than that measured on the unbound Type 1 control, ranging from 18 to 43MPa. This is probably also, in part, a consequence of low density.

Sub-base 8_{cw}, flexed under foot immediately after laying, and based on the experience gained with sub-base 3_{aw}, trafficking was delayed until the following morning. The surface modulus of the foundation, measured immediately before trafficking (12 hours after laying sub-base) is shown in Figure 7.

Figure 7 also gives the foundation surface modulus immediately before trafficking for foundation 13_{w} and the unbound Type 1 foundation. Foundation 13_{w} also flexed markedly when walked upon immediately after laying. Trafficking was attempted on both Day 1 and Day 3, but was halted after two passes on each occasion because of the flexing of the foundation. The moisture content of the material as-delivered was found to be greater than the design moisture content by only 0.4 percent. However, this brought the moisture content to 1.4 per cent above OMC, which could have significantly reduced its ability to withstand immediate trafficking. The first 50 passes were applied successfully on Day 5, after which trafficking followed the scheduled pattern.

There were no problems trafficking the unbound Type 1 foundation immediately after compaction.

Subject to adequate structural stability of the sub-base material, the FWD results indicate that a surface modulus of > 50MPa is required for trafficking to take place without causing serious damage to the foundation. The lowest value of surface modulus calculated from FWD testing on the Type 1 foundation was just above 50MPa. The results indicate that it may have been possible to traffic foundation 13_{w} slightly earlier than 5 days after laying.

6.6.2 Delayed trafficking

All the sub-bases (4_{bw}, 7_{cw}, 11 and CBM1) scheduled for trafficking to commence seven days after laying were trafficked successfully at this time. The surface modulus values of the foundations immediately before trafficking are shown in Figure 8.

Figure 8 also shows the 7 day foundation stiffness for foundations 8_{cw} and 13_{w}. It was not possible to test foundation 3_{aw} with the FWD at 7 days, due to the rutting caused by the early trafficking.

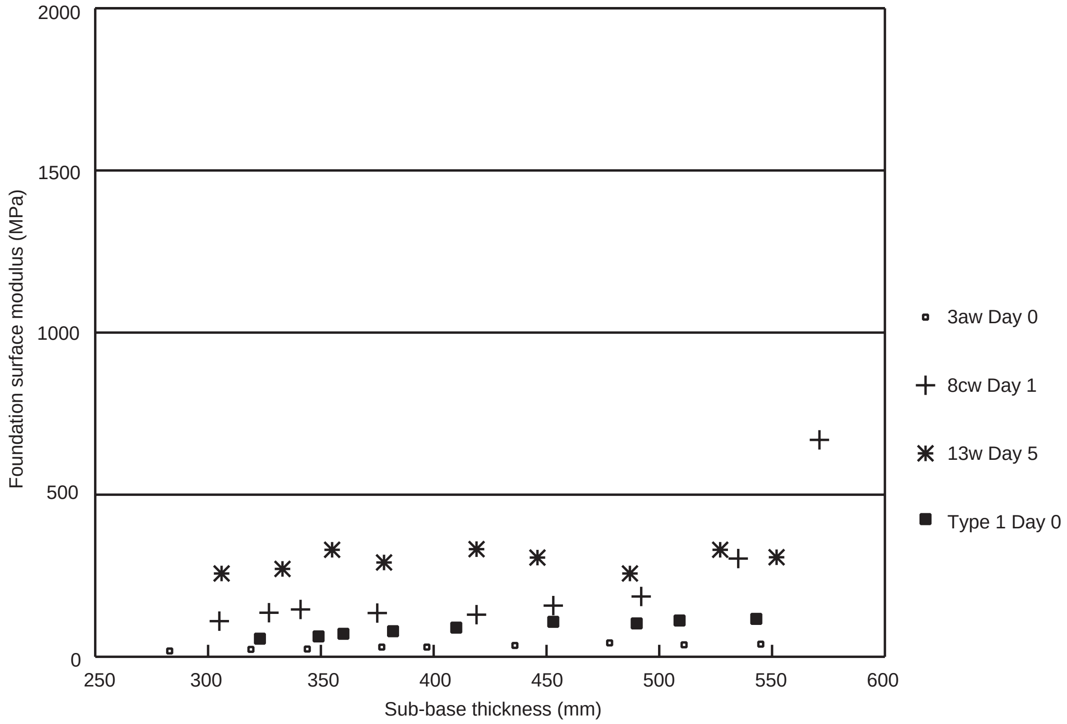

Figure 7 Foundation surface modulus before early trafficking

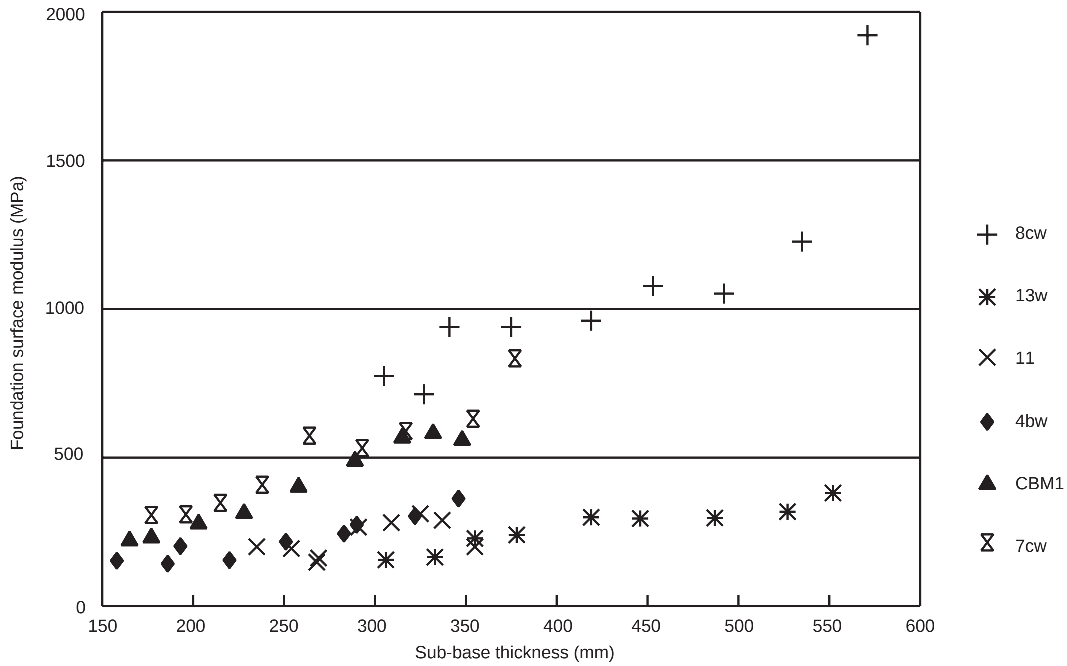

Figure 8 Foundation surface modulus at 7 days

6.6.3 Deformation and cracking

The deformation after 5,000 standard axles is shown, for each trial foundation, in Figure 9.

Foundation 3_{aw} (FA/GyL on clay):

The first 50 passes were completed seven days after laying the sub-base. The sub-base was then left uncovered and, as it dried, trafficking created clouds of dust. The foundation was subsequently damped down with water and covered between trafficking sessions to prevent further dust. Narrow transverse cracks appeared in the thin end of the sub-base after trafficking.

During the first week after laying the whole foundation heaved by approximately 4mm. There was no measurable heave after this period.

There was little further deformation, after that caused by the early trafficking and, on completion of 5,000 standard axles, the transverse cracks noted earlier in trafficking had disappeared. The profile was then reshaped to accommodate the FWD testing.

The application of 300 passes of the 95kN wheel load caused considerable further deformation at the thinner end of the trial sub-base and the ruts, of approximately 150mm, made further trafficking impossible. The additional deformation varied from 30mm at the point where the sub-base was thinnest to no more than 1.5mm, where the sub-base was thickest. This small deformation was thought to be a result of erosion by trafficking.

Foundation 8_{cw} (CC/FaL+ on clay):

There was up to 8mm deformation during the first 50 standard axles but, thereafter, there was less than 2mm further deformation during trafficking to 5,000 standard axles, largely due to erosion of the surface.

The foundation did not appear cracked after the first 5,000 standard axles and therefore the 95kN load was applied for 1,000 passes. This had no visible effect on the foundation and so there was no further trafficking.

Foundation 13_{w} (Bs/GsSs on clay):

Longitudinal cracks appeared in both wheel tracks after the first 50 passes, over the thinner half of the sub-base, and widened slightly after the next application of traffic. However, these were not apparent by the end of trafficking.

There was approximately 2mm of deformation after the first 50 passes of the wheel which increased slightly during subsequent trafficking.

After trafficking at the 95kN load, some small transverse cracks appeared where the sub-base was less than 350mm thick. These were first noted after 300 passes of the heavy load and, as they did not develop during the remaining 700 passes, no further trafficking was applied.

Unbound Type 1 foundation (T1 on clay):

There was almost 20mm deformation, where the sub-base was thinnest, after the first 50 standard axles had been

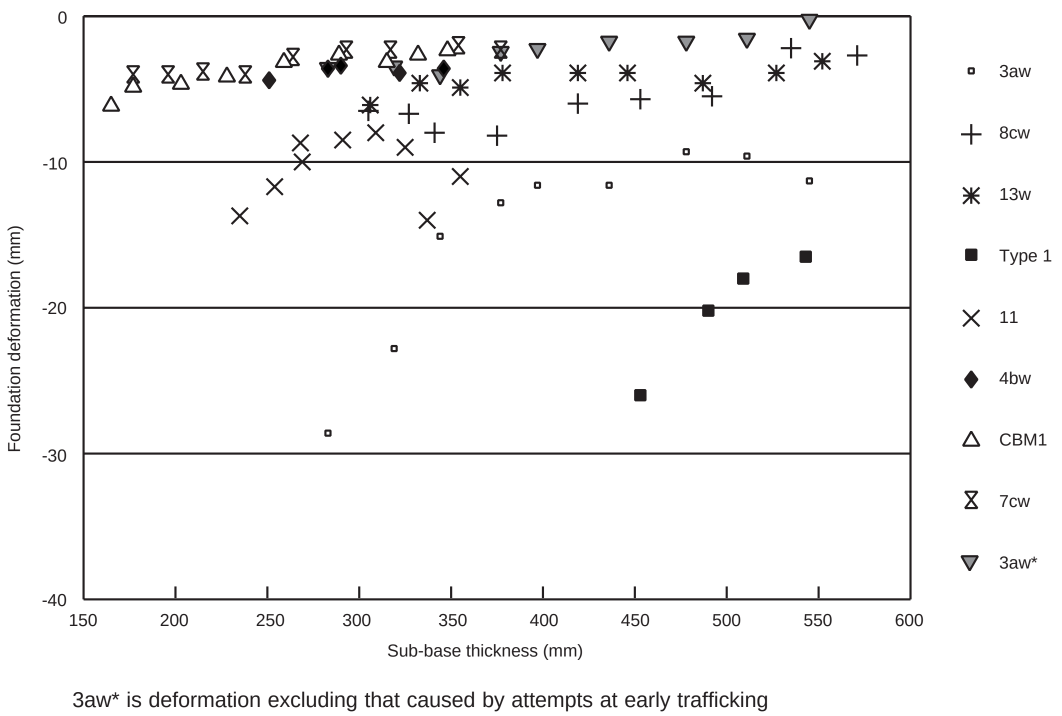

Figure 9 Deformation after 5,000 standard axles

applied. After 1,000 standard axles the deformation was greater than 40mm where the sub-base thickness was less than approximately 375mm. Figure 9 shows that the foundation with unbound Type 1 sub-base deformed more than any other foundation, for the same thickness of sub-base. From LR1132 (Powell et al, 1984), on a subgrade of CBR 3 percent, 300mm of Type 1 would have been expected to have no more than 40mm deformation after 1,000 standard axles of construction traffic. In these full scale trials the Type 1 foundation therefore deformed at a greater rate than expected.

Where there were large ruts the Type 1 was levelled and re-compacted to enable trafficking of the thicker end of the sub-base to continue. There was less than 20mm of deformation, on completion of 5,000 standard axles trafficking, where the sub-base was at least 500mm thick.

The 95kN load was not applied as significant deterioration of the foundation had already occurred.

Foundation 11 (T1/FaLGy on clay):

Longitudinal cracks appeared along the outside of the wheelpaths after each session of trafficking, but were not apparent when the next session of trafficking commenced.

Deformation appeared to increase steadily over the period in which the first 5,000 standard axles of trafficking was applied, with most of this deformation appearing during the first 1,000 standard axles. However, measurements outside the wheeltrack indicate that approximately 2.5mm of this deformation was due to the subsidence of the whole foundation, during application of the first 1,000 standard axles. This was confirmed by the measurements of rutting, which were consistently less than the deformation.

Application of the 95kN wheel load had no significant effect on the deformation and, as it did not cause cracking, there was no further trafficking.

Foundation 4_{bw} (FA/CdGs on clay):

Cracks became apparent after 500 passes were completed, along the outside edges of the wheelpaths at the thin end of the sub-base. The cracks lengthened with trafficking and the material in the wheelpaths broke into small blocks. Deformation in the wheel tracks and heave either side progressed with trafficking. At 1,200 standard axles, the thin end of sub-base punched down into the subgrade in the wheel tracks giving a deformation of over 40mm which, combined with the heave on either side, made further trafficking impossible.

The failed area of foundation, in the region where the sub-base was constructed less than 250mm thick, was excavated to a depth of approximately 200mm and replaced with CBM1 which was left for 3 days to gain cemented strength. Trafficking then continued and was completed to schedule resulting in a small increase in deformation over the remaining thicker half of the sub-base.

CBM1 foundation (CC/C on clay):

Application of 5,000 standard axles resulted in approximately 6mm of deformation. Application of the 95kN load caused some cracking at the outside edge of the wheelpath and noticeable flexing. A further 1000 standard axles were applied at the 56.4kN load resulting in an average increase of 1mm in deformation.

Foundation 7_{cw} (CC/CdC on clay):

Measured deformation in the wheel tracks was no greater than that measured outside the wheel tracks, and is not, therefore, considered to be due to trafficking.

6.6.4 Foundation surface modulus

In general foundation surface modulus increased with time, the increase being greater where the sub-base was thicker, as expected. However, where the sub-base was thin enough to be weakened by the trafficking, the increase in surface modulus between 7 and 28 days (or 0 and 28 days for 3aw) was minimal and in some cases surface modulus decreased. This is illustrated in Figure 10. Where the 95kN load was applied the surface modulus of the foundations decreased by up to 20% on all but foundation 8_{cw} and those areas of foundations 7_{cw} and 13_{w} where the sub-base was thickest.

Figures 11 to 13 show the foundation surface modulus relative to sub-base thickness after all trafficking was completed, although the amount of trafficking was not the same for each foundation. If the foundation had cracked or rutted significantly during application of 5,000 standard axles loading, the surface modulus was calculated from the results of FWD testing at this time. Otherwise the 95kN wheel load was applied and the surface modulus calculated from FWD testing after this heavy loading. As the surface modulus is generally reduced by the heavy loading it is thought that micro-cracking may have occurred even where there are no visible cracks. The stiffness/thickness relationship for trafficked foundations is the best estimate which can presently be made of the appropriate foundation stiffness to incorporate into pavement designs. For thick sub-bases laid in roads, however, naturally occurring cracks may reduce foundation stiffness.

For each foundation there is an approximately linear relationship between the surface modulus and the sub-base thickness. The results, extrapolated to the same thickness of sub-base, where necessary, show that all the bound foundations have greater surface modulus values on completion of trafficking than the unbound granular, Type 1 foundation. Foundations 7_{cw} and 8_{cw} have modulus values very similar to the CBM1 control foundation, even though foundation 8_{cw} had first been trafficked at less than 24 hours after laying the sub-base.

Repeat measurements of surface modulus were made between four and five months after construction. The surface modulus had increased on all bound foundations. Table 11 shows the ratio of the surface modulus of each foundation, four to five months after laying the sub-base, compared to the surface modulus obtained between 30 and 35 days after completing construction. For comparison, the ratio between IT stiffness values at 360 days and values at 28 days, taken from the Phase 1 results on the sub-base mixtures, is also given.

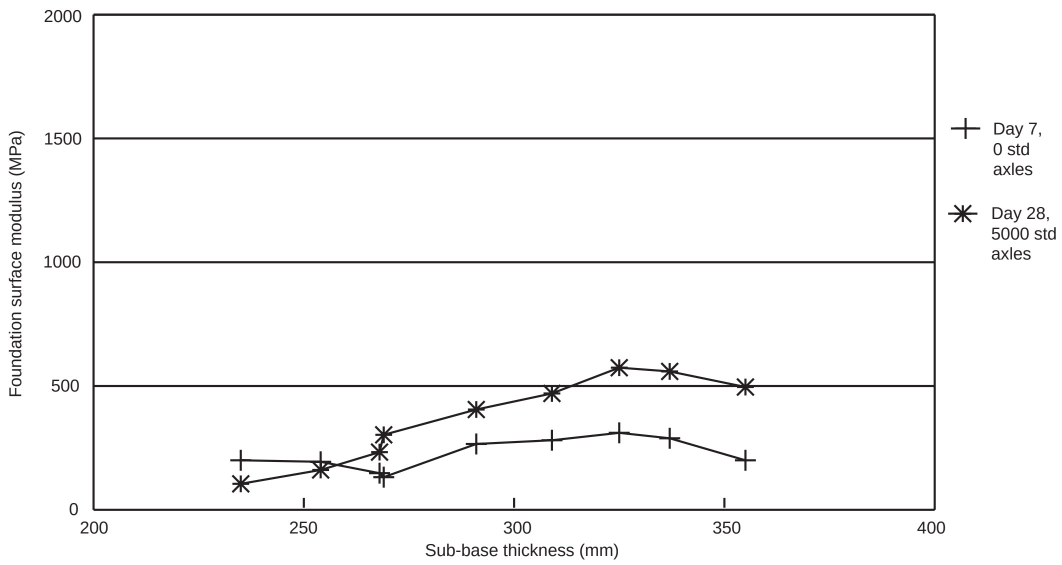

Figure 10 Illustration of surface modulus decreasing with time and traffic where the sub-base is thinnest (Foundation 11)

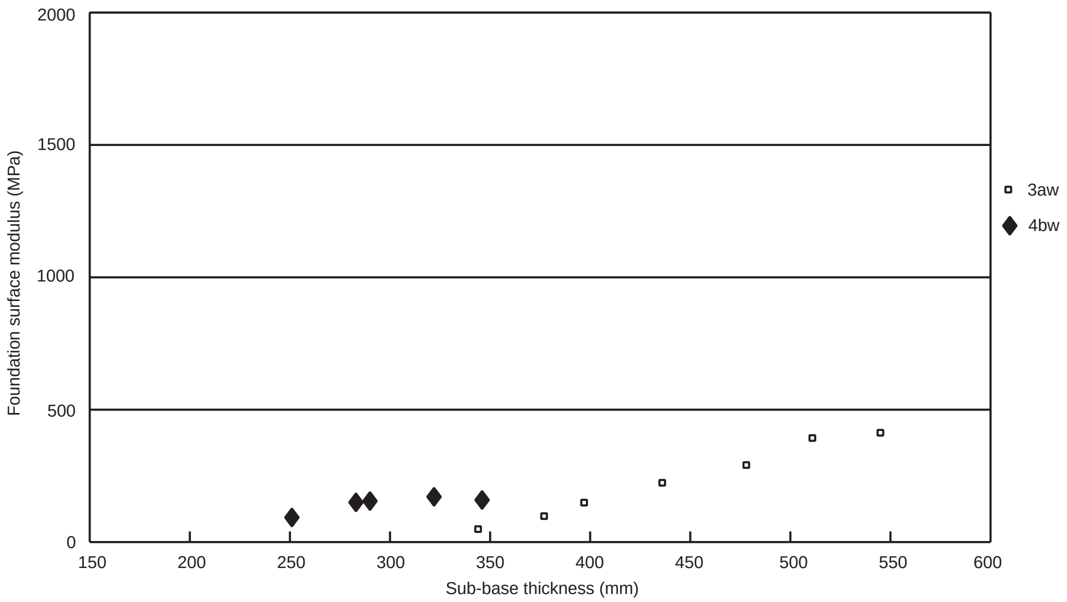

Figure 11 Pulverised fuel ash family, foundation surface modulus at 30 to 35 days, after completion of trafficking

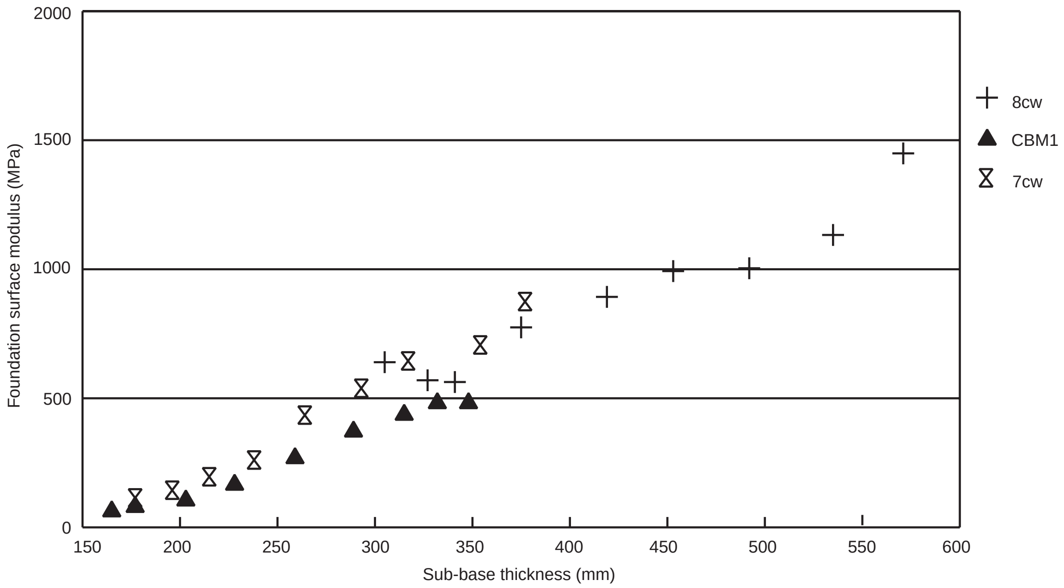

Figure 12 China clay sand family, foundation surface modulus at 30 to 35 days, after completion of trafficking

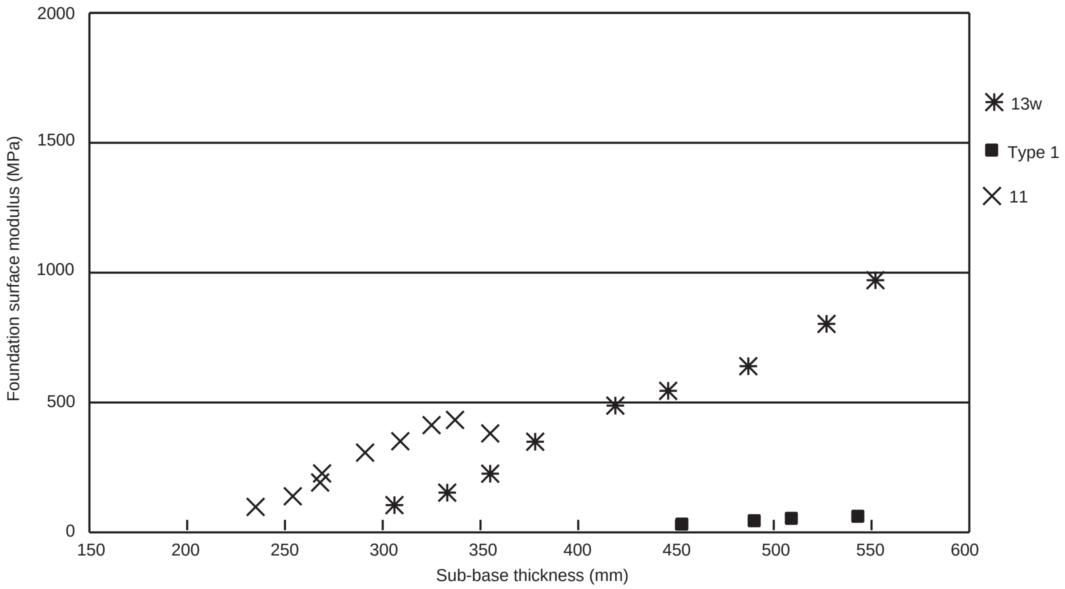

Figure 13 Coarse aggregate family, foundation surface modulus at 30 to 35 days, after completion of trafficking

Table 11 Ratio of surface modulus values and of mixture stiffness values

Material	3_{aw}	4_{bw}	7_{cw}	8_{cw}	*11*	13_{w}	*CBM1*
Composition	*FA/ GyL*	*FA/ CdGs*	*CC/ CdC*	*CC/ FaL+*	*T1/ FaLGy*	*Bs/ GsSs*	*CC/ C*
Foundation surface modulus at 3 to 4 months: foundation surface modulus at 30 to 35 days	1.4	1.7	1.5	1.4	3.2	2.0	-
IT mixture stiffness from Phase 1, 360 day : 28 day	1.4	1.8	1.5	1.4	1.7	2.3	1.6

Bulk materials: FA = pfa, CC = china clay sand, T1 = granite Type 1, BS = air-cooled blastfurnace slag

Binders/activators: C = cement, L = quicklime, Fa = pfa, Gs = granulated blastfurnace slag, Cd = cement kiln dust, Gy = FGD gypsum, Ss = steel slag fines, + = sodium carbonate

It can be seen that the ratios between 360 day and 28 day mixture stiffness, obtained from laboratory testing of specimens, are comparable to the ratios of the foundation surface modulus, obtained by FWD measurements at approximately one month after laying sub-base and at 3 to 4 months after laying. The exception is foundation 11, which improved more than would have been expected. The results indicate that, for stiff sub-bases, the surface modulus of the foundation is very much influenced by the sub-base stiffness modulus.

6.7 Comparison of foundations

Foundations have been compared based upon their early life performance. As the stiffness of the hydraulically bound materials continues to increase, the foundation surface modulus will also increase and the ranking of foundations may alter.

As there was some overlap between the wedge dimensions for each sub-base, it was possible to compare surface modulus of foundations having the same thickness of sub-base. Table 12 presents the foundation surface modulus on completion of trafficking, at between 30 and 35 days after laying, where the thickness of sub-base for each foundation is 325mm.

Table 12 Surface modulus of foundations with 325mm of sub-base, at 30 to 35 days, after completion of trafficking

Sub-base material	*Foundation surface modulus (MPa)*
7_{cw}, CC/CdC	650
8_{cw}, CC/FaLGy	570
CBM1, CC/C	470
11, T1/FaLGy	410
4_{bw}, FA/CdGs	165
13_{w}, Bs/GsSs	130
3_{aw}, FA/GyL	<50
Unbound Type 1	<30

The foundations were also compared in terms of the thickness of sub-base required for a foundation stiffness, on completion of trafficking, of at least 100MPa, and the thickness of sub-base required for deformation of less than 40mm under a loading of 5,000 standard axles. The ranking of the sub-base materials is approximately the same for both these parameters, as can be seen from Table 13. The values for foundation 8_{cw} have been obtained by extrapolation.

Table 13 Sub-base thickness comparison

Sub-base material	*Sub-base thickness (mm) for foundation stiffness at 30 to 35 days, after completion of trafficking, >100MPa*	*Sub-base thickness (mm) for < 40mm deformation after 5,000 standard axles of PTF trafficking*
8_{cw}	150	150
7_{cw}	170	<150
CBM1	190	<150
11	240	<250
4_{bw}	270	250
13_{w}	300	<300
3_{aw}	380	370
Unbound Type 1	>550	450

While the deformation of foundation 3_{aw} would probably have been considerably less had it been left to gain cemented strength before the initial trafficking, the surface modulus on completion of all trafficking is consistently less than that of the other foundations with the same thickness of bound sub-base. This is probably partly due to the low density achieved.

6.8 Summary of full-scale trials

The full scale trials, during which trial sub-bases were laid on a subgrade of CBR 3 per cent, are summarised below.

6.8.1 Production

Mixtures in the fine aggregate family were produced in a continuous flow forced action mixer. Medium and coarse aggregate mixtures were produced in batches in forced action or paddle mixers. Sub-bases were laid in pre-prepared bays and compacted with a vibrating plate.

The trials illustrated some of the difficulties in choosing the optimum initial moisture content, taking into account both compaction and hydration of binder. Often mixtures were made drier than the target value, which was wet of optimum. The target value was chosen to make a compromise between optimum compaction condition and the optimum strength (after curing) condition. Even so, some sub-base mixtures were rather wet and trafficking required some moisture to be lost or taken up by curing before trafficking could fully commence.

6.8.2 Immediate trafficking

The results indicate that, for a sub-base comprising secondary aggregates and binders to be trafficked successfully on a subgrade of CBR 3 per cent, the foundation stiffness should be at least 50MPa before trafficking commences (measured with an applied stress of 200kPa on a 300mm diameter plate).

Although immediate trafficking did not prove possible in the trials it was quite feasible to traffic sub-base 8_{cw} less than 24 hours after laying. Sub-base 13_{w} was also successfully trafficked 5 days after laying, two days earlier than those bound materials left for the conventional 7 days. Using large plant, for production of the relatively small amounts of material laid at the PTF, may have contributed to the difficulties of immediate trafficking. Considerably more material would usually be produced by a continuous flow, forced action mixer, in order to calibrate the machine to produce a mixture with precisely the proportions and moisture content required. Too little water in mixture 3_{aw} may have contributed to the low density achieved. In contrast, mixture 13_{w} was rather wet which contributed to the difficulties in immediate trafficking. The batch mixed material, mixture 8_{cw}, was the most successful in terms of immediate trafficking. The design moisture content of this mixture was reduced, during mixing, to approximately OMC, and the proportions of each constituent were closely controlled.

6.8.3 Temperature during the trials

Throughout the full scale trials the weather was very warm with the minimum air temperature recorded being 9°C. Sub-bases 3_{aw}, 8_{cw}, 13_{w} and 4_{bw} saw an average 24 hour temperature of approximately 25°C during the first 7 days, while the temperature was approximately 16°C for the first 7 days of sub-bases 11, and 7_{cw} and the CBM1 control. It is therefore unlikely that similar sub-bases on site would cure much faster than they did at the PTF.

6.8.4 Performance of coarse aggregate family

The sub-bases with coarse aggregate all performed satisfactorily under the loading conditions at the PTF. Although foundation 13_{w} could not be trafficked immediately such trafficking has proved possible with similar mixtures used in full scale construction applications. Both sub-bases 11 and 13_{w} needed to be thicker than the materials with china clay sand as aggregate, for the same stiffness performance at 30 to 35 days. However, Table 11 shows that the surface modulus of foundations 11 and 13_{w} increased more, over a period of 3 to 4 months, than that of the other foundations.

6.8.5 Performance of medium aggregate family

The foundations comprising sub-bases based on china clay sand, 7_{cw}, 8_{cw} and the CBM1 control, performed particularly well during the trials. The performance appeared to be unaffected by the different binders or by the early trafficking of sub-base 8_{cw}.

6.8.6 Performance of fine aggregate family

The performance of the trial foundations comprising sub-bases based on pulverised fuel ash was less predictable. Early trafficking of foundation 3_{aw} was not possible and when trafficking was carried out, seven days after laying the sub-base, large amounts of dust were raised. However, little further rutting occurred during trafficking at the 56.4kN wheel axle load, although the foundation soon deteriorated under the heavy 95kN loading. While sub-base 4_{bw} performed well where it was thickest, it failed catastrophically where it was thinnest. The sub-base cracked either side of the wheelpath leading to punch through and shearing of the clay, indicating that there was little load spreading across the cracks. These fine aggregate foundations seemed most sensitive to the exact sub-base thickness and loading applied.

6.8.7 Development of surface modulus

During trafficking most foundations increased in stiffness where the sub-base was thicker, and decreased in stiffness where the sub-base was thinner.

7 Assessment of benefits

Road construction currently accounts for approximately one quarter of the UK’s consumption of primary aggregates. Therefore, the use of secondary aggregates and binders in pavement foundations has large potential benefits. The determination of material performance data and a generic specification provides the framework for successful use of these materials in road construction. An increase in the use of secondary aggregates and binders will help decrease the demand for primary mineral reserves. This will help meet government recycling targets and will also be environmentally beneficial. An additional benefit is the utilisation of material which would otherwise be stockpiled. The growing political emphasis on maintaining a sustainable environment means that the use of secondary materials is likely to become more economically attractive.

The use of hydraulically bound mixtures of secondary materials would usually also produce stiffer and stronger foundations than those produced with conventional unbound Type 1 sub-base. This should result in the possibility of thinning the upper pavement layers and/or increasing the life of the pavements.

7.1 Pavement design

The trial foundations constructed using secondary aggregates and binders as sub-base are considerably stronger and stiffer than those built using conventional unbound Type 1 granular material. However, pavement designs incorporating these sub-bases need to be based on the material properties after any cracking, due to shrinkage, thermal effects or trafficking, has occurred. Shrinkage and thermal cracking did not occur on the trial foundations, probably due to the relatively short lengths. However, some foundations were cracked by trafficking.

At this stage it is, therefore, not possible to be precise in assigning a sub-base stiffness modulus to each trial material for use in pavement design.

The following pavement design example has been based on a sub-base stiffness modulus which the results of this trial, previous work and examination of the French design methods, indicates to be reasonable for trafficked material. A sub-base stiffness modulus of 2,000MPa is thus assumed for pavement design; stiffness values of the fine aggregate family are likely to be lower than this, whereas those of the coarse aggregate family are likely to be higher.

It should be noted that the sub-base stiffness values in Figure 14 are obtained from laboratory tests on specimens, which are necessarily intact. For pavement design the lowest values along a site must be used, and these will be where material is cracked. Chaddock and Atkinson (1996) show that the surface modulus of a foundation, where the sub-base is cracked, can be reduced to 20 per cent of the value calculated from FWD measurements over uncracked sub-base. The actual reduction in stiffness will vary according to the sub-base material.

As there is no model available which has been calibrated for pavements with hydraulically bound sub-bases, pavement modelling was carried out using the TRL General Stress program, which models the pavement as a linear-elastic layered structure. Layers are assumed to be homogenous and isotropic. The results of applying this model to materials known to be cracked must, therefore, be treated with caution as the model has not been calibrated for this type of pavement.

A pavement incorporating a sub-base of stiffness modulus 2,000MPa, as above, was modelled using the General Stress program. The layer thicknesses were found which would give a pavement of equivalent performance to a conventional design, as specified in Volume 7 (DRMB 7.2.3). For a pavement with a design life of 100msa, built on a subgrade of strength equivalent to that on which the trial foundations were built, CBR 3 per cent, alternative designs are:

Standard design:	50mm HRA wearing course 360mm DBM roadbase 300mm Type 1 sub-base.
Alternative design:	50mm HRA wearing course 150mm DBM roadbase 350mm hydraulically bound sub-base (Stiffness modulus = 2,000MPa)

The potential savings of bituminous material are significant, although these savings would reduce as the strength of the subgrade increased.

The economic benefits of sub-bases constructed utilising secondary aggregates and binders are difficult to assess as the costs of the materials are very variable. Costs would also change as the market developed. However, a cost comparison was carried out, based on the assumptions that the as-laid price per cubic metre of material is £50 for DBM, £15 for Type 1 and £25 for hydraulically bound sub-base utilising secondary materials. If the above designs were used over a 1km length of 2-lane dual carriageway, then the cost using standard materials is approximately £340,000, compared to a cost of £270,000 using the design incorporating secondary materials. A significant saving of £70,000 is implied. It is, therefore, likely that at least a good proportion of the secondary materials could be used economically.

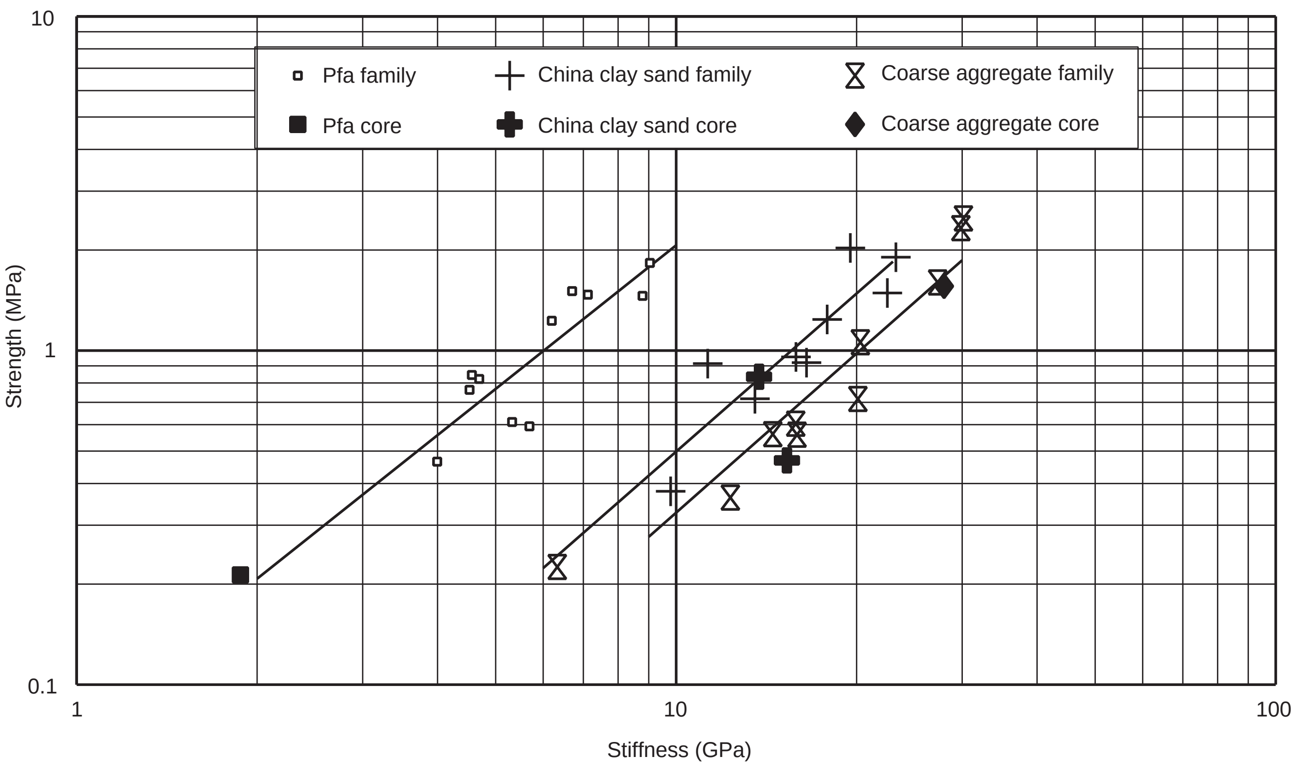

Figure 14 Indirect tensile strength/stiffness for 90 day data

8 Generic specification

In order to investigate the possibility of a generic specification for sub-bases incorporating secondary aggregates and binders, all varieties of each family tested during the project were plotted according to their strength and stiffness at 90 days. It can be seen from Figure 14 that there is an approximately linear relationship, on a log/log scale, between strength and stiffness for the members of each aggregate family.

For the same strength, the fine aggregate family has lowest stiffness and the china clay sand (medium aggregate) family has a slightly lower stiffness than the coarse aggregate family. It is possible that the comparatively low values of stiffness and strength recorded on cored specimens of the pulverised fuel ash family are due to the coring process damaging the material and to the low density of the compacted material.

To enable the use of different mixtures incorporating secondary aggregates and binders, the procedure outlined in Figure 15 is proposed. The procedure may be refined as more experience of using these materials is gained.

The mixture constituents are chosen, based on the location of the works, availability of materials, costs and past experience. At least three variants of the chosen mixture are initially tested. Depending on the proportions of constituents it may not be necessary to perform the OMC test on all variants.

Preliminary tests are carried out to select the most suitable variant of the mixture. These tests would ideally be IT strength and stiffness tests, carried out at 28 and 90 days. A simple indirect tensile strength test may be sufficient at this stage. Testing can be accelerated by curing specimens at a higher temperature.

The likely time interval between mixing and compacting is estimated by considering the location of the road works relative to the mixing plant. If necessary, indirect tensile tests are carried out to assess the effect of an interval longer than two hours on the structural properties of the sub-base mixture. Modification of the mixture is recommended to avoid losses in strength and stiffness of over 10 per cent.

Expansion on curing is assessed by monitoring the dimensions of test specimens.

Environmental factors are also assessed. If the road construction is to take place at low temperatures, and the sub-base design assumes a curing period before trafficking is permitted, then the effect on the structural properties of the proposed materials is assessed by curing specimens at the anticipated temperature. The mixture proportions are amended to ensure an adequate in-situ strength and stiffness at the time construction traffic is permitted access to the site. Alternatively, the curing time before trafficking is extended. If these measures do not result in adequate structural properties, then the use of the mixture is limited to warmer seasons of the year.

Frost and water durability are assessed by conducting frost heave and soaked strength tests on compacted specimens. If these tests are adopted, then the curing time should be selected to permit an adequate cemented strength at the time of immersion and testing. This time should reflect the period during which on-site protection against cold and wet can be provided. As experience is gained in the use of these materials better judgements on the need for durability tests can be made.

The structural properties of the resulting mixture are determined and plotted on a strength and stiffness graph, which is similar to that shown in Figure 14, and the mixture associated with a previously derived family. The in-situ performance of the proposed mixture can then be predicted from the known performance of a family member and a foundation design proposed. In this manner, the materials engineer and road designer can have greater certainty of the behaviour of the material prior to carrying out a trial to assess the adequacy of plant to mix, lay and compact the material. The trial would also permit the assessment of the structural performance of the material during the construction of the road.

9 Discussion

The full scale structural assessment of foundations comprising sub-bases on a subgrade of CBR 3 per cent, indicated that sub-bases incorporating secondary aggregates and binders could be constructed thinner than unbound granular, Type 1 sub-base and yet have a better long term performance. Theoretical analysis suggests that significant reductions in the overlying asphalt material should be possible. As subgrade strength increases, the difference between the thicknesses of the hydraulically bound sub-bases and the unbound granular, Type 1 sub-base would reduce, eventually to nothing, because of the practical need to specify a minimum sub-base thickness. These hydraulically bound sub-bases, however, would result in stiffer foundations that offer greater support to the overlying pavement layers and, hence, reduced asphaltic thickness or an increase in the pavement life.

There were some variations between the target structural properties of mixtures and the properties of the mixtures used in the trials. It was also found, for some materials, that the moisture content assumed to be necessary for good hydration of binder tended to be too high for stability of the material during compaction and immediate trafficking. The trials have indicated that mixtures should be compacted at OMC, when vibratory equipment is used, and the thickness design should be based upon the structural properties at this moisture content.

Although the trial sub-bases performed very well, the trials in the PTF were under cover and, as a consequence, the effect of wet weather on the performance of the sub-bases incorporating secondary aggregates and binders has not been assessed. Also, no examination was made of the effects of frost on the foundation materials used in this research. Foundation materials are particularly vulnerable to frost damage when the working platform is directly exposed over the winter. If foundation materials are likely to experience freezing temperatures and sub-surface water is expected to be available, then these materials should be assessed for their susceptibility to frost and only used if

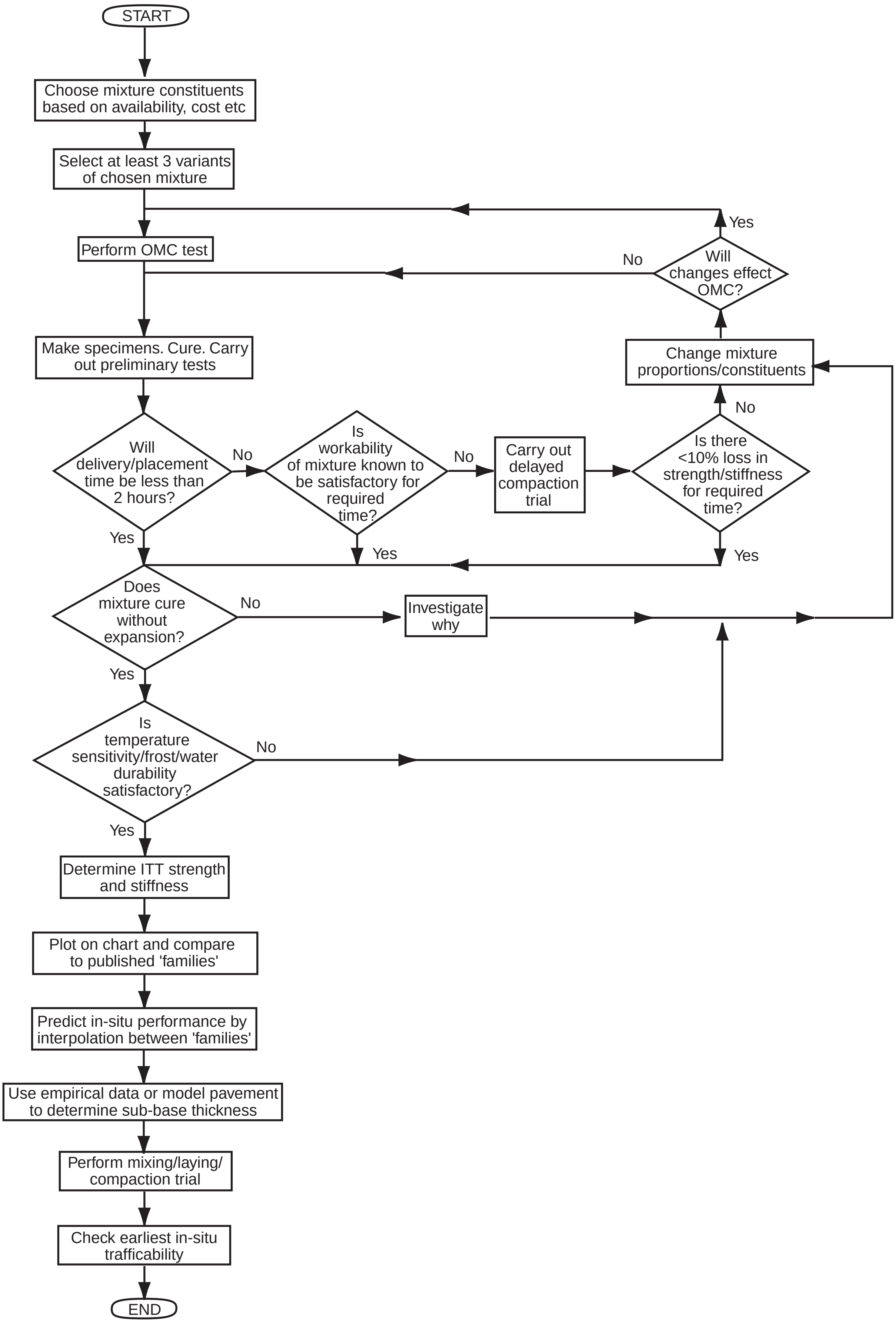

Figure 15 Strategy for material investigation

proven to be suitable. Testing for frost and water durability should result in selecting mixtures with satisfactory properties.

Another limitation of the PTF trials was that the trial sub-bases were only 10m long. It is uncertain what the effects of natural cracks, which would form in longer stretches of sub-base, would be on the foundation stiffness. Also the measured foundation stiffness is based on only one subgrade strength. Investigation of the performance of these sub-bases built on subgrades of other strengths is recommended to provide more performance information on which to base foundation and pavement designs.

During the trafficking period most foundations increased in stiffness where the sub-base was thicker, and decreased in stiffness where the sub-base was thinner. Sub-bases designed thicker than the critical value, which increase in stiffness with time when trafficked, would provide increased value in terms of a greater potential for reducing roadbase thickness or an increased service life of the overall road pavement construction.

10 Conclusions

A variety of hydraulically bound secondary materials, for potential use as sub-base, were assessed in the laboratory to investigate their structural properties of strength and stiffness. Mixtures were then laid as sub-base, directly on clay, to form trial foundations in TRL's Pavement Test Facility. These foundations were tested under conditions representing construction traffic of at least 1000 standard axles. Performance of the trial foundations was assessed by measuring deformation and by calculating the surface modulus from the results of FWD measurements. The performance of the foundations constructed utilising hydraulically bound secondary aggregates and binders was compared to that of two control foundations, incorporating, as sub-base, conventional unbound Type 1 granular material or CBM1. On the basis of the experimental work described, the following conclusions can be drawn:

1. Laboratory testing has proved that it is possible to design mixtures including a wide range of secondary aggregates and binders that are suitable for use as sub-base. Tests should include OMC, IT strength and stiffness, durability and, if necessary, frost heave.
2. The results of the IT strength and stiffness tests indicate that the relationship between the strength and stiffness of a mixture varies according to the size of the main aggregate used in the mixture; fine, medium or coarse
3. Adding more binder to mixtures generally increases their strength and stiffness. The mixtures with fly ash and slag as aggregate were least affected by changing the proportions of the mix constituents.
4. Durability tests indicated that gypsum is not a suitable material for use as the fine aggregate, constituting the major proportion of mixture components.
5. Some mixtures are significantly affected by low temperature curing, even after 90 days. In general binders based on pfa and quicklime cure more slowly at low temperature, while cement kiln dust with cement is little affected by low temperature curing.
6. The trials showed that some foundations constructed with sub-bases comprising secondary aggregates and binders can be successfully trafficked less than 24 hours after laying sub-base material. Control of moisture and compaction density have been identified as being significant factors in determining the immediate trafficability of a material. The optimum moisture content for compaction should be used and the design based on the properties of the mixture at this moisture content. The FWD tests indicate that a surface modulus of at least 50MPa should be attained before trafficking commences (measured with an applied stress of 200kPa on a 300mm diameter plate). A measure of the internal deformability of the sub-base is also required to ensure that the sub-base will not rut excessively.
7. The trial foundations produced using sub-bases comprising secondary aggregates and binders all performed better than the control foundation incorporating unbound, granular Type 1 sub-base and some performed better than the control foundation constructed with a CBM1 sub-base. This improved performance results in the opportunity to thin the upper pavement layers and/or increase the life of the pavement.
8. Two road pavements were theoretically modelled, one a conventional design and one using hydraulically bound material as sub-base. The hydraulically bound sub-base was assumed to have a stiffness of 2000MPa, which was considered a reasonable figure for in-situ cracked material. The model indicates that this would result in a significant reduction in the amount of bituminous roadbase required, with associated cost saving.
9. The environmental benefits of using secondary aggregates and binders in road foundations are a reduction in the requirement for primary aggregate and less disposal of secondary material to landfill.
10. A common material assessment procedure has been developed, suitable for all alternative and conventional hydraulically bound sub-base materials. To assist in classifying materials assessed by this procedure, a simple 'family' grouping is proposed. As experience in the use of these materials is gained, appropriate thickness designs for sub-bases incorporating novel materials can be estimated by linking them to the known performance of other 'family' members.

11 Recommendations

On the basis of the satisfactory performance of sub-bases incorporating secondary aggregates and binders, in the Pavement Test Facility, and the experience with similar materials in other countries, it is recommended that specification trials be carried out using these materials in full-scale road schemes.

Further laboratory testing and small scale trials are recommended to develop the mix design of materials comprising secondary aggregates and binders. This work would confirm the adoption of OMC for compaction and include verification of the proposed criteria for immediate trafficking. For materials designed to have gained strength and stiffness by curing before trafficking, laboratory investigation of low temperature curing is recommended to ensure that a mixture will have the required structural properties when laid in the field at different times of the year.

Trials on the road network will develop specifications for foundations incorporating secondary aggregates and binders and provide more information on which to base foundation designs, taking into account the effect of site variations in subgrade and climatic conditions. Site surface modulus measurements will be critical to this process. It is also proposed that these trials be used to assess the amounts by which roadbase layers might be thinned when they are laid on stiffer foundations than those produced by the unbound granular material designs given in HD 25 (DMRB 7.2.2.). Alternatively, the extent by which pavement life is increased for roadbases of unchanged thickness should be assessed. On the successful conclusion of these trials, materials comprising secondary aggregates and binders will be recommended for inclusion in DETR standards.

It is suggested that the guidelines in Appendix B are followed when carrying out specification trials. These are tentative guidelines only which will be amended as more information becomes available.

12 Acknowledgements

The work described in this report was carried out in the University of Nottingham and the Civil Engineering Resource Centre (Resource Centre Manager: Mr P G Jordan) of TRL. The interest and valuable contribution of representatives of the British Cement Association, British Steel, Buxton Lime Industries, Bardon Aggregates/ECC International, National Power, PowerGen, Tarmac HBM (UK), Highways Agency, the Department of Environment, Transport and Regions and also of Mr J Kennedy is gratefully acknowledged. Gratitude is also due to Brunner Mond for the free provision of sodium carbonate and to Wrekin Construction Ltd and RMC plc for their help in mixing the materials.

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Appendix A

Table A1 Sources of component materials

Component material	*Source*
Blastfurnace slag	Cambrian Stone, Llanwern Works, Newport, South Wales.
Cement	Blue Circle Industries, Hope Works, Sheffield, South Yorkshire.
Cement for CBM 1	Ready Mix Concrete (Transite), Eversley, Hampshire.
Cement kiln dust	Blue Circle Industries, Westbury, Wiltshire.
China clay sand	English China Clay, Lee Moor Quarry, Plymouth, Devon.
Pulverised fuel ash	PowerGen Plc., Ratcliffe-on-Soar, Nottinghamshire.
Granulated slag	Cambrian Stone, Llanwern Works, Newport, South Wales.
Gypsum	PowerGen Plc., Ratcliffe-on-Soar, Nottinghamshire.
Lime	Buxton Lime Industries, Tunstead Quarry, Wormhill, Derbyshire.
Sodium carbonate	Brunner Mond (UK), Loftock Works, Nr. Northwich, Cheshire.
Steel slag	Cambrian Stone, Llanwern Works, Newport, South Wales.
Type 1 sub-base	Redland Aggregates, Mountsorrel Quarry, Leicestershire.

Appendix B: Guidelines for specification trials of foundations incorporating secondary aggregates and binders

Although the use of mixtures incorporating secondary aggregates and binders is common in some European countries, until more experience is gained on their use in UK pavement designs, it is recommended that a trial area be constructed prior to the main works, in order to verify the thickness design and assess the earliest in-situ traffickability.

It is suggested that the following procedure be followed to design a mixture incorporating secondary aggregates and binders:

1 Choose the mixture constituents according to availability, cost and past experience.

2 Make at least 3 mixtures of varying proportions.

3 Calculate the Optimum Moisture Content (Section 3.1).

4 Measure the Indirect Tensile Strength and the Indirect Tensile Stiffness, at 28 and 90 days after mixing (Section 3.3).

5 Assess the workability time, if the time between mixing and laying material is likely to be more than two hours (Section 4.4). The workability time, or the interval which can be allowed between mixing and compaction without significant loss of structural properties, can be determined by carrying out IT strength and stiffness tests at various times after mixing. As an approximation it could be assumed that the workability time is two hours for mixtures containing cement and four hours, for mixtures without cement.

6 Check that there is no expansion on curing by measuring specimen dimensions.

7 Check the temperature sensitivity and frost/water durability.

Until formal thickness designs are developed it is suggested that the procedure below is followed for pavement design:

8 Determine ITT strength and stiffness of selected mixture variant, and plot on strength/stiffness chart (Section 8). Estimate performance in-situ by interpolation between families/known mixtures.

9 Either: a Use empirical thickness design data, or

b Carry out initial design.

For design purposes, until more information is available, it is suggested that conservative estimates of material elastic modulus or stiffness be used. For in-situ material, based on the performance of foundations trafficked in the PTF trials, suggested values are:

Fine aggregate family:	500MPa
Medium aggregate family:	2000MPa
Coarse aggregate family:	3000MPa

The above are values of sub-base stiffness, not foundation surface modulus. It must be emphasised that these values are suggestions only, based on the PTF trials at TRL. The actual values used should be based on the lower material stiffness values calculated from site measurements using the FWD. These are likely to be obtained where hydraulically bound sub-base material is cracked, which will happen due to thermal expansion etc.

Where sub-base mixtures have good aggregate interlock it may be reasonable to construct a sub-base layer as thin as 150mm. Where there is little aggregate interlock a thicker layer is suggested, say a minimum of 300mm.

10 It is suggested that the performance of foundations incorporating sub-bases comprising secondary aggregates and binders, especially those with little or no aggregate interlock or cohesion, would be improved significantly by stabilisation of low CBR subgrade prior to placing sub-base material. This procedure would also aid the construction process.

11 Mixing should be carried out using forced action mixers.

12 Compaction, to achieve at least 95% MDD, should be carried out by :

Vibratory roller for the coarse aggregate family.
Vibratory or pneumatic tyred roller for the medium aggregate family.
Pneumatic tyred roller for the fine aggregate family.

13 It is suggested that a foundation surface modulus value of at least 50MPa (measured at a stress of 200kPa on a 300mm diameter plate), and an in-situ CBR value of at least 25 percent and preferably 40 per cent, be achieved before trafficking commences.

By careful monitoring of road trials, information will be built up to enable more accurate design and to establish, with more certainty, the benefits to be gained by using sub-bases including secondary aggregates and binders.

Abstract

As part of the LINK Transport Infrastructure and Operations Programme, research was carried out to enable the use of secondary aggregates and binders in pavement foundations. This report describes laboratory testing of mixtures including secondary aggregates and binders, and a full-scale trial of six sub-bases constructed with these mixtures, carried out in TRL's Pavement Test Facility (PTF). The secondary materials investigated were confined to those of immediate interest to the industrial sponsors and were china clay sand, blastfurnace slags, basic oxygen steel (BOS) slags, pulverised fuel ash, gypsum and cement kiln dust.

Initially thirteen mixtures of secondary materials, sometimes combined with conventional aggregates and binders, were selected for testing in the laboratory. The mixtures divided into three families, according to whether the major component was fine, medium or coarse aggregate. The fine aggregate was either pulverised fuel ash or gypsum, the medium was china clay sand and the coarse was either crushed granite or air-cooled blastfurnace slag.

Each mixture was tested for compactability, compressive strength, indirect tensile strength and stiffness and durability. Six of the mixtures also underwent further testing, to examine the effect, on the structural properties, of small changes in both moisture content and the proportions of constituents. The effect of curing temperature and the workability time were also investigated. The components of these mixtures were:

- Pulverised fuel ash, gypsum and quicklime.
- Pulverised fuel ash, cement kiln dust and granulated blastfurnace slag.
- China clay sand, cement kiln dust and ordinary Portland cement.
- China clay sand, pulverised fuel ash, quicklime and sodium carbonate.
- Crushed granite, pulverised fuel ash, quicklime and gypsum.
- Air cooled blastfurnace slag, granulated blastfurnace slag and basic oxygen steel (BOS) slag.

The mixtures were then laid as sub-base in full scale trials together with control sub-bases of unbound granular, Type 1 and CBM1 material. Each sub-base was laid as a wedge, on pre-shaped subgrade, the thickness varying along the direction of trafficking. The performance of the resulting foundations, under wheel loads representative of construction traffic, was assessed by measuring deformation in the wheelpath, ruts and the surface modulus of the foundation obtained from Falling Weight Deflectometer (FWD) testing.

The results of the laboratory and full-scale trials are presented and demonstrate the potential for the use of secondary aggregates and binders in road pavement foundations. Based on these results, a procedure for the mix and thickness design of sub-bases comprising secondary materials is proposed. The benefits of constructing foundations incorporating secondary materials are discussed.

Related publications

TRL248 *Stabilised sub-bases in road foundations: Structural assessment and benefits* by B C J Chaddock and V M Atkinson. 1997 (price £25, code E)

SR831 *Errors in the sampling and testing of sub-base aggregates* by P T Sherwood and D C Pike. 1984 (price £20)

LR1132 *The structural design of bituminous roads* by W D Powell, J F Potter, H C Mayhew and M E Nunn. 1984 (price £20)

LR901 *Strength of clay subgrades: Its measurement by penetrometer* by W P M Black. 1979 (price £20)

CT89.1 Aggregates in road construction update (1996-1998). *Current Topics in Transport: selected abstracts from TRL Library's database* (price £20)

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