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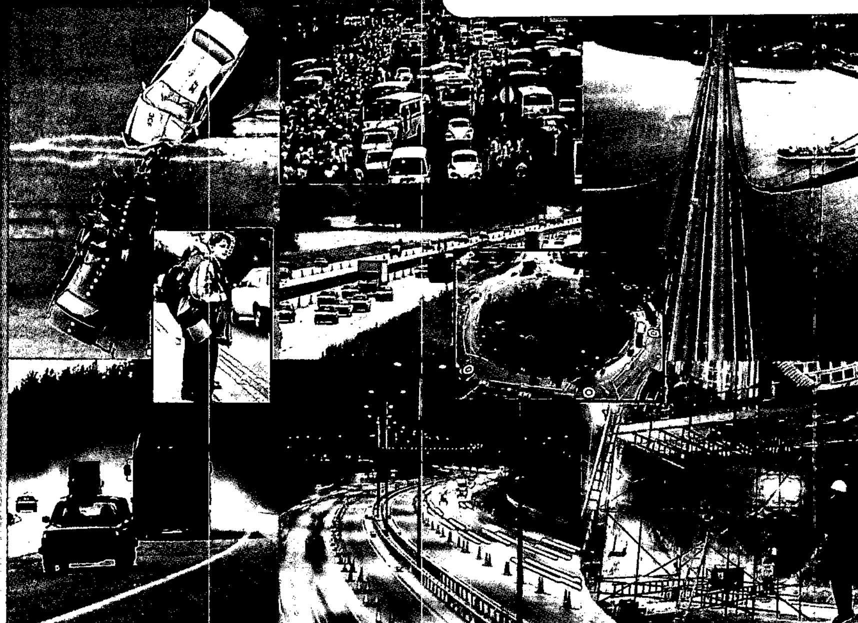

Trench reinstatement trial at TRL's Pavement Test Facility

by M H Burtwell and D I Blackman

TRL Report 197

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TRANSPORT RESEARCH LABORATORY

TRL REPORT 197

TRENCH REINSTATEMENT TRIAL AT TRL'S PAVEMENT TEST FACILITY

by M H Burtwell and D I Blackman

This report describes work commissioned by the Road Engineering and Environmental Division of the Highways Agency and the National Joint Utilities Group under E102A/HM, Effect of the Presence of Services on Road Performance.

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EXECUTIVE SUMMARY

The Department of Transport and the National Joint Utilities Group jointly collaborated on a trenching trial at TRL's Pavement Test Facility (PTF) to examine in detail the effects of two designs of trench reinstatements on the long-term performance of a Type 3 road.

The work also enabled the research into long-term damage, recommended by Professor Horne, to be set against a case for the Secretary of State for Transport to make regulations under the New Roads and Street Works Act 1991, Section 78 'Contribution to costs of making good long-term damage'. The work described in the report therefore contributes to the debate on the need to investigate long-term damage on roads due to the presence of utility services.

The trench sections were constructed in accordance with the Code of Practice issued under the Act entitled Specification for the Reinstatement of Openings in Highways (1992) (the Specification). Trafficking was applied to each of eight test sections in sequence to simulate 4½ years of in-service use. The trench excavation, pipe laying and reinstatement was carried out by a regular operational team from British Gas (South East) Plc, who had no training or expertise above that normally provided for trenching work.

The report describes the design of the Type 3 construction and the trench. It presents the results of the performance of the pavement during the initial 2 years trafficking and the following 2½ years of the life of the trenched road. It also provides details of the materials sampling, compaction in respect of CBRs, shear strengths and moisture contents, and static and dynamic strains on the pipe assembly. It considers the effects of granular and foamed concrete backfills in a trench and any effects on the wetting up of the subgrade.

After 2½ years of trafficking post-trenching the main conclusions are:

1. No consequential pavement damage was observed using either granular or foamed concrete backfills with no premature maintenance required to the trenches or the original pavement.

   (i) There was no significant change in deflection in the foamed concrete backfills and the subgrade strains were reduced minimally. The granular backfills showed a similar level of deflection to that of the original pavement and stiffened under trafficking to produce lower strains than those found immediately after reinstatement. Deflections on the inter-trench and control sections showed minimal change which was not significant.

   (ii) There was no observed visual deterioration in the surface condition of the trenched, inter-trenched and control sections. The settlement within the trenches was about 1mm or approximately 5 per cent of that allowed, under the performance requirements of the HAUC Specification, before remedial work would be required.

2. The effect of using foamed concrete as a backfill did not affect the condition of the Gault clay subgrade.

3. Strains induced by a 11.5t axle load in a buried cast iron pipe, in new condition and of typical size and in-situ depth, with granular or foamed concrete backfill, were very small compared with the allowable strains.

After 4½ years of trafficking the main conclusions are:

4. The good performance of the trenches and the pavement indicated that sustained sound performance is likely to be achieved throughout the design life of the pavement.

5. The strains in the pipes beneath the foamed concrete and the soil pressures in the surrounding finefill were smaller than those beneath the granular reinstatement, at all stages of the experiment.

The general conclusions are:

6. The standard of workmanship in excavating and reinstating the trenches was typical of current utility practice carried out in regular operational conditions.

7. The overall standard of the pavement construction, trench excavation and the various stages of pipe laying and full reinstatement were in most cases undertaken at the lower end of the requirements of the HAUC specification.

8. The work represented the worst case conditions in respect of width of excavation, depth of cover, pipe assembly, compaction, foamed concrete strength, re-use of excavated material and traffic loading.

TRENCH REINSTATEMENT TRIAL AT TRL'S PAVEMENT TEST FACILITY

ABSTRACT

The purpose of the research described in the report is to examine the effects of eight trench reinstatements, built to the HAUC Specification, on the long-term performance of a Type 3 road and the effects of traffic loads on buried pipes. A trial was constructed at TRL's Pavement Test Facility to study the structural performance of granular and foamed concrete backfill materials and their effects on overall pavement performance, pipes and the wetting up of the subgrade. A typical Gaussian distribution of trafficking, at 11.5t axle load, was applied across each of the trenches. Performance measurements were carried out at simulated 6 month intervals over a simulated 4½ year test period and included the recording of visual condition, deflection, surface profile and strains in the pipe and structure due to traffic loading.

Within the HAUC guarantee period, no consequential damage was observed using either backfill material and no premature maintenance was required to the trenches or the original pavement. No significant changes were found in deflection, visual deterioration on the control or trenched sections. Settlement within the trenches was 5 per cent of that allowed under HAUC. The good performance of the trenches and the pavement indicated that sustained sound performance is likely to be achieved throughout the design life of the pavement.

1. INTRODUCTION

In 1984 the Government set up a committee, chaired by Professor M R Horne, to review the existing Public Utilities Street Works Act 1950 (PUSWA Act 1950). Their report entitled "Roads and the Utilities" (Horne et al 1985) made 73 recommendations for improvement in street works based on two major principles; the street authorities should be made responsible for co-ordinating street works and the undertakers should be made responsible for all aspects of excavation and reinstatement. The recommendations included the provision of a new specification for the reinstatement of utility openings in highways, HAUC SWP163 (1992), containing standards of performance to be achieved by utility reinstatements after completion. The report also recognised that the evidence available at that time did not provide a solution to the problem of long-term damage. Though there was evidence that damage had occurred, this had not been quantified or analysed for the assessment of maintenance implications.

The main purpose of the research described in the report is to examine the effects of trench reinstatements (built to the HAUC Specification) on the long-term performance of a Type 3 road and the effects of traffic loads on buried pipes. To further this objective a trial was carried out at TRL's Pavement Test Facility (PTF) to study the structural performance of two backfill materials, granular material and foamed concrete, as permitted in the reinstatement specification for a Type 3 road classification. The trial was funded jointly by the Department of Transport (DOT) and the National Joint Utilities Group (NJUG), with excavation and reinstatement work carried out by British Gas (South East) Plc.

The work has also enabled some of the research into long-term damage, recommended by Professor Horne, to be carried out. The results of the work may be relevant to any future consideration by the Secretary of State for Transport of the need to make regulations under the New Roads and Street Works Act (1991) Section 78 'Contribution to costs of making good long-term damage'.

2. BACKGROUND

The New Roads and Street Works Act 1991 requires all Utilities making openings in highways and footways to reinstate them to the HAUC Specification standards and to guarantee such reinstatements for a minimum period of two to three years depending on the depth of excavation and method of reinstatement. If during the guarantee period the reinstatement falls below the performance standards required, remedial works must be carried out by the Utility in accordance with the HAUC Specification.

Although much progress has been made in recent years towards more effective reinstatements that offer improved performance, much work remains to be done if the benefits of the new Act are to be realised for the travelling public.

The Horne Report recommended improved methods and materials to raise the standard of reinstatement. New techniques for excavation and reinstatement have been proposed and tested by a number of Utilities and new materials for reinstatement have been trialed. There has been, for example, extensive use of foamed concrete as a backfill material by both British Gas Plc and the Water Companies. Use of foamed concretes has been encouraged primarily because the materials are self compacting and highly suitable for small openings where manoeuvrability of compaction plant is difficult. Foamed concrete is free flowing and can be produced in a range of densities from 600 to 2000kg/m^3 at strengths of 1 to 18 N/mm^2 at 28 days. It is suitable for use as a combined backfill/sub-base/roadbase in a Type 3 flexible road at the required minimum crushing strength of 2N/mm^2 at 28 days but cannot be used within 100mm of the road surface. Other advantages include its ability to set within 24 hours to allow the trench to be surfaced without settlement problems.

As the Street Authorities implement and monitor the effects of the new Act, it will be important for them to understand more clearly the effects of the presence of a reinstated trench on the performance of the surrounding pavement. Future increases in permitted axle loadings (11.5 tonnes maximum from 1999) suggest that it will be necessary to verify that current and possible future reinstatement techniques will adequately withstand these loads. For this reason, it is important that the stresses arising in buried apparatus due to traffic are well understood.

In January 1993 TRL began a major trial funded by DOT and NJUG to evaluate the effects on the life of the pavement of different reinstatement options and the effects on buried apparatus and the surrounding subgrade.

3. OBJECTIVES

The Horne Report, in Chapter 10 entitled 'Cumulative and long-term damage' under recommendations 10.23 and 10.24 on the possibility of general compensation, discusses the theoretical and practical considerations of compensation payments to the highway authorities in the case of long-term damage. The work described in the report, although not aimed at specifying the calculation of the charges, may be a helpful contribution to any debate on the need to investigate long-term damage on roads due to the presence of utility services.

The overall technical objectives of the trial were to evaluate, in a programme of testing under realistic wheel loads, the following:

a) The effects of the trench opening on overall pavement performance when reinstated with granular fill.

b) The effects of the trench opening on overall pavement performance when reinstated with foamed concrete.

c) The effects on the surrounding subgrade of the use of granular fill and foamed concrete reinstatements.

d) The effects of different backfill techniques on the loads applied to buried pipes installed in trenches.

Further detailed objectives, related to trafficking and performance testing, are given in sections 5.1 and 10.1 of the report.

4. EXPERIMENTAL DESIGN

4.1 GENERAL

Trenches excavated in the highway are assessed on the basis of their performance under full operational site conditions. Factors such as variable road construction, road surface temperature, weather, excavation problems and variable workmanship can contribute to the poor performance of trench reinstatements. It is recognised that performance is affected particularly by temperature and weather, and consistent control can not always be achieved with road trials. It was therefore important to evaluate trench performance under closely controlled conditions and the Pavement Test Facility (PTF) provided this opportunity.

4.1.1 Pavement Test Facility

The Pavement Test Facility is shown in Figure 1. It permits the accelerated testing of road pavements under closely controlled conditions of loading and pavement temperature. Loading is applied to the pavement by a wheel assembly mounted on a gantry frame which spans the pit and is positioned over a selected area of the experimental pavements. The line of loading can be varied to simulate the transverse distribution of wheel loading and the wheel load can be applied in one direction only or in both directions. Wheel loads are controlled to within ± 2% of the set load and are applied at speeds of up to 20 km/h. The temperature of the pavements under test is controlled by infra-red heating to ensure that the effects of other variables are not masked.

4.1.2 Test Sections

Eight test sections, TTS1 to TTS8, including two shared inter-trench sections, were designated in the PTF test pavement. The layout of the sections is shown in Figure 2. In order to meet the objectives of this work and to improve confidence in the results obtained the test sections were grouped into four pairs. One pair were used as controls, in which no trenching was undertaken; two pairs were trenched, one being reinstated with granular sub-base (GSB1) material the other with foamed concrete in compliance with the HAUC Specification; the fourth pair were used as inter-trench sections. These inter-trench sections were situated between a pair of GSB1 and foamed concrete trenched sections and were undisturbed. They were used to observe the effects of the presence of the trench on the performance of the surrounding pavement and subgrade. Further details on the test sections are given in section 4.5 of this report.

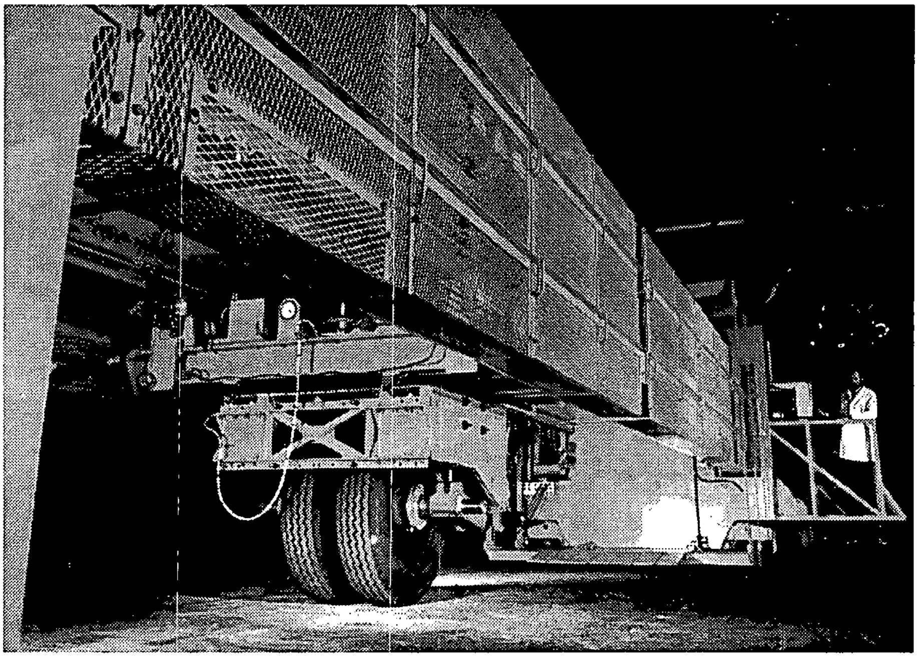

Fig.1 The Pavement Test Facility (PTF)

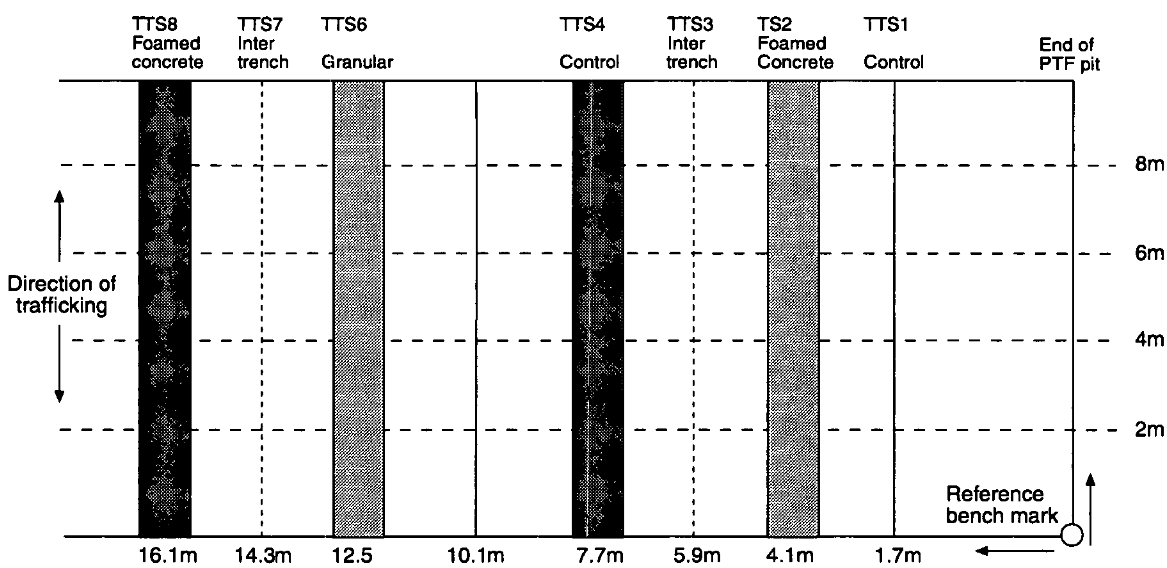

Fig.2 Layout of Trench Test Sections

4.2 PAVEMENT CONSTRUCTION

The test pavement was constructed in a 17.6 metre length of the 17.7 metre length PTF pit, and was of a typical Type 3 flexible design with a design life of 20 years or 2.5 million standard axles (msa). A wet-mix macadam roadbase was chosen because it is typical of most local roads in the UK. A large number of utility excavations have been carried out in roads of this type and many will be carried out in the future.

The PTF pit (25m long x 10m wide x 3m deep) is concrete lined and contains an imported Gault clay subgrade. This is placed to a depth of 2.45 metres and has a California Bearing Ratio of 3 - 8% as measured by in-situ CBR methods (see section 4.3) and a moisture content of about 24 per cent. To verify these characteristics a range of tests were carried out on the subgrade in a grid formation of 15 representative sample sites. The measurements taken were:

California Bearing Ratio (CBR)
Shear strength
Unconfined compressive strength
Moisture content

All test measurements were taken in close proximity to each other to ensure continuity of data.

4.3 SUBGRADE PROPERTIES

The in-situ CBR values were measured using a hand-held soil assessment cone penetrometer to a depth of approximately 75mm and the range of values recorded. These penetrometer CBR values were corrected using a relation between penetrometer CBR values and the CBR values of remoulded soil established by Black (1979) (see Appendix A Figure A1). Shear strengths were measured using a Pilcon hand vane tester, fitted with a 28mm x 19mm diameter vane. For each section two measurements were taken at the clay surface. The resulting shear strength values were corrected using a calibration factor to compensate for torque characteristics of the gauge. Compressive strengths were measured using the equipment and procedure as specified in British Standard BS1377: Part 7:1990 Method 7 (1990). The moisture content of the soil immediately surrounding the sample sites was also measured using the oven drying method as specified in BS1377:Part 2:1990. A summary of the results is shown in Table 1.

In-situ measurements using the TRL Dynamic Cone Penetrometer (DCP) (TRL 1993) were also carried out to determine the strength of the subgrade. The DCP is capable of operating at depths of up to 1200mm and where pavement layers have different strengths the boundaries can be identified and the thickness of the layers determined. Correlations have been established in earlier work (Van Vuuren 1969, Kleyn and Van Heerden 1983, Smith and Pratt 1983) between measurements with the DCP and California Bearing Ratio (CBR) so that results can be interpreted and compared with CBR specifications for pavement design. Measurements were taken between 28 August and 3 September 1992. A single DCP measurement was made at each of 18 locations at 2, 5 and 8m from the edge of the pit on PTF sections TTS1 to TTS8 at a depth of up to 900mm. The results of the subgrade CBR measurements are shown in Table 1. The subgrade in the PTF bay is a TRL standard test bed, whereas typical subgrades found in Type 3 in-service roads have considerable variability and are often contaminated with other materials.

4.4 MATERIALS AND DESIGN THICKNESSES

The eight test sections were built to the design given in Table 2. To record the design thicknesses that were achieved, optical level readings were taken on the individual layers during construction. The finished mean compacted layer thicknesses, derived from readings taken on a grid of 13 rows (0.5 metres apart) and 20 columns (section centreline ± 0.9 metres) are given in Table 3. The surface of the wearing course had no added chippings so that any cracking which might occur during trafficking could easily be seen.

TABLE 1

Subgrade test results

Test	Range	Units
Moisture content	23.1 - 25.1	%
In-situ CBR (penetrometer)	2.9 - 8.0	%
In-situ CBR (DCP)	6 - 10	%
Shear strength (vane)	120 - 205+	kPa
Shear strength (compression)	88 - 160	kPa

TABLE 2

Materials and layer thicknesses of pavement construction

Layer	Specification	Material	Thickness (mm)	Comments
Wearing course	BS594:1985 Part 1:Table 5 Column 21	Hot Rolled Asphalt	50	30/14mm size, 50 pen recipe Type F, no chippings
Basecourse	BS4987:1988 Part 1:Clause 6.5	Dense Bitumen Macadam	70	20mm nominal size, 100 pen binder
Roadbase	DOT Spec Clause 805	Wet Mix Macadam	230	-
Sub-base	DOT Spec Clause 803	Granular Type 1	225	-

TABLE 3

Final layer thicknesses

Pavement Layer	Thickness (mm)	Standard Deviation
Sub-base	226	12.54
Roadbase	230	10.83
DBM basecourse	69	7.36
HRA wearing course	53	4.25

During the pavement construction a total of 20 samples from the various pavement layers were taken and sent to Surrey County Council Materials and Testing Laboratory for compliance testing in accordance with specification. Details of the sampling and testing are given in Section 4.4.5 of the report.

4.4.1 Sub-base

The Type 1 granular sub-base was laid using a mechanical excavator and compacted in two layers using a Bomag BW120AD twin-drum vibrating roller, classified as having a mass per metre width of roll of 1300kg to 1800kg, and 16 passes were required for a 150mm compacted thickness (Plate 1). The first layer had a compacted thickness of 150mm and a compacted thickness of 75mm was achieved on the second layer.

After the second layer had been compacted Nuclear Density Gauge (NDG) measurements were taken at three positions along each of the eight test sections. The gauge is capable of making rapid density measurements accurate to within 1 to 2 per cent (TRRL Working Party 1982). The in-situ densities were recorded using a Troxler 3411B nuclear moisture/density gauge, following the procedure set out in BS1377:Part 9:1990. The readings were taken in the direct transmission mode at a depth of 200mm (8 inches). Level sample areas of the surface were chosen for the testing in order to reduce the effect of surface air voids on the accuracy of the density measurements. Samples of the material were taken at the test positions and tested for moisture content. The detailed results are given in Tables B1 and B2 of Appendix B and the readings are referred to as Test ID 'a'. When time permitted 'back-to-back' readings, i.e. where the gauge was rotated by 180 degrees with the radiation source still in the hole, were taken in accordance with BS1377:Part 9:1990. Such readings are referred to as Test ID 'b' in the Tables.

Following the container method in BS1377:Part 9:1990, a density calibration of the granular Type 1 material was undertaken. The NDG results for the material were cor-

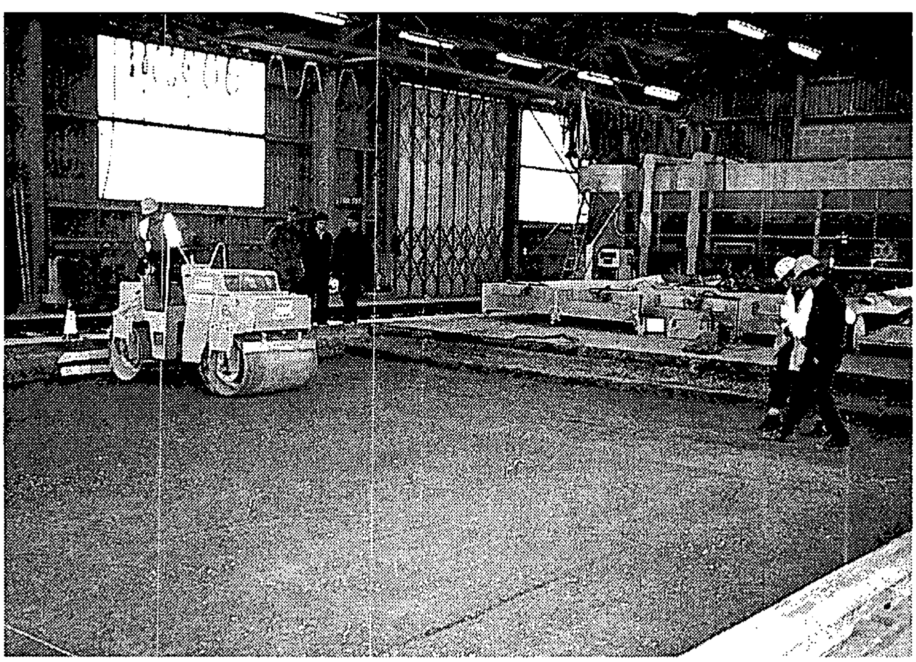

Plate 1 Road construction - final compaction of Type 1 sub-base layer

rected using this calibration. These corrected bulk density values were then converted to equivalent dry density values using the nuclear gauge moisture contents at each test position. A check using the standard oven dry method confirmed that the NDG moisture contents were acceptable.

For comparison purposes only, the Clegg Impact Soil Tester was used on the Type 1 sub-base layer as close to the NDG measurements as possible. The values were in the range 15 to 34. Some utility companies interpret a target value of 22 for sub-base material as denoting that "a satisfactory degree of compaction is likely to have been achieved".

4.4.2 Roadbase

The wet-mix macadam roadbase was laid in two layers using a mechanical paver finisher. The first layer had a compacted thickness of 110mm and the second layer a compacted thickness of 120mm. Each layer comprised three strips, two of 3.1 metres width with a central strip of 3.8 metres. Initial compaction was carried out using an 8-10 ton three-drum deadweight roller. The Bomag BW120AD was then applied to provide the final compaction. Nuclear density measurements were taken on each layer using the same method used for the Type 1 sub-base. The detailed results are given in Tables B3 and B4 of Appendix B.

It was not possible to carry out a density calibration, using the container method, on the material so the densities are uncorrected. These were converted to equivalent dry densities using the NDG moisture contents. For comparison purposes only, measurements were also made with the Clegg Impact Soil Tester using the same method as described for the Type 1 sub-base. The Clegg values were in the range 25 to 46. An evaluation of the compaction control achieved with the Clegg tester suggests that it should not be used as the sole control for reinstatement with granular materials.

Plate 2 shows the finished surface of the compacted wet-mix macadam roadbase.

4.4.3 Basecourse

The dense bitumen macadam basecourse was laid in a single layer, comprising three strips, to a compacted thickness of 70mm. Full compaction was achieved using a combination of an 8-10 ton three-drum deadweight roller and a Bomag BW120AD twin-drum vibrating roller. Samples of the basecourse were taken from the lorry for composition analysis and binder content testing. The recorded temperature of the material at the time of delivery was 125°C. Several temperature measurements were taken during compaction and they were found to be in the range 95°C to 122°C. The NDG was used in back scatter mode to measure the bulk density of the bituminous layers. The measurements were taken at 2m, 5m and 8m along each test section. The results for the basecourse layer shown in Appendix B Table B5 indicate variable compaction and lie in the range 2.223 to 2.470 Mg/m^3 with a mean value of 2.34 Mg/m^3.

4.4.4 Wearing Course

The hot rolled asphalt wearing course was laid in a single layer, comprising three strips, to a compacted thickness of 50mm. Full compaction was achieved using a combination

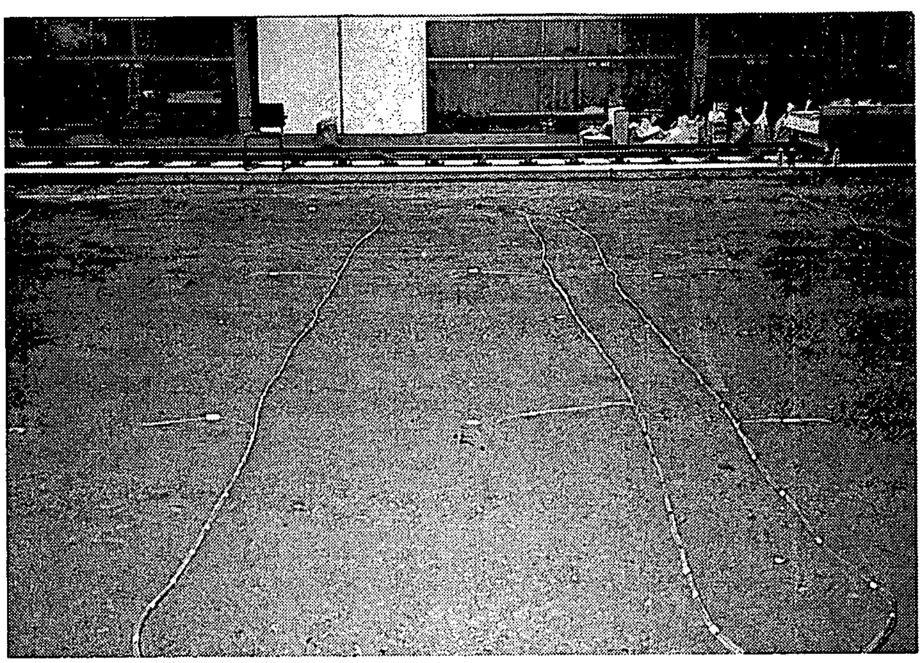

Plate 2 Compacted wet-mix macadam roadbase

of the 8-10 ton three-drum deadweight roller and a Bomag BW120AD twin-drum vibrating roller. Samples of the wearing course were taken from the lorry for composition analysis and binder content testing. The recorded temperature of the material at the time of delivery was between 150°C and 160°C. Several measurements were taken during compaction and they were found to be in the range 80°C to 110°C, compatible with requirements. The NDG was used in back scatter mode to measure the bulk density of the wearing course. The results are shown in Appendix B Table B5 and were in the range 2.246 to 2.468 Mg/m^3 with a mean value of 2.40 Mg/m^3. These densities indicate that good compaction had been achieved.

4.4.5 Materials sampling and testing

Details of the sample testing are given in Tables C1 to C4 of Appendix C. The moisture contents were in the range 1.9 per cent to 2.4 per cent for Type 1 sub-base and 3.0 per cent to 4.0 per cent for wet-mix macadam. The mean values were 2.3 and 3.4 per cent respectively. Three of the five Type 1 sub-base samples, tested in accordance with BS812: Part 103:1989, had too much material retained on the 5mm sieve. Two of the eight wet-mix macadam roadbase samples tested had too much fine material passing the 75 micron sieve and one had too much coarse material passing the 20mm sieve. In general, the compliance with specification of the granular materials was acceptable with only minor deviations from the expected moisture content values for these materials.

Three samples of the 20mm dense macadam basecourse material and four samples of the 30/14 Hot Rolled Asphalt wearing course were sampled from the lorry and tested for grading and binder content analysis to BS598: Part 102:1989 Clause 4.2. All the basecourse samples were found to be marginally out of specification in respect of low binder contents. The residual binder penetrations and softening points were in the range 51 to 58 pen and 47.4°C to 49.8°C respectively, after recovery.

One of the wearing course samples was marginally out of specification due to coarse grading on the 75 micron sieve. The residual binder penetrations and softening points were in the range 31 to 37 pen and 53.0°C to 54.6°C respectively, after recovery.

The results were received from Surrey County Council Materials Testing Laboratory some time after the pavement had been laid and trenching had started.

4.5 TRENCH DESIGN

The reinstatement options for the four experimental trenches TTS2, TTS4, TTS6 and TTS8 were designed in accordance with the requirements of the HAUC Specification for a Type 3 flexible road. The trench dimensions, bedding and finefill materials are shown in Table 4 and the trench construction layers and thicknesses are shown in Table 5.

4.6 PIPE DESIGN

The instrumented pipes were installed to represent typical Utility apparatus installations. The choice of pipe material was based on the frequency with which cast iron pipes occur in practice. Although most new installations involve the use of polyethylene pipe, the majority of existing pipe is cast iron.

As the number of trial sections was limited to four it was not practical to use more than one type of pipe. The greatest lengths of buried pipes are in "rigid" materials i.e. spun grey and ductile iron. The most convenient method of determining the relative pipe loads in the trials was to measure the external longitudinal bending strains and some circumferential strains in grey spun iron pipes of this size.

For each trench three lengths of pipe were joined using bolted split sleeves giving a total length of 9.3m as shown in Figure 3. This length was determined by the width of the PTF.

The central 3.6m length of pipe in each trench was Class B spun iron to BS1211: 1958 (1958) having a nominal internal diameter of 4" and a wall thickness of 0.3". The two end lengths of pipe were Class K9 ductile iron to BS4772: 1988 (1988) having a nominal internal diameter of 100mm and wall thickness of 6.1mm. The latter were each 2.85m in

TABLE 4

Trench dimensions and bedding/backfill materials

Trench/Material Details	Dimension/type
Width	300 - 350mm
Length	10m
Cover depth to pipe	900mm
Bedding/finefill	Crushed rock or sand (BS882 grading)
Thickness of bedding	50mm + pipe diameter
Finefill cover above pipe crown	100mm nominal

TABLE 5

Trench construction layers and thicknesses

Trench Type	Layer	Thickness (mm)	Material	Specification
TTS2 and TTS6 Granular	Wearing course	40	Hot rolled asphalt	BS594: Part 1:Table 5 Column 21:1985
	Basecourse	60	Dense bitumen macadam	BS4987: Part 1:Clause 6.5:1988
	Roadbase	320	Type 1 granular	DOT Clause 803
	Sub-base	250	Type 1 granular	DOT Clause 803
	Backfill	130	Re-excavated or Type 1 granular	HAUC Specification or DOT Clause 803
	Finefill above pipe	100	Crushed rock or sand	BS882:1983
	Bedding	50	Graded sand	BS882:1983
TTS4 and TTS8 Foamed concrete	Wearing course	40	Hot rolled asphalt	BS594:1985
	Basecourse	60	Dense bitumen macadam	BS 4987: Part 1:Clause 6.5:1988
	Roadbase/sub-base/backfill	700	Foamed concrete	3 - 4 N/mm² at 28 days BS1881: 1983
	Finefill above pipe	100	Crushed rock or sand	BS882: 1983
	Bedding	50	Graded sand	BS882: 1983

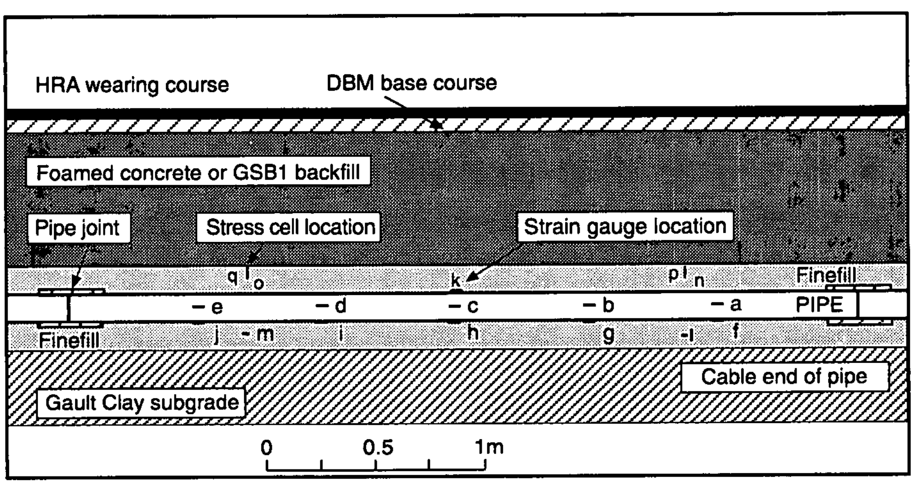

Fig.3 Longitudinal section of instrumented pipe in trench

length. Where required, shim steel sleeves were used inside the bolted split sleeves (identical to those used by Carder 1982) to ensure a rigid connection was achieved between the different types of pipe which had slightly different external diameters i.e. 3.9mm difference. The 100mm cast iron test pipe was chosen to represent a severe case in terms of the stress generated in the pipe assembly.

Details of the pipe instrumentation are given in section 6.2.

5. TRAFFICKING

5.1 GENERAL

Trafficking of the experimental pavements was undertaken specifically to examine:

a) The effect of the presence of a trench on the performance of the surrounding pavement and subgrade.

b) The relative performance of foamed concrete and conventional granular fill when used for trench reinstatement.

The pavement was constructed to a typical Type 3 flexible design, having a design life of 20 years with a predicted traffic loading of 2.5 million standard axles (msa). Assuming a 0 per cent growth rate this implies trafficking at a rate of 0.125 msa per annum. The traffic loading was selected so that it was considered to be the absolute maximum for a Type 3 road and therefore probably represented a worst case in relation to current UK permissible axle loads.

It is recognised that utility work is concentrated in residential areas where, for geometrical and other reasons, the applied axle loads could be less than those occurring elsewhere on the network. However, this is taken account of in the relevant design methods, which provide for a cumulative total of standard axles. At present no empirical evidence exists to indicate whether the application of increased axle loads i.e. 11.5t on in-service Type 3 roads is more damaging than on other types of road. The choice of a 5.75t wheel load for the present experiment therefore has no greater effect on the Type 3 road than any other category. Nevertheless, the repeated application of this wheel load to the test pavements and trenches would represent an unusually severe in-service application.

Each of the eight test sections were trafficked for a simulated 2 years, at a rate of 0.125 million standard axles per year, before any trenching was undertaken; it was assumed that trenching of a newly opened road would rarely be required during the initial 2 years of its life. Trafficking of the test sections was therefore divided into two phases; before and after trenching. This, combined with a programmed series of measurements taken before, during and after trenching, enabled the comparative performance of the trenching techniques, and their relative effects on subgrade conditions, pipe loading and pavement life to be evaluated.

Currently, there is a 2 year guarantee period for permanently reinstated trenches less than 1.5 metres deep. To observe the presence of deterioration beyond this time period the second phase of trafficking was carried out to simulate $2^1/_2$ years or 0.375 msa.

To achieve the primary objective additional trafficking lines, which were not trenched, were subjected to the same traffic loading. To measure the worst case situation where trenching occurs in the wheelpath, these additional lines were spaced with their centrelines 1.8 metres from the centreline of the trenched sections. This spacing was chosen because it is the typical wheelbase of heavy commercial vehicles in service. Due to constraints imposed by the available area of the PTF pit it was not possible to assign each trenched section an additional trafficking line. This was overcome by defining an inter-trench trafficking line, by which a trench of each backfill type shared an additional trafficking line.

In addition to the 4 trenched sections and 2 inter-trench sections, a further 2 sections were provided as control sections. These were positioned such that their centrelines were 2.4 metres from the centrelines of adjacent pavements and were subjected to the same trafficking as all other sections. The increased separation was chosen in order to ensure that their performance was not influenced by the trenching and trafficking of the other sections or trenches.

5.2 SEQUENCE OF LOADING

The initial 2 years trafficking on each section was applied in four six month cycles. To ensure that acceptable comparisons could be made between similarly trafficked sections, and to ensure also that the sequence of loading had no influence on subsequent performance, the sections were trafficked in the following sequence:

1. Section TTS3 - Inter-trench
2. Section TTS7 - Inter-trench
3. Section TTS4 - Foamed concrete
4. Section TTS8 - Foamed concrete
5. Section TTS1 - Control
6. Section TTS5 - Control
7. Section TTS2 - Granular
8. Section TTS6 - Granular

After the first phase of trafficking and following the opening and backfilling of the trenches, the trafficking order for the second $2^1/_2$ year phase was revised. This made provision for the assessment of individual trench effects on the inter-trench sections. The 6 monthly trafficking sequence was repeated as follows:

1 Section TTS5 - Control
2 Section TTS3 - Inter-trench
3 Section TTS7 - Inter-trench
4 Section TTS2 - Granular
5 Section TTS8 - Foamed concrete
6 Section TTS4 - Foamed concrete
7 Section TTS6 - Granular
8 Section TTS1 - Control

5.3 DIRECTION OF LOADING

The Pavement Test Facility is capable of trafficking pavements either uni-directionally or bi-directionally. Whilst roads in service are normally only subject to uni-directional traffic loading previous work at the PTF suggests that pavement performance is unaffected if it is loaded bi-directionally. The obvious benefit from adopting bi-directional loading is the 100 per cent increase in the rate at which wheel passes can be applied to the pavements. It was decided therefore that for this work the machine would be operated in bi-directional mode.

5.4 AXLE LOAD

The wheel load applied to the pavements represented one end of an axle loaded to the maximum 11.5 tonne limit which will be introduced in 1997 (i.e. the load on the wheel assembly was 5.75t). The unitary axle load used in pavement design is the 8.16t standard axle. Therefore, each application of the total test load represented 3.94 standard axles, based on the established fourth power relationship reported by Liddle (1962).

For the experiment the machine was fitted with a conventional dual wheel assembly as opposed to a super single wheel and tyre configuration.

5.5 PAVEMENT TEMPERATURE

Since the majority of the experimental work was undertaken during the winter season, use was made of the PTF pavement heating system. This enabled all the test sections to be trafficked at a constant temperature, thereby eliminating any possible differences in performance due to the temperature dependent performance of the bituminous material. The temperature was maintained at 20°C at a depth of 40mm below the surface by infra-red heating of the surface before and during each phase of trafficking.

5.6 DISTRIBUTION OF LOADS

To simulate the typical transverse distribution of traffic which occurs in practice, the machine was programmed to apply each 6 month traffic period in a normal Gaussian distribution across the pavement. Since the dual wheel assembly is approximately the same width as the trenches the reinstatement supported the full wheel load when on the centreline and gave rise to a total trafficked pavement width of 1.46 metres. This represented a worst case wheel loading on each trench. It resulted in an untrafficked width of 0.34 metres between adjacent trenched and inter-trench sections and 0.94 metres between trenched and control sections. A schematic diagram of the trafficking distribution is shown in Figure 4.

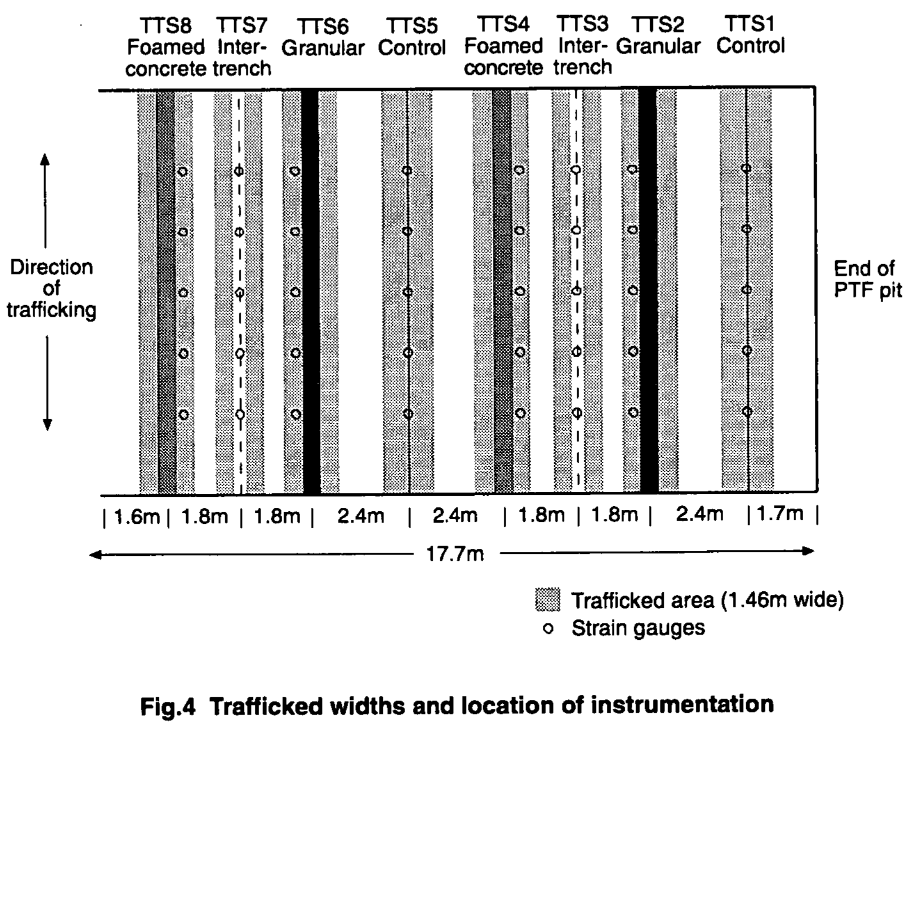

Fig.4 Trafficked widths and location of instrumentation

5.7 DURATION OF TRAFFICKING

The PTF is capable of operating 24 hours a day, and for this experiment it was possible to apply each 6 month trafficking period in 18 hours, with some additional time necessary for machine maintenance purposes and to heat the pavements upto 20°C prior to trafficking. Additional time was also required to carry out the comprehensive measurement programme.

Taking all these factors into account, the initial 2 year trafficking phase was completed in a ten week period from 27/10/92 to 4/1/93.

Following excavation and reinstatement of the trenches, the second $2^1/_2$ year trafficking phase was completed in a nine week period, from 1/2/93 to 4/4/93.

6. INSTRUMENTATION

6.1 SUBGRADE INSTRUMENTATION

Five soil strain gauges were installed in each of the eight test sections to measure the transient vertical component of soil strain. The gauges utilise a linear variable differential transformer (LVDT) and were of the type described by Potter (1969). They were placed at $1^1/_2$ metre spacings along the central 6 metres of each section with their centres 100mm below the surface of the subgrade (Plate 3).

It was intended to install all of the gauges along the centrelines of the pavements. However, due to the depth of the trenches, those gauges in the trenched sections would require to be retrieved and re-installed during the construction of the trenches. This in turn, would mean that no direct comparisons could be made between the values of strain taken before and after trenching. It was therefore decided to install the gauges along the centreline of the control and inter-trench sections but those in the trenched sections were offset by 0.4 metres (to the inter-trench side of the centreline). The layout of the gauges is also shown in Figure 4.

6.2 PIPE INSTRUMENTATION

To monitor the response of the pipeline to ground movements, due to both construction operations and the static and dynamic effects of traffic loading, eleven individual electrical resistance strain gauges were bonded to the pipe (Figure 3, gauges a to k). Only the central one of the three pipes in each trench was instrumented. One soil stress measuring cell (Kulite) was installed below the pipe and two above at each of the two instrumented locations (Figure 3, gauges l to q). Dynamic and static measurements were made as close as possible to the times when other pavement measurements were taken.

Plate 3 Installation of in-situ strain gauge in the Gault clay subgrade

Deformation of the pipes in longitudinal bending was measured by electrical resistance strain gauges connected in a quarter bridge, three wire, system. Five strain gauges (Figure 3, gauges f to j) were bonded to the external underside and five more (Figure 3, gauges a to e) to one side of the pipe, all at 0.6m separation (Plate 4). In addition, one gauge (Figure 3, gauge k) was mounted on the crown at the midpoint of the pipe. The gauges were encapsulated to prevent water ingress and the electrical circuits were checked by immersing the instrumented pipes, complete with cables, in water for 24 hours to ensure that the insulation would be maintained when the pipes were buried. The strain gauges used were Micro-Measurements type CEA-06-500UW-350, having a resistance of 350.0Ω ± 0.3 per cent and a gauge factor of 2.06 ± 0.5 per cent (specified at 75°F, 23.9°C) and were self temperature compensated for steel. Bridge completion was by the use of 350.0Ω high stability resistors.

Vertical total stress measurements were made using Kulite type 0234 soil cells. These have a pressure range of 0 to 200kPa giving a full scale output of 100mV (nominal). Individual calibrations for each cell were used in calculating stresses. The cells were located 1m either side of the midpoint of the trench. Initially it was intended to place these cells directly on the clay at the bottom of each of the

Plate 4 In-situ pressure cells above the pipe

trenches but in order to maintain the same arrangement in all of the trenches, a thin layer of finefill, on which to bed the pressure cells, was placed and hand tamped. The finefill material used was 6mm to dust carboniferous limestone at a moisture content of 7 per cent ± 1 per cent. The depth of the cells below the pavement surface was recorded. The bedding material in the trenches was excavated to provide a recess into which the bolted split sleeves were located so that the possibility of bridging was avoided.

7. TRENCH EXCAVATION

7.1 EXCAVATION

A Kukla C225 rock chain excavator, fitted with tungsten carbide picks, running on caterpillar tracks and fitted with a transverse spoil conveyor belt (Plate 5) was used to excavate the four 320mm wide trenches, in Sections TTS2, TTS4, TTS6 and TTS8, to a depth of 1050mm. It is capable of excavating up to 500m per day to a depth of 1.6m. Good clean vertical trench walls were achieved with the plant with no undercutting although some surface damage occurred. It was thought this might be due to a low binder content in the wearing course but samples tested at Surrey County Council Materials Laboratory showed full compliance in respect of grading and binder penetration to BS594:1985 with the exception of one sample which was marginally out of specification on the 75 micron sieve. The excavated granular material was stored nearby and sheeted over to prevent loss of moisture content, loss of fines and contamination. It was examined by TRL and British Gas personnel to establish its suitability for re-use as backfill in accordance with the classification in Section S5 of the

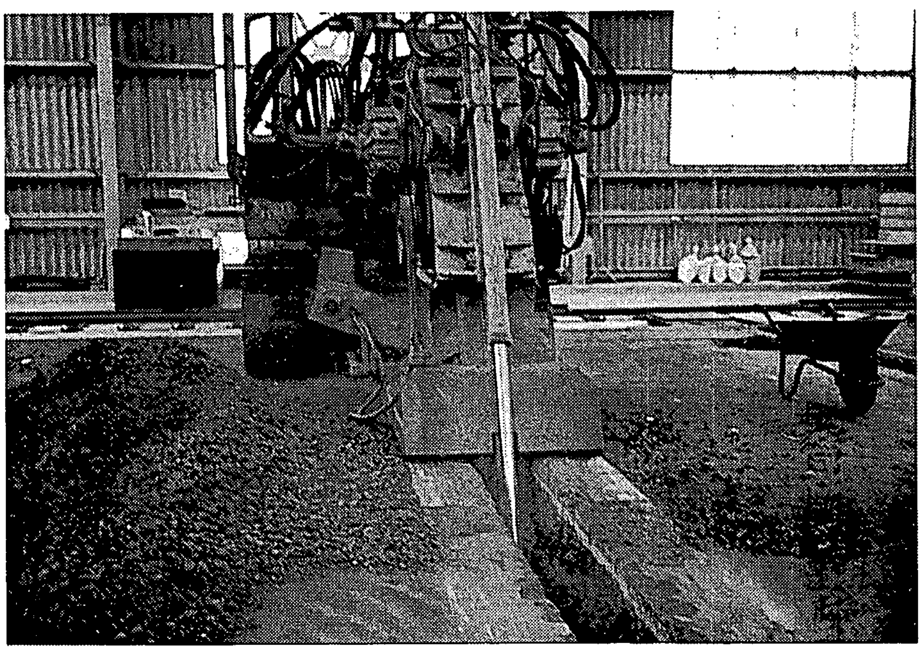

Plate 5 Kukla excavating a trench

HAUC Specification. It was considered to be acceptable as a Class C material although the moisture content was lower than its optimum. This simulated the worst case in terms of re-use of excavated material.

The personnel who carried out the excavation were selected from the nearest British Gas Plc depot and were typical trained and experienced utility operatives. The excavation plant was standard and the workmanship was typical of current utility practice. The depth of pipe cover was selected to be significantly deeper than usual and the trench width was similarly chosen to be wider than that commonly used for the installation of a 100mm nominal diameter pipe. These aspects represented extreme excavation conditions.

7.2 SUBGRADE MEASUREMENTS

Immediately after excavation and before pipe installation and backfilling, subgrade moisture contents, CBR values and shear strength values were measured in the 4 trenches. The data were either obtained on the (cleared) trench bottom at a depth of approximately 1050mm from the pavement surface or at depth of approximately 850mm from the pavement surface for the trench side-walls values.

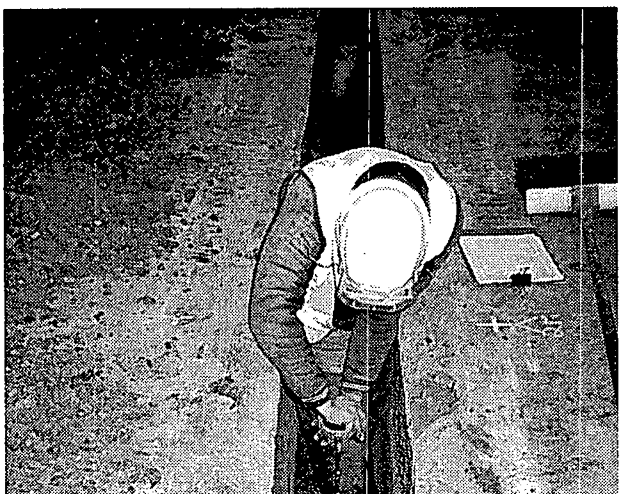

Plate 6 Shear strength measurements on the trench bottom

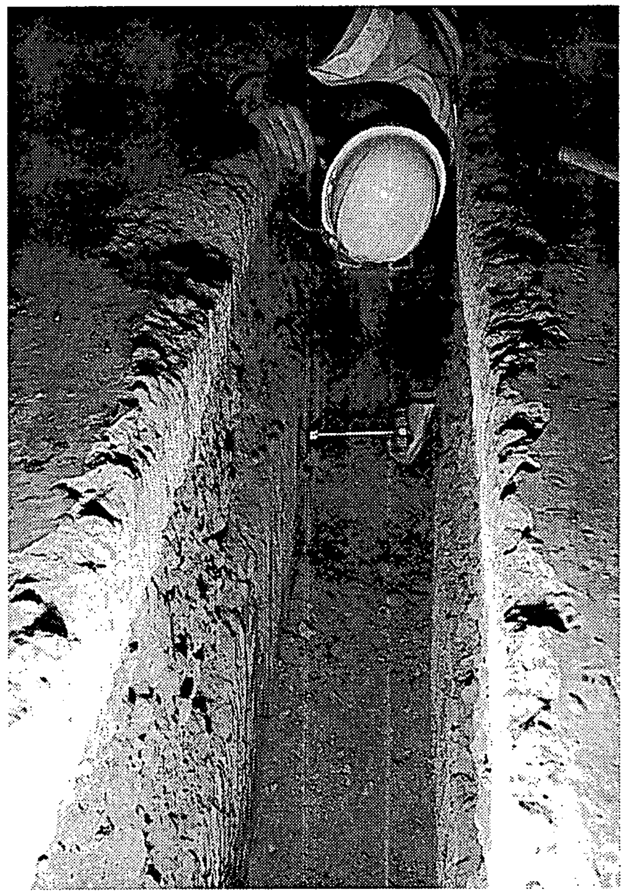

Plate 7 Shear strength measurements on the trench side walls

As before, shear strengths were measured using a Pilcon hand vane tester (Plates 6 and 7) fitted with a 28 x 19mm diameter vane. Measurements were taken at the clay surface. The recorded shear strength values were corrected for gauge torque characteristics, using a previously established calibration.

The in-situ CBR values were measured using a hand-held soil assessment cone penetrometer (Plate 8), at a depth of approximately 50mm and where a range of CBR values were observed, the average was recorded. The penetrometer CBR values were corrected using the relation between penetrometer CBRs and the CBRs of remoulded soils established by Black (1979).

The subgrade results are given in Figures D1 to D3 of Appendix D and were used for comparison with the subgrade strength measured at the end of the trial after excavation of the foamed concrete trenches (see section 12 in the report).

7.3 PIPE LAYING AND FINEFILL

Trench TTS8 was accidentally excavated marginally deeper than specified. Finefill, used as bedding for the pipe, was placed to the required depth and compacted using a Wacker BS45Y vibro-tamper with extension legs and a standard

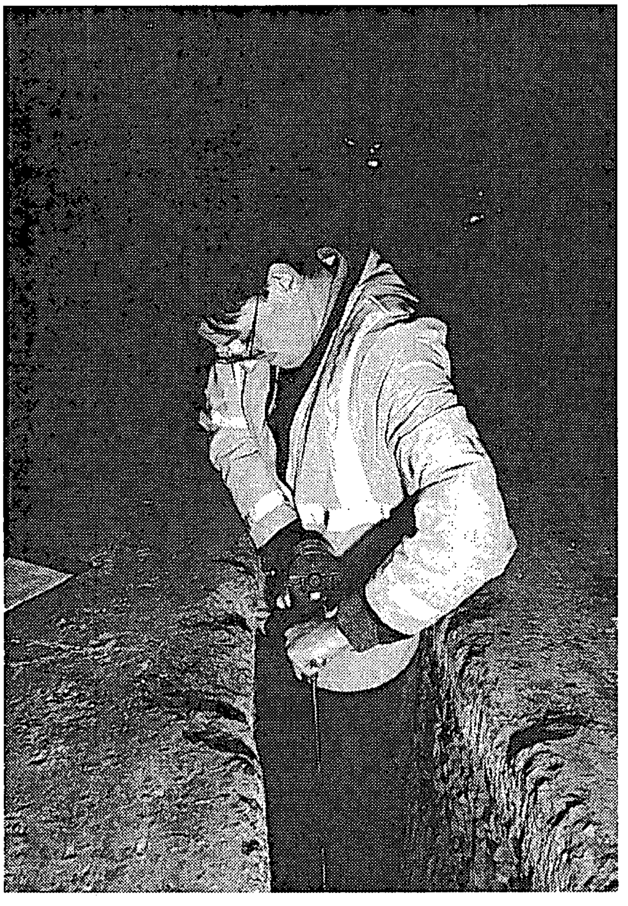

Plate 8 Soil assessment hand-held cone penetrometer on the trench bottom

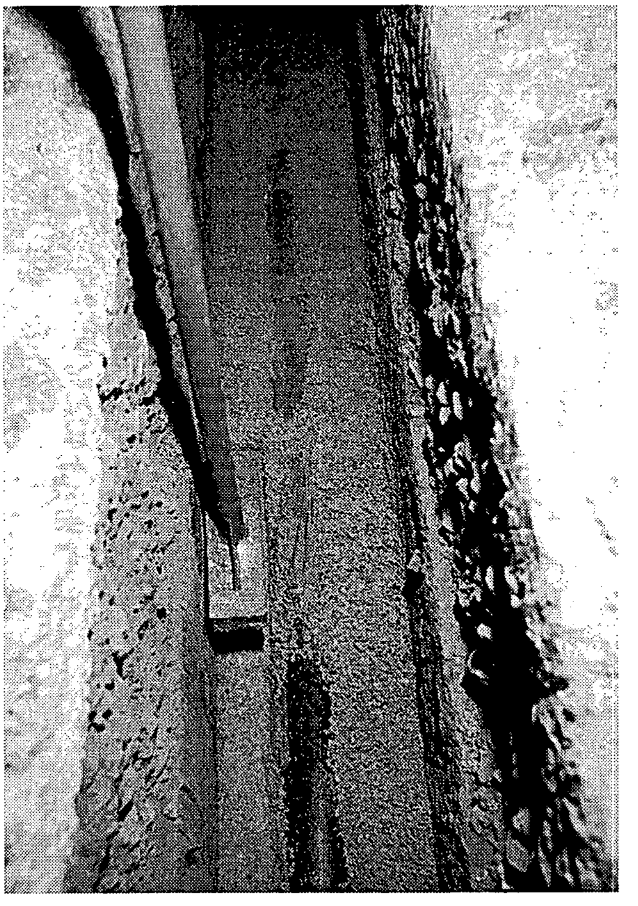

Plate 9 Hand tamping of the finefill around the pipe

foot width of 200mm. The Wacker was not used to compact material within approximately 300mm of the pressure cells; this was carefully hand tamped to avoid damaging the cells.

Fine material was placed and hand-tamped around the pipe (Plate 9) and then the pipe covered by finefill, which was compacted using the Wacker, to approximately 800mm below the pavement surface. The finefill above the pipe was excavated at positions corresponding to the pressure cells below the pipe. This enabled two pressure cells to be located at 25mm and 90mm above the pipe giving a total of six pressure cells in each trench. The upper cells were offset from one another along the axis of the pipe by approximately one cell diameter to avoid mutual interference. The fill was replaced around the cells and carefully hand tamped. The upper cell in each case was located with its upper face flush with the top of the finefill. Approximately 20mm of additional hand tamped finefill was then placed over the area containing the cells to protect the cells during reinstatement of the trenches. The cables from the pressure cells were taken out along one edge of the trench and a protective covering of finefill was applied. The hand tamping on the cables gave the worst case for compaction.

A sample of the finefill material was sent to Surrey County Council Materials Laboratory for composition analysis, moisture content and compaction testing. It was tested to BS1377:Part 2:1990 and found to be within the grading specification with a moisture content of 6.6 per cent. Its maximum dry density was found to be 2.29 Mg/m^3 with an optimum moisture content of 6.9 per cent as tested to BS1377:Part 4:1990 Method 3.7.

8. BACKFILL SUB-BASE AND ROADBASE REINSTATEMENT

8.1 GRANULAR BACKFILL

Reinstatement of the trenches was carried out using granular backfill in Sections TTS2 and TTS6, compacted using a Bomag BG100 vibrating roller (over 3500kg/m single drum). In order to simulate the worst case when re-use of suitable material is permissible, the excavated spoil classified as HAUC Class C, was used to reinstate the initial backfill layer. A sample of excavated material was sent to Surrey County Council Materials Laboratory and tested to BS1377:Part 2:1990 Clause 9.2 and confirmed as a cohesive granular material. Its maximum dry density was 1.90 Mg/m^3 with an optimum moisture content of 15.0 per cent. The sampled material had an average moisture content of 10 per cent indicating that it was drier than its optimum value (see Appendix F Figures F1 and F2).

Subsequent layers, including the roadbase (see Table 5 for the specified thicknesses), were backfilled using Type 1 granular material (carboniferous limestone at a moisture content of 2.8 per cent ± 0.5 per cent). A sample of Type 1 material was sent to Surrey County Council Materials Laboratory and tested to BS1377:Part 2:1990 Clause 9.2. It complied with the grading specification and had a moisture content of 2.7 per cent. Its maximum dry density was 2.18 Mg/m^3 with an optimum moisture content of 3.10 per cent as tested to BS1377:Part 4:1990: Method 3.7. A cross section of the granular trench is shown in Figure 5.

8.2 FOAMED CONCRETE BACKFILL

The foamed concrete for test trenches TTS4 and TTS8 was supplied by Ready Mix Concrete (RMC) to a specification supplied by British Gas Plc. The foaming agent was synthetic, Cormix AE4, and the material was supplied as a standard pre-batched concrete mix for foaming on site. Samples of the foamed concrete were taken from the mixer truck, from the mixer chute on delivery of the material into the trenches and from the trench during laying to determine the 7 and 28 day compressive strengths (Plate 10). A check of the density was made, at the time of sampling, using a bucket method and it was found to be in the range 1420 to 1432 kg/m^3. The surface of the foamed concrete was levelled off using a 'scraper board' to nominally within 100mm of the existing pavement surface (Plate 11).

Polystyrene moulds, 150mm in size, were selected for the sampling (Plate 12) because of their insulating properties which simulate 'field' conditions inside the trench. Steel moulds were also used as a comparison. The results of the

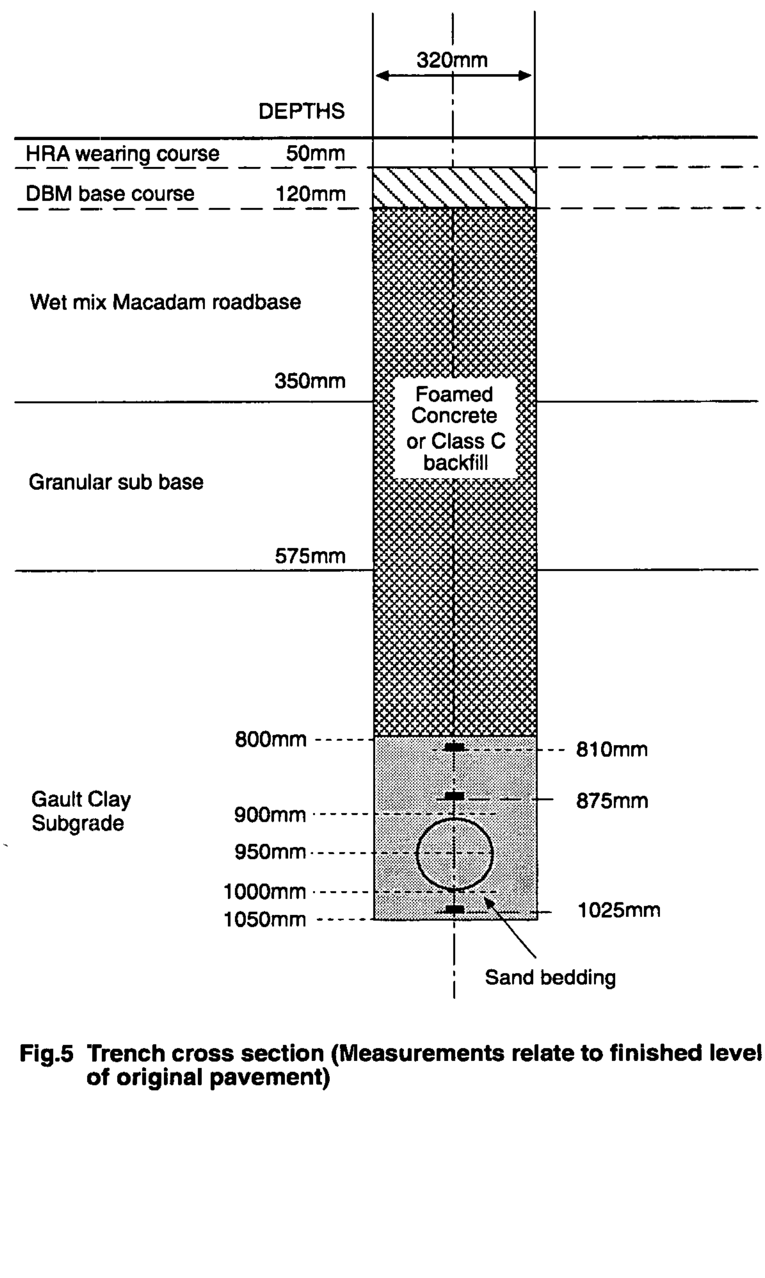

Fig.5 Trench cross section (Measurements relate to finished level of original pavement)

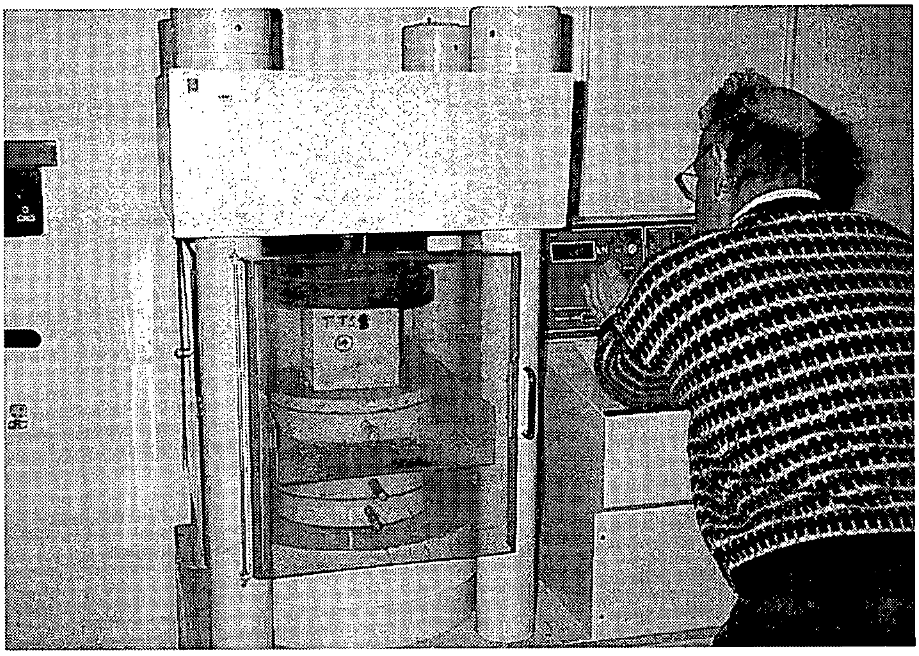

Plate 10 Cube test to determine compressive strength

Plate 11 Levelling off the foamed concrete in a trench

7 and 28 day compressive strengths are shown in Tables 6 and 7. In general, samples cured in the polystyrene moulds produced a higher compressive strength at 7 and 28 days. This method was adopted under the recommendation of highway materials laboratories. Compressive strengths from moulds upturned in water were more representative of expected values than strengths found by other methods of curing. Experience has now shown that these results are most representative of in-ground conditions when obtained from 150mm cube samples prepared using lidded foamed plastic moulds, stored at ambient temperature within the mould (DOT et al 1992).

Two samples taken from the mixer truck produced the highest compressive strengths when cured in polystyrene moulds. The compressive strength of the foamed concrete was very close to the minimum allowed by the HAUC Specification i.e. 2N/mm^2 at 28 days. It represented material quality likely to be achieved in operational conditions.

8.3 COMPACTION

Compaction data were recorded during the backfill and sub-base construction of the trenches. A Bomag BG100 trench compactor, fitted with a compacting wheel of 240mm width, was used as the main compactor. It falls into the 'over 3500kg/m single drum' category in the HAUC Specification. The finefill material surrounding the pipes and associated instrumentation was manually compacted with hand rammers and finished with a Wacker BS45Y vibro-tamper. For subsequent layers the Bomag was used for the main compaction and the vibro-tamper for levelling and compaction of the layers at the trench ends. Clegg and nuclear density gauge readings were taken on each com-

Plate 12 Polystyrene cube samples of foamed concrete

pacted layer (100mm in total) using the methodology described earlier in section 4.4.1 of this report. It was noted that the applied compaction generally met the minimum requirements of the HAUC Specification in respect of layer thicknesses and number of compaction passes for each type of plant used.

During the compaction of the unbound layers and after the foamed concrete had set, British Gas Plc (ERS) undertook soil assessment cone penetrometer readings and Clegg measurements with observations of the placement and compaction procedures. The details of these measurements are given in Appendix E, Tables E1 to E10.

8.3.1 Granular trench TTS2

After the finefill material had been compacted a layer of excavated material was placed and compacted using the Bomag compactor. Type 1 granular material was then placed and compacted in three layers. A summary of the compaction results is shown in Figure F1 and Table F1 of Appendix F.

The Type 1 granular material was compacted with 4 passes on the top layer (Plate 13). To ensure that the level of the final layer would meet the required target of 130mm below the pavement surface, a thin layer of sieved Type 1 granular material was added and compacted with a further 4 passes.

At each test location, two nuclear density gauge readings were taken back-to-back. For the layer of Class C material (Plate 14), the probe depth setting was 100mm (4 inches) and for all the Type 1 layers, the depth setting was 150mm (6 inches). The density values recorded were corrected with a calibration established using the vibrating hammer test given in BS1377:Part 4:1990 Method 3.7. The resulting dry density values were the average of the back-to-back corrected readings, calculated using the moisture contents as sampled for each layer. The Clegg Impact Values were also measured at the same positions. The Clegg values were in the range 17 to 27. Some utility companies interpret a target value of 22 for sub-base material and 30 for road base material as denoting that "a satisfactory degree of compaction is likely to have been achieved" (Plate 15).

A comparison with the BS1377: 1990 vibrating hammer test has been included in the results as an indicator of the effectiveness of the method specification. For an acceptable state of compaction, a typical ratio of the field density compared to the maximum vibrating hammer density (relative compaction) would be about 95 per cent. The relative compaction values achieved in the present trial were in the range 93 to 100 per cent.

8.3.2 Granular trench TTS6

This trench was backfilled with one layer of excavated material and three layers of Type 1 granular in the same manner as trench TTS2. Figure F2 in Appendix F shows a summary of the compaction data for TTS6 and Table F2 shows the corresponding Nuclear Density Gauge data. To achieve a final target depth of 130mm below the existing surface, an additional thin layer of Type 1 was added to the top layer after 4 passes of the Bomag compactor. Another 4 passes were made to complete the compaction of the layer. The relative compaction values achieved were in the range 90 to 99 per cent. The Clegg values were in the range 22 to 29.

TABLE 6

Foamed Concrete - 7 day compressive strengths

Sample Origin	Compressive Strength (N/mm^2)	Wet Density (kg/m^3)	Dry Density (kg/m^3)	Moisture Content (%)	Curing Method
Mixer truck	2.3	1320	1192	10.7	
Mixer truck	2.6	1368	1237	10.6	Cured in polystyrene moulds
TTS4	2.0	1302	1173	11.0	upturned in water
TTS8	2.8	1334	1203	10.9	
MEAN	2.4	1331	1201	10.8	
Mixer truck	2.3	1441	1254	14.9	In water after removal from
MEAN	2.3	1441	1254	14.9	polystyrene moulds
Mixer chute #	1.6	1430	1225	16.7	
Mixer chute #	2.0	1455	1254	16.0	In water after removal from
Mixer chute #	1.7	1433	1254	16.0	steel moulds
MEAN	1.8	1439	1238	16.2	

TABLE 7

Foamed Concrete - 28 day compressive strengths

Sample Origin	Compressive Strength (N/mm^2)	Wet Density (kg/m^3)	Dry Density (kg/m^3)	Moisture Content (%)	Curing Method
Mixer truck	4.2	1371	1246	10.0	
Mixer truck	3.9	1347	1224	10.0	
TTS4	2.8	1335	1212	10.1	Cured in polystyrene moulds
TTS4	3.2	1340	1219	9.9	upturned in water
TTS8	3.3	1325	1204	10.0	
TTS8	3.3	1307	1187	10.1	
MEAN	3.4	1337	1215	10.0	
Mixer truck	2.8	1400	1219	14.8	
TTS4	2.1	1422	1206	17.9	In water after removal from
TTS8	2.7	1424	1222	16.5	polystyrene moulds
MEAN	2.5	1415	1216	16.4	
Mixer chute #	2.5	1526	1280	19.2	
Mixer chute #	3.1	1498	1242	20.6	In water after removal from
Mixer chute #	2.4	1498	1242	20.6	steel moulds
MEAN	2.7	1514	1263	19.9	

Sampled from mixer chute discharging into trench TTS4

Plate 13 Compaction of Type 1 sub-base material with Bomag compactor

Plate 14 Nuclear density gauge in use on excavated material (Class C)

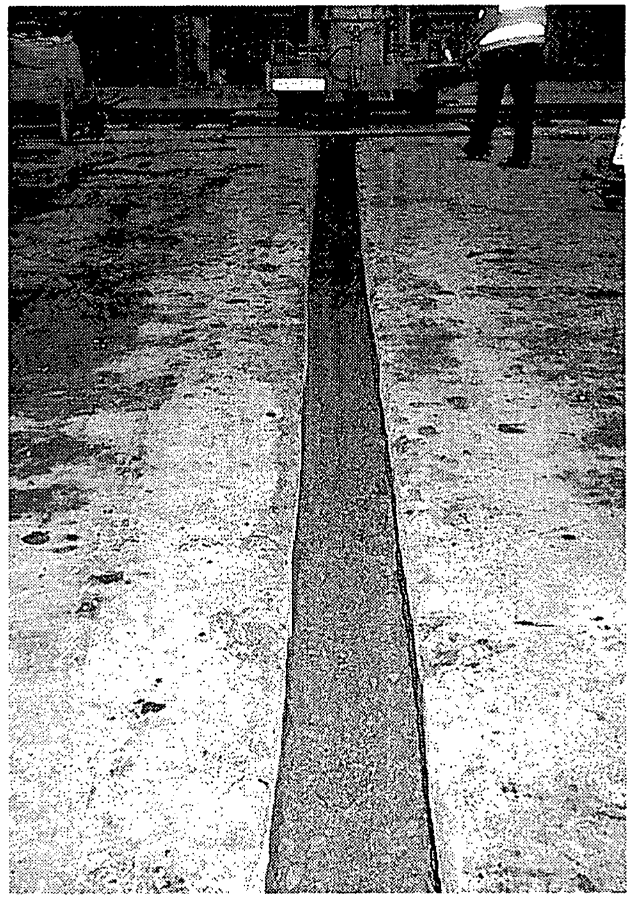

Plate 15 Finished compaction Type 1 sub-base (section TTS6)

8.4 TRENCH LAYERS AND INSTRUMENTATION LOCATIONS

Details of the final trench construction layers (compacted thickness) and the instrumentation locations are given in Table 8.

9. REINSTATEMENT OF PAVEMENT SURFACE

9.1 MATERIALS

The original construction had a surfacing of Hot Rolled Asphalt (Recipe Type F, 50 pen, no chippings) and it is a requirement of the HAUC Specification that a consistent surfacing across the pavement is maintained. The materials selected for the final reinstatement were 20mm Dense Macadam basecourse to BS4987 (1988) and 30/14mm Hot Rolled Asphalt wearing course with no chippings with layer thicknesses of 70mm and 50mm respectively. Surface trimback, approximately 150mm wide, was used on all the

TABLE 8

Details of trench layers and instrumentation locations

		Section TTS2	Section TTS4	Section TTS6	Section TTS8
Depth to base of trench		1049 *	1082 *	1104 *	1174 *
Depth to top of lowest cells	cable end (c)	1040mm	1040mm	1045mm	1040mm
	far end (f)	1040mm	1040mm	1040mm	1040mm
Depth to top of bedding material		798 *	799 *	794 *	796 *
Depth to crown of pipe	c	875mm	860mm	872mm	900mm
(before compaction)	f	872mm	860mm	875mm	890mm
Depth to top of middle cell	c	835mm	820mm	835mm	850mm
	f	830mm	820mm	840mm	850mm
Depth to top of upper cell	c	800mm	800mm	800mm	800mm
	f	805mm	800mm	800mm	800mm
Depth to top of fines (above cells)		780mm	780mm	780mm	780mm
Depth to top of 1st compacted layer		686mm		677mm	
Depth to top of 2nd compacted layer		465mm		467mm	
Depth to top of 3rd compacted layer		284mm		313mm	
Depth to top of 4th compacted layer		128mm		121mm	

* Data from British Gas Plc Research and Technology Division (mean of 8 measurements)

test sections prior to the laying of the wearing course because of the unexpected brittleness of the existing HRA wearing course (apparent at the time of cutting the trenches). Thickened edge sealant (Ayton Asphalte) was applied to the vertical edge of the basecourse layer and the horizontal planed surface of the trimback was tack coated before the laying of the bituminous wearing course.

A sample of the basecourse and wearing course materials were taken and sent to Surrey County Council Materials Laboratory for composition analysis, binder penetration and softening point testing. The results (DBM4 and WC5) are shown in Appendix C, Tables C3 and C4. The DBM sample was marginally outside the specification on the 14mm sieve which had too much fine material passing and a low binder penetration. The HRA sampled complied with specification.

9.2 COMPACTION

A Wacker BS65Y vibro-tamper (setting 3) was used to compact the basecourse (Plate 16) and a Bomag BW120AD twin vibratory roller (category 1000 - 2000kg/m, Table A8, HAUC Specification) was used to compact the wearing course (Plate 17). Details of the compaction passes are given in Table 9. The number of compaction passes applied were generally at the lower end of the requirements of the HAUC Specification; it is likely therefore, that the bituminous material was in the worst acceptable condition. On delivery the temperature of the DBM was 154°C and at the time of compaction it had dropped to about 90°C. The wearing course temperature was 164°C on delivery and it had dropped to about 130°C at the time of compaction. A good joint between the existing surfacing material and the new material was achieved (Plate 18).

During the reinstatement Clegg values and depth results were also taken by British Gas Plc (ERS) on the basecourse and wearing course layers with observations of the placement and compaction procedures. The details of these measurements are given in Appendix E, Tables E2, E5, E7 and E10.

10. PERFORMANCE TESTS

Following reinstatement of the trenches the remaining 2½ years trafficking were applied to all test sections as described earlier (see section 5.2). A typical normal

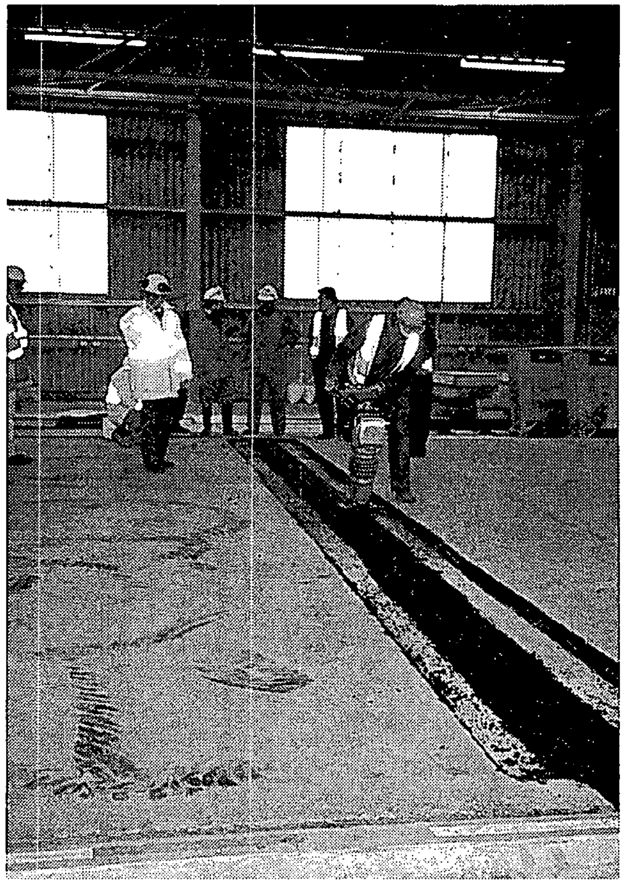

Plate 16 Compaction of the DBM basecourse layer with a Wacker BS65Y

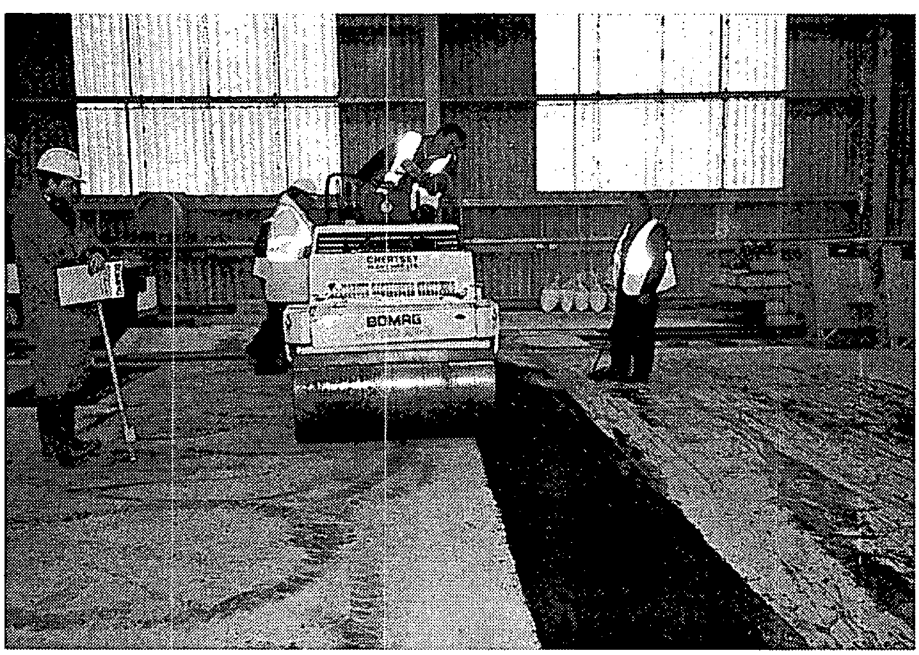

Plate 17 Compaction of the HRA wearing course layer with twin vibrating roller

TABLE 9

Compaction of bituminous materials

Trench Test Section	Material	Compactor	No. of Passes
TTS2	20mm DBM Basecourse	Vibro-tamper	9**
	30/14 HRA Wearing Course	Bomag BW120	5 *
TTS4	20mm DBM Basecourse	Vibro-tamper	9**
	30/14 HRA Wearing Course	Bomag BW120	5 *
TTS6	20mm DBM Basecourse	Vibro-tamper	9**
	30/14 HRA Wearing Course	Bomag BW120	5 *
TTS8	20mm DBM Basecourse	Vibro-tamper	9**
	30/14 Wearing Course	Bomag BW120	5 *

* + 3 passes without vibration to nip-in material
** + 4 passes at setting 1 to nip-in material

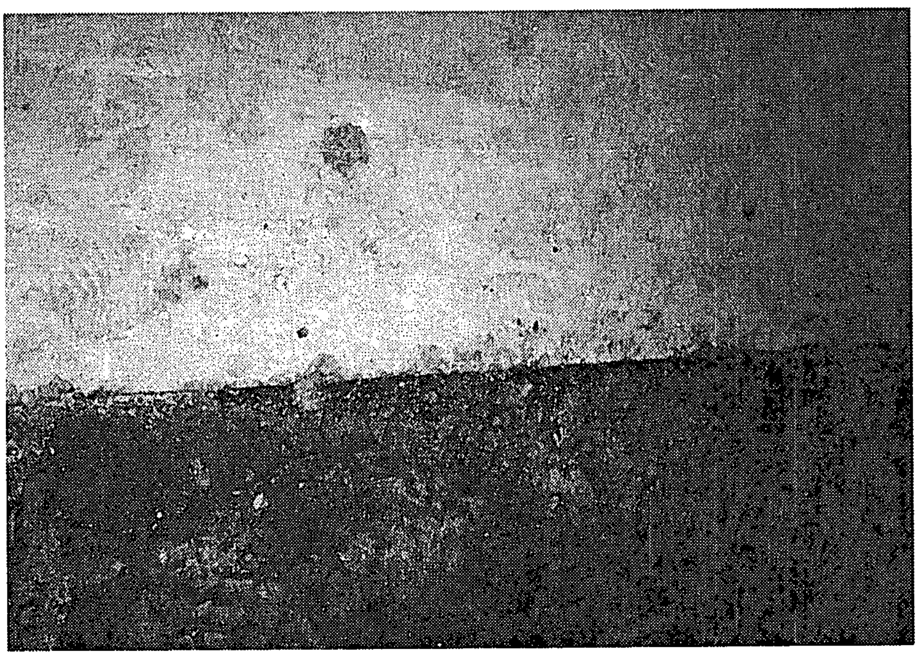

Plate 18 Good joint at trench edge (section TTS4)

(Gaussian) distribution of trafficking was achieved across each of the trenches. Plate 19 shows one of the sections after $4^1/_2$ years trafficking. Performance testing was carried out at simulated 6 months intervals in service from the time of the initial pavement laying to 6 months beyond the HAUC guarantee period (2 years) for a trench <1.5m deep. Tests included visual condition assessment, FWD measurements, surface profiles and straightedge and wedge measurements together with data logging of the pipe and structural strains due to traffic loading.

10.1 OBJECTIVES

The experiment was designed to provide information on a number of aspects of trench reinstatement performance and the methods of measurement adopted therefore depended on the particular objective. The performance requirements given in Section S2.2 of the HAUC Specification were used as the criteria to judge surface profile and hence trench settlement or rutting.

The main objectives of the performance testing were:

a) to quantify the long-term damage, if any, caused to the pavement when a trench is excavated and reinstated;

b) to determine the effects on pavement life of using different backfill materials; and

c) to evaluate any need for premature pavement maintenance.

10.2 PERFORMANCE MEASUREMENTS

In order to establish relative performance before and after trenching, it was necessary to carry out a number of measurements on the pavement to establish its condition at any given time. The tests were carried out on a pre-determined grid, cell size 1m x 1m, over the main part of the experimental area, thus allowing measurements to be made on the reinstated trench and the undisturbed surrounding pavement. Using the grid as a basis for referencing the position at which measurements were taken, the following tests were carried out at the equivalent of 6 monthly intervals in service corresponding to the application of approximately 62,500 standard axles from the start of trafficking on the trench.

Deflection using Falling Weight Deflectometer

In-situ gauges in undisturbed pavement to measure traffic-generated strains

Settlement using straightedge and wedge

Transverse profile using optical levelling

Visual condition

Extraction of cores at end of experiment

10.3 SUBGRADE STRAINS

Strain gauges were installed as described in section 6.1 of the report. They were positioned along the centre lines in test sections TTS1 and TTS5 (controls), TTS3 and TTS7 (inter-trench) and 400mm to the side of the centre lines in

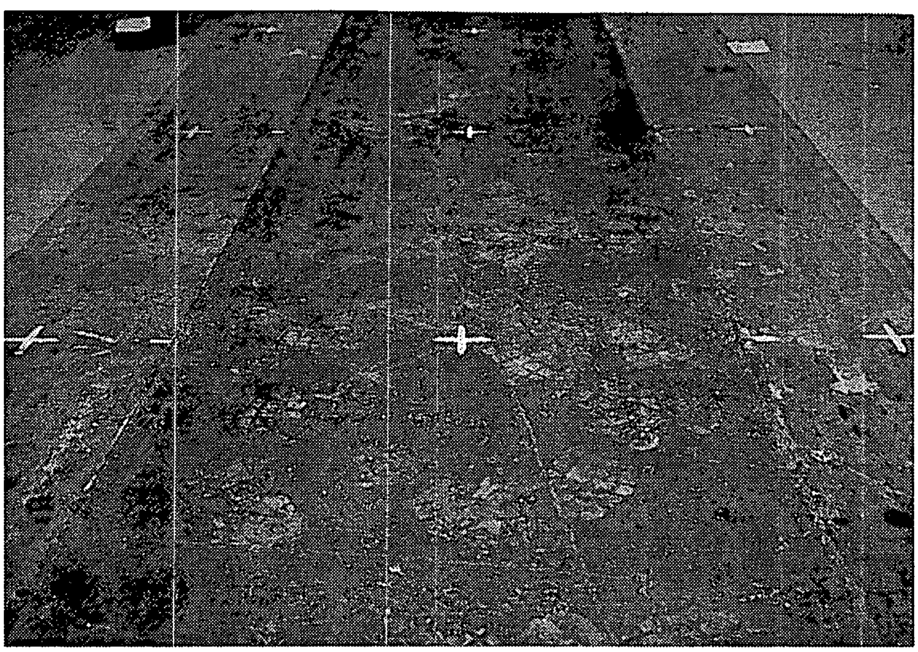

Plate 19 Typical distribution of trafficking across each trench

test sections TTS2, TTS4, TTS6 and TTS8 as shown in Figure 4. Strains were measured with the wheel moving at approx 20km/h directly over the gauge line.

10.3.1 Subgrade Results

The strains, in Figure 6, showed a normal pattern; an overall increase over the initial 2 years of trafficking, recovery on cessation of trafficking during trenching followed by return to the former level 6 months after trenching and little change thereafter.

The controls (TTS1 and TTS5) and inter-trench (TTS3 and TTS7) sections did not differ from the original pavement structures over the initial 2 years trafficking. Although the trend was similar to the inter-trench sections, section TTS1 (control) showed a higher but not significant strain value throughout the experiment.

The strains immediately after trenching and backfilling with granular Type 1 material (TTS2 and TTS6) were slightly higher than those found on the original pavements. After 2½ years these strains were virtually unchanged from their level immediately after trenching. The foamed concrete test sections (TTS4 and TTS8) showed a small but not significant reduction in strains; they rose slightly during the first 6 months trafficking after trenching and then continued unchanged for the remaining 2 years trafficking.

The subgrade strain measurements were measured 6 months beyond the specified 2 year guarantee period for trenches of approximately 1m deep and were not used to predict long-term pavement lives. Although pavement life is an important criteria it was agreed that further trafficking to destruction would be required to accurately determine this. In view of the good performance of the trenches and pavement over a period of service of 4½ years it was considered that sufficient knowledge had been gained about subgrade strains to indicate that sustained sound performance is likely to be achieved throughout the design life of the pavement.

In summary, the measurements showed no significant differences in performance of the trenched and untrenched pavements, based on subgrade strains. In respect of the reinstatement methods, measurements of subgrade strain indicated no deterioration of the foamed concrete reinstatement after 2½ years of trafficking following reinstatement. At the same age, the granular backfill, having produced subgrade strains rather higher than before trenching, had stiffened, leading to lower strains than immediately after reinstatement and only slightly higher than those measured on the foamed concrete test sections. Absolute pavement life could not be determined without further trafficking to destruction. The indication was that sustained sound performance of the trenches and pavement is likely to be achieved throughout the design life of the pavement.

10.4 FALLING WEIGHT DEFLECTOMETER MEASUREMENTS

Surface deflection is used extensively to assess pavement performance. For instance, there is a standard method of interpreting Deflectograph deflections from which estimates of residual life can be obtained. In the UK deflections are also measured using a Falling Weight Deflectometer (FWD) which provides a further means of assessing pavement strength and rates of deterioration. The FWD was specifically used at the PTF because detailed measurements along the trench were required. Unfortunately, the Deflectograph was too large to operate within the PTF bays.

FWD measurements are influenced by the temperature of the bituminous layers and need to be corrected to a standard temperature before useful interpretations and comparisons can be made. However, at present in the UK there is no standard method of correcting the FWD deflections to a uniform temperature, normally 20°C. Such a correction has to be developed for each particular site.

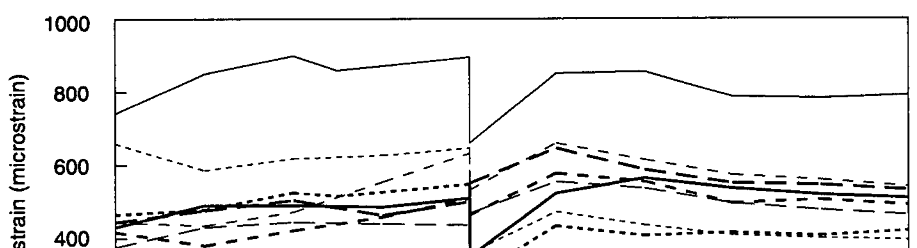

Fig.6 Subgrade strain development

All FWD deflections were measured at ambient temperature, allowing for the last pavement trafficked to cool down overnight before testing with the FWD. Pavement temperatures were measured at a depth of 40mm at the time of FWD testing. Because the inter-trench sections were shared measurements were made on these sections each time an adjacent trenched section had been trafficked. This was to establish any effects caused directly by the trafficking of either a granular or foamed concrete trench.

10.4.1 FWD results

The temperatures found during the measurements were in the range 9 to 13°C and it was originally planned to correct all deflections to 20°C. To determine the required correction factor one of the sections was tested with the FWD soon after trafficking, at a pavement temperature of 19°C, then again 2 days later at a pavement temperature of 12°C. The results from these tests showed that the deflections had reduced by 16 per cent. When this effect was considered in conjunction with the effect of 'recovery', it was found that the change in deflection due to changes in temperature was 1 per cent per °C. To minimise any errors associated with this relationship, and since all the measurements were taken in the range 9 to 13°C, it was decided to correct the deflections to 10°C rather than 20°C. This relationship also agrees with the effect of temperature on deflection reported by Kennedy and Lister (1979).

The deflection results, corresponding to FWD measurements after trafficking each section, are shown in Figures 7 to 10. On all pavements they decreased during the first two years trafficking, mainly due to the consolidation of the construction layers and stiffening with age. The control sections, TTS1 and TTS5, and the inter-trench sections, TTS3 and TTS7, showed a decrease in deflection at 24 months between the pre-trenching and after trenching measurements. This was probably due to the physical 'recovery' or increase of the elastic stiffness of the bituminous material which occurred during the break in trafficking. This 'recovery' quickly disappeared during the first phase of post-trenching trafficking as shown in Figures 7 to 10.

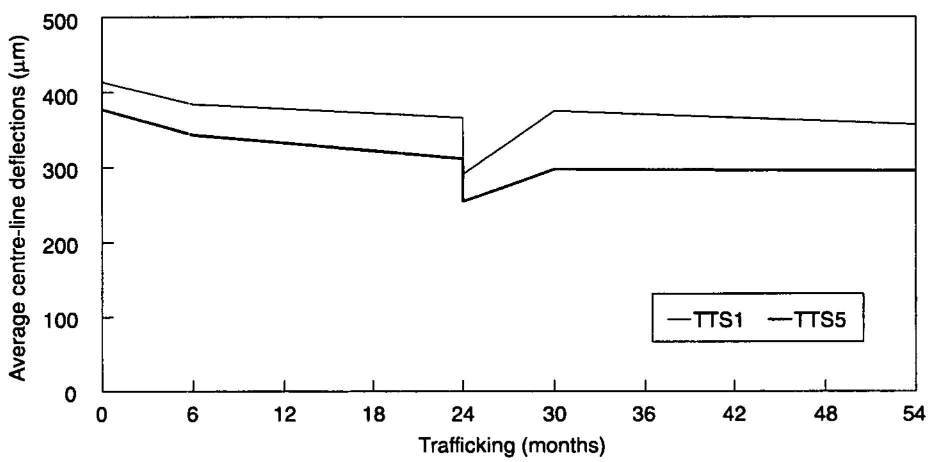

Fig.7 Deflections measured by FWD (control sections)

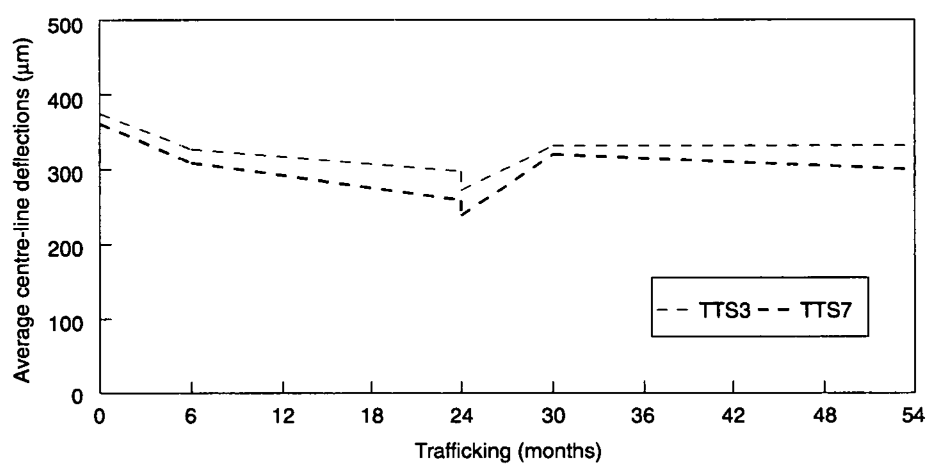

Fig.8 Deflections measured by FWD (inter-trench sections)

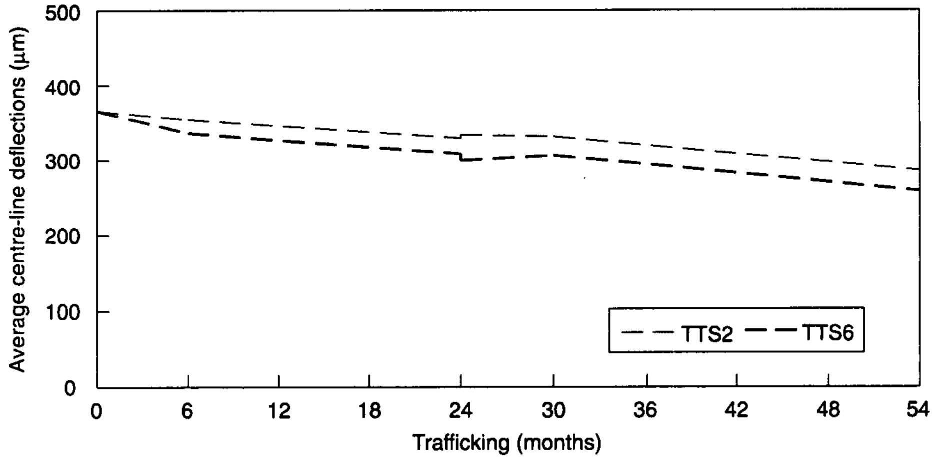

Fig.9 Deflections measured by FWD (granular reinstatements)

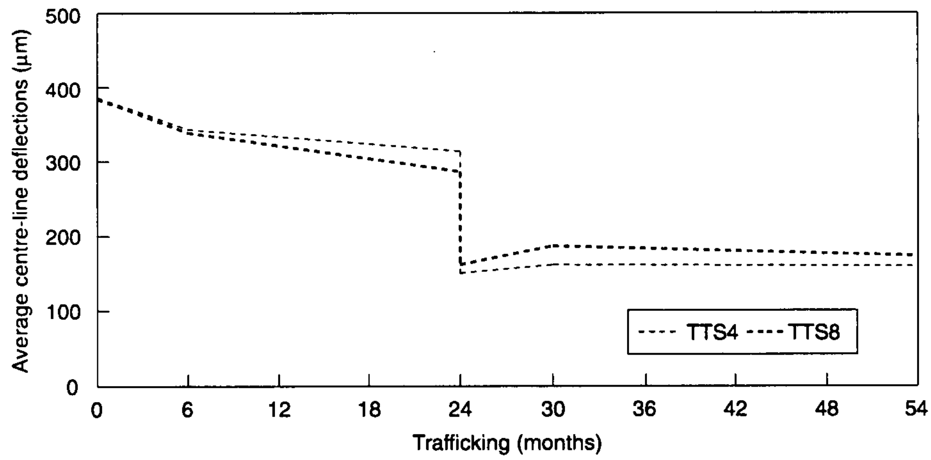

Fig.10 Deflections measured by FWD (foamed concrete reinstatements)

In summary, after the 2½ year trafficking period, the deflections on the control sections, TTS1 and TTS5, were the same as those found before trenching. The deflections on the inter-trench sections, TTS3 and TTS7 were slightly higher, about 10 - 15 per cent, after trenching. The granular reinstatements, TTS2 and TTS6, showed the same level of deflection as the original pavement with a small decrease in deflection over the 2½ year trafficking period. The foamed concrete sections, TTS4 and TTS8, showed deflections significantly lower than the original construction. Overall, the trends in deflection during the 2½ year trafficking period were not significantly higher than those expected for each section.

10.5 SETTLEMENT AND VISUAL CONDITION

10.5.1 General

Settlement or change in transverse profile was measured using both an optical level and a straightedge and wedge. Data was collected after the original pavement was constructed and at intervals of 6 months' trafficking thereafter. The more accurate optical measurements are presented in the report.

Optical levels were taken along the section centre-lines and along lines 0.9m either side of centre line (every 0.5m from 1.5m to 8.5m) on sections TTS1, TTS3, TTS5 and TTS7. This approximates to a 2m straightedge:

Settlement (change in profile) = (average c/l level) - (average at c/l ± 0.9m)

On the trenched sections, TTS2, TTS4, TTS6 and TTS8, optical levels were taken along the centre-line and along lines at ±0.1m, 0.2m, 0.3m, 0.5m and 0.9m from the centre-line. The development of settlement after trenching was calculated from the change in levels after trenching. Thus, any settlement brought about by trenching, but before trafficking, became the new datum from which changes in profile due to trafficking were measured.

Visual condition assessments of all the pavement were made after the initial 2 years' trafficking and again at intervals of 6 months for the remaining 2½ year trafficking period to assess the extent of cracking and loss of material in the surface of the reinstatements and the surrounding pavement.

10.5.2 Results

After the first 2 years of trafficking and prior to trenching, all sections showed an increase in settlement or change in profile of 6mm. This was the average increase along the centre-line. Figures 11 and 12 show that over the next 2½ years of trafficking after trenching the control and inter-trench sections produced an increase in settlement of about 1mm. Figures 13 and 14 show the settlement in the trench sections developed over the initial 6 months after trafficking with an average value of 3mm over a 1.8m width. The granular backfill had a slightly higher level of settlement compared to the foamed concrete sections but this was not significant. The settlement in the trench was also plotted and calculated from centre-lines and levels 0.2m either side of the centre-line. This showed that there was settlement of 1mm or less in the trenched sections and after 2½ years no maintenance was required on the trench surfacings or at depth. In addition, no premature maintenance was required on the surrounding pavements within 1.8m of the edge of the trenches.

In summary, there was no visible deterioration in the surface condition of the test sections after trenching. In respect of settlement and visible surface defects the experimental results fully complied with the performance requirements given in section S2.2 of the HAUC Specification for a 320mm width of trench. The settlement within the trenches was about 1mm or approximately 5 per cent of that allowed before remedial work would be required. The majority of the settlement (change in profile) could be attributed to the horizontal movement of the bituminous material rather than the increase in settlement of the backfill materials. From performance measurements i.e. profiles and visual condition surveys made over 2½ years there was no observed consequential damage of any kind. The performance of the trench reinstatements were at least as good

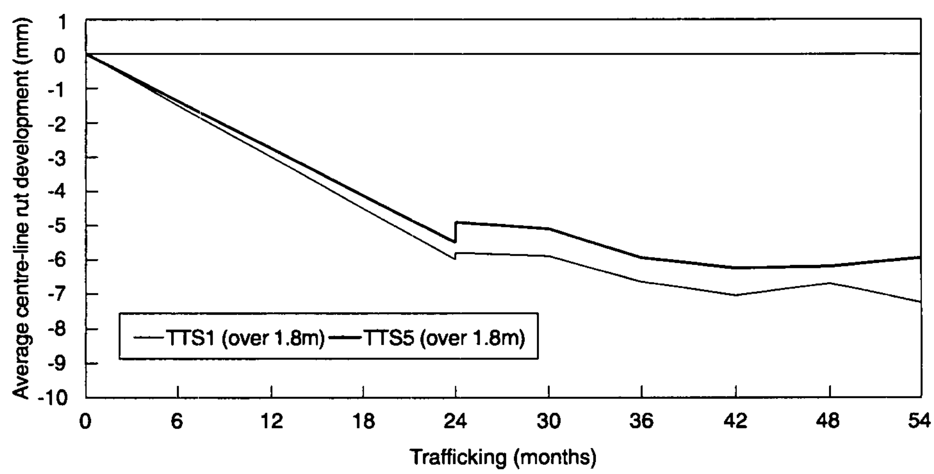

Fig.11 Profile development in control sections

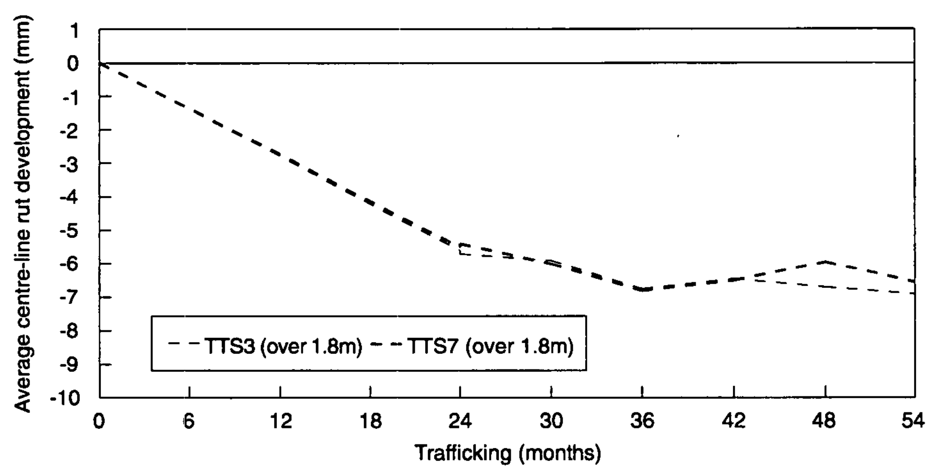

Fig.12 Profile development in inter-trench sections

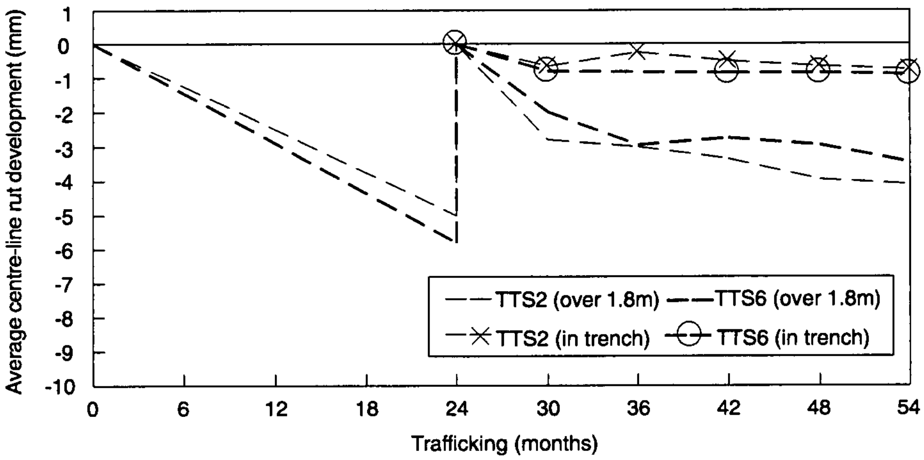

Fig.13 Profile development in granular backfill sections

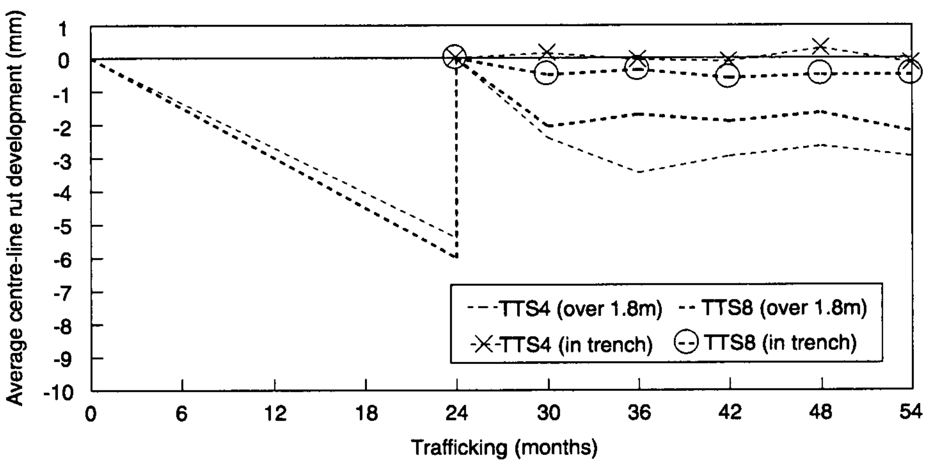

Fig.14 Profile development in foamed concrete backfill sections

as the original pavement construction with no premature maintenance required either on the trenches or the original pavement.

11. PIPE MEASUREMENTS

11.1 STATIC MEASUREMENTS

The strain gauges and stress cells were powered using a regulated supply at 10Vdc. A Solartron microvoltmeter with 1μV resolution was used to determine voltage changes from which the soil pressures and pipe strains were determined.

During the installation of the pipes and reinstatement of the trenches, only static measurements of soil pressure and pipe strains were taken. A datum for the strain gauges was established for each pipe (except for the pipe in Section TTS8) by taking a set of readings with the pipe laid on the bedding material prior to any further backfilling. Subsequent readings were then taken at all stages of backfilling and reinstatement of the pavement and approximately daily thereafter.

The datum for each pressure cell was obtained by taking a reading immediately before it was installed. Each cell was also checked to ensure that it was operational. Measurements were taken at all stages at the same times as the strain gauge readings were taken.

11.2 DYNAMIC MEASUREMENTS

Dynamic strain and soil pressure measurements were made during loaded wheel passes at the beginning and end of each 6 month trafficking period. Signals from the strain gauges and pressure cells were amplified as required using Axxon CyberAmp programmable amplifiers and then fed into a Cambridge Electronic Design (CED) 1401 high speed digital data acquisition unit. This equipment is controlled by programmes running on an IBM PC-AT compatible computer. The CED1401 is a 16 channel unit with a single 148kHz 12 bit analogue to digital converter and 16Mbytes of RAM. The amplifiers allow for up to 20000 times amplification. The low frequency signals generated by this experiment meant that the high sampling frequencies possible with this equipment were not required: sampling was carried out at 1kHz on each channel. Each signal acquired was of approximately 7 seconds duration which was sufficient for two wheel passes; it had been observed that the character and, to a lesser extent, the amplitude of the signals differed in some cases depending upon the direction of the wheel's travel. The limitation of 16 channels required that one of the instruments on each of the trenches was not monitored. This was chosen to be one of the strain gauges closest to one end and on the side of the pipe in each case.

The overall resolution of the strain measurements was ± 194.2/2048 = ± 0.09μs and the resolution of the pressure measurements was ± 200/2048 = ± 0.1 kPa = ± 100Pa.

11.2.1 Results

(i) Static measurements

Figures G1 to G12, illustrating the variation in the static measurements with time, is given in Appendix G. Table G1 gives the cross reference between the positions shown on Figure 3 and the legend numbers included in the figures in Appendix G. During installation the pipes were strained with respect to the datum sets of measurements by up to approximately 160μstrain for the top/bottom gauges and 20μstrain on the sides of the pipes. In relation to these strains the changes in static strain during the trafficking period were small: in the granular material changes were generally less than 40μstrain and less than 20μstrain in the trenches reinstated with foamed concrete. The largest changes in static strain occurred when the wheel load was applied to the trenches. This produced peaks of 50μstrain in the pipes beneath both types of reinstatement. These strains may be compared to the acceptable failure strain for these types of pipe specified in BS1211:1958 of approximately 2000μstrain. Furthermore, Pocock et al (1980) showed that actual failure of new pipes occurred at strains in excess of 3000μstrain.

During installation the soil pressure recorded below the pipes towards the cable end of each trench gave a high reading of up to 200kPa which decayed by varying amounts during the monitoring period. The soil pressures monitored at all other locations showed a general increase over the monitoring period, although within this there was a considerable variation. The peak changes beneath the granular material were approximately twice those experienced in trenches reinstated with foamed concrete: up to approximately 80kPa and 40kPa respectively. In addition, the pressures beneath the foamed concrete were less variable than those beneath the granular backfill.

(ii) Dynamic measurements

Figures H1 to H12, illustrating the variation in the dynamic measurements with time, is given in Appendix H. Appendix G Table G1 gives a cross reference between the positions shown on Figure 3 and the legend numbers included in the figures in Appendix H. Dynamic strains experienced by the pipes beneath granular reinstatements were up to 40μstrain during trafficking. Beneath the foamed concrete the strains were even less: after the initial set of data at the start of the first stage of trafficking the strains remained approximately constant at less than 12μstrain. The top and bottom pipe strains in the granular trenches demonstrated a saw tooth effect; there was an increase in strain (tension on the bottom of the pipe and compression on the top) during each 6 month trafficking period followed by a relaxation before the next trafficking session. This trend was less clearly defined by the side pipe strains and the magnitudes of the strains were generally smaller. These effects did not occur in the pipes beneath foamed concrete. All of these strains are, again, extremely small in comparison to the failure strains given above.

The dynamic soil pressures beneath the granular reinstatements were up to 60kPa at the start of trafficking. These decayed with repeated trafficking, showing a saw tooth trend, to a maximum value of 23kPa at the end of the $2^1/_2$ years trafficking. Beneath the foamed concrete the dynamic soil pressures were generally constant at or below 3kPa with the exception of one of the cells beneath the pipe in section TTS4 which peaked at 10kPa at the start of trafficking and decayed to 4kPa at the end of the $2^1/_2$ years trafficking.

In summary, peak dynamic strains, experienced by the pipes during trafficking, beneath the granular reinstatements remained fairly constant at up to 40μstrain. Beneath the foamed concrete they remained constant at less than 12μstrain. These strains may be compared with the acceptable failure strain for this type of pipe in new condition of approximately 2000μstrain. It should also be noted that the results reported were achieved in a severe application of pipe loading.

12. WETTING UP OF THE SUBGRADE

12.1 GENERAL

In order to study the effects of using foamed concrete as a trench backfill on the Gault clay subgrade, examination of the finefill for evidence of infiltration of the foamed concrete was carried out. A series of in-situ soil tests was made immediately before the trenches were backfilled and after re-excavation at the end of the total $4^1/_2$ years trafficking.

Arrangements were made for British Gas Plc to excavate the 2 trenches, TTS4 and TTS8, after sawcutting and removal of the bituminous material. It was agreed that a number of cores would be extracted from the foamed concrete using a dry cutting method to avoid wetting the subgrade. Cores of 1:1 ratio to diameter core length were required for testing purposes i.e. nominally 150mm diameter. Location of the cores was 2.5, 3.5, 4.5, 5.5, 6.5 and 7.5m from the end of each foamed concrete trenched length. Three cores, 2 from trench TTS8 and 1 from trench TTS4, were tested for their residual compressive strength by Contest Melbourne Weeks Ltd (NAMAS laboratory) on behalf of RMC Ltd who supplied the foamed concrete. The measured compressive strengths after 270 days were 2.90N/mm^2 and 3.30N/mm^2 for trench TTS8 and 2.85N/mm^2 for trench TTS4. The mean of these values i.e. 3.01N/mm^2 is about 9 per cent higher than the initial mean value of 2.7N/mm^2 at 28 days.

12.2 SUBGRADE MEASUREMENTS

Moisture contents and vane shear strengths were measured on the surface of the exposed clay at various locations along the trench sidewalls and along the centre-line of the trench bottom. In addition, CBRs were measured on the bottom of the trench. The locations along the sidewalls were at the interface of the foamed concrete and clay subgrade, approximately 850mm from the pavement surface. After re-excavation readings taken along the surface of the trench bottom were 150mm below the original level as excess material was accidentally removed from the trench during this phase of the work. Moisture contents were obtained by the oven dried method given in BS1377: Part 2:1990. The shear strengths were measured using a Pilcon hand vane tester; the reported results were corrected using a previously established calibration factor. A hand-held soil assessment cone penetrometer was used to measure the CBRs; the results presented were corrected using the relationship established by Black (1979).

12.2.1 Subgrade Results

Changes in the moisture contents are the most important, and the most accurate and repeatable, indicator of the state of the clay subgrade. The recorded values are presented in Table I1 of Appendix I . These show only very small increases in the average moisture content, about 1.4 and 1.5 per cent, at the bottom of the trenches; it must be stressed that the final set of readings were taken at a level 150mm below the initial set and, therefore, may be less than the values at the initial positions. Values obtained on the trench sidewalls were virtually unchanged and indicate that there was no significant change in the subgrade moisture content associated with the foamed concrete backfill.

Shear vane results are given in Table I2 of Appendix I and the CBRs are shown in Table I3. These results also indicate a small change in the softness of the clay at the trench bottom but this may be due to the condition of the clay at the lower level as mentioned previously. Along each of the sidewalls the average values show virtually no change in strength; any point to point changes are within the accuracy of the devices, or are brought about by variations in the precise location of the test points.

In summary, the use of foamed concrete as a backfill has not affected the condition of the Gault clay subgrade.

13. DISCUSSION

Professor Horne examined the need for research to be carried out into the long-term damage to roads caused by the presence of utility services. His recommendation that existing procedures for assessing the residual life of roads should be exploited to monitor the effects of utility reinstatements on road damage were accepted by the government. New specifications were, therefore, drawn up and applied nationally without variation, including the provision of a specification on the standard of performance to be achieved by reinstatements after completion.

All field trials carried out on in-service roads will suffer, to an uncertain degree, from the unquantifiable effects of water and temperature variations within the highway structure and from an imprecise knowledge of the weight, and probably the frequency of vehicle axles. Such trials, although providing valuable performance data, do not allow the effects of individual factors affecting that performance to be observed or explained.

In the PTF trial, a range of pavement and pipe measurements were examined under controlled conditions where the effects of weathering and temperature were minimal. The work carried out was more difficult in respect of pavement construction, excavation, pipe laying, materials compaction and reinstatement than that undertaken on in-service roads due to the confines of the PTF. However, because of the control exercised, the trial provided detailed information about many of the performance characteristics of pavements and pipes in relation to the requirements of the HAUC Specification and increased axle loading.

Decisions by practitioners at time of excavation are based on their personal engineering judgement and experience. The selection of suitable backfill materials, including the re-use of excavated material, is particularly difficult when the environmental conditions at the time of laying cannot be guaranteed. During the experiment the performance of foamed concrete was assessed against conventional Type 1 granular sub-base and the results have shown that it can be used with confidence.

The test trenches were not completed to an untypically high standard; the reinstatements were within the range of the requirements of the HAUC Specification but at the lower end and certainly represented this standard of quality in respect of excavation, width of excavation, depth of cover, pipe design, compaction passes, foamed concrete strength, re-use of excavated material and traffic loading.

14. CONCLUSIONS

After $2^1/_2$ years of trafficking post-trenching the main conclusions are:

1. No consequential damage was observed using either granular or foamed concrete backfills with no premature maintenance required either on the trenches or the original pavement.

   (i) There was no significant change in deflection in the foamed concrete backfills and the subgrade strains were reduced minimally.

   (ii) The granular backfills showed a similar level of deflection to that of the original pavement and stiffened under trafficking to produce lower strains than those found immediately after reinstatement.

   (iii) The subgrade strain measurements showed no significant differences in the trenched and untrenched sections.

   (iv) The deflections on the inter-trench and control sections showed minimal change which was not significant.

   (v) There was no visible deterioration in the surface condition on the trenched, inter-trench and control sections. The settlement within the trenches was about 1mm or approximately 5 per cent of that allowed, under the performance requirements of the HAUC Specification, before remedial work would be required for a 320mm width of trench.

2. The effect of using foamed concrete as a backfill did not affect the condition of the Gault clay subgrade.

3. Strains induced by a 11.5t axle load in a buried cast iron pipe, in new condition and of typical size and in-situ depth, with granular or foamed concrete backfill, were very small compared with the allowable strains.

After $4^1/_2$ years of trafficking the main conclusions are:

4. The good performance of the trenches and pavement indicated that sustained sound performance is likely to be achieved over the design life of the pavement.

5. The strains in the pipes beneath the foamed concrete and the soil pressures in the surrounding finefill were smaller than those beneath the granular reinstatement, at all stages of the experiment.

The general conclusions are:

6. The standard of workmanship in excavating and reinstating the trenches was typical of current utility practice carried out in regular operational conditions.

7. The overall standard of the pavement construction, trench excavation and the various stages of pipe laying and full reinstatement were in most cases undertaken at the lower end of the requirements of the HAUC specification.

8. The work represented the worst case conditions in respect of width of excavation, depth of cover, pipe assembly, compaction, foamed concrete strength, re-use of excavated material and traffic loading.

15. ACKNOWLEDGEMENTS

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APPENDIX A: RELATION BETWEEN MEASURED CBR VALUES AND CONE PENETROMETER CBR VALUES

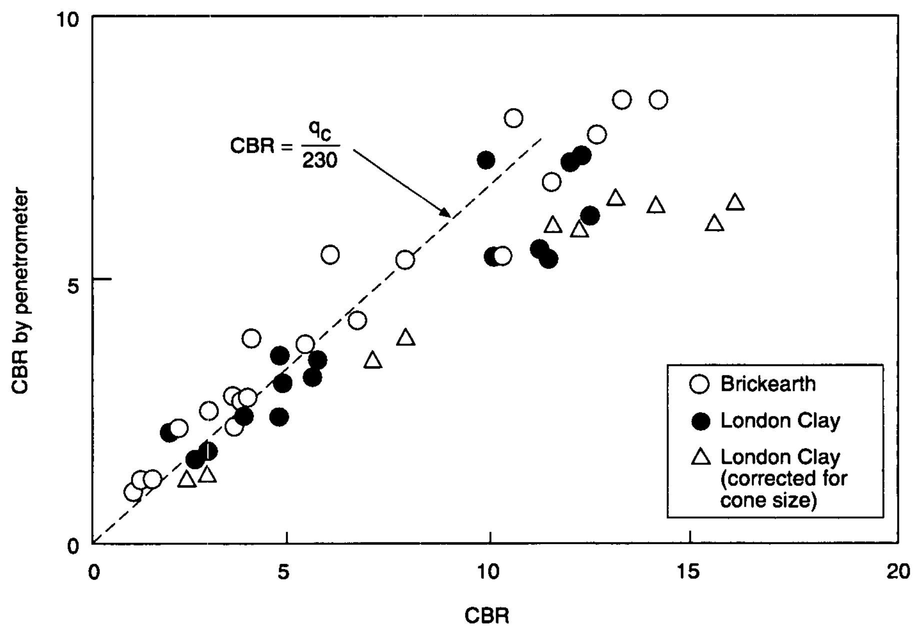


Fig.A1 Relation between measured CBR values and cone penetrometer CBR values

Reference: TRRL Report 901, WPM Black (1979)

APPENDIX B: MOISTURE CONTENT AND DENSITY MEASUREMENTS OF GRANULAR AND BITUMINOUS MATERIALS

TABLE B1

Granular Type 1 sub-base
Sections TTS1 - TTS4

Trench ID	Offset from Datum	Test ID	Troxler 3411B Values Bulk Density (Mg/m^3)	Troxler 3411B Values Moisture Content (%)	Corrected Bulk Density* (Mg/m^3)	Corrected Dry Density** (Mg/m^3)
TTS1	5	a	2.100	2.2	2.049	2.005
		b	1.944	2.4	1.940	1.895
	4	a	2.153	2.6	2.085	2.033
	9	a	2.172	1.7	2.099	2.064
TTS2	2	a	2.205	2.5	2.122	2.070
	6	a	2.033	2.2	2.002	1.959
	8	a	2.180	2.4	2.104	2.055
TTS3	2	a	2.294	2.4	2.184	2.132
	5	a	2.078	2.9	2.033	1.976
	8	a	1.963	2.3	1.953	1.909
TTS4	2	a	2.067	1.7	2.026	1.992
		b	2.058	2.3	2.019	1.974
	5	a	2.191	2.6	2.112	2.058
		b	2.178	2.3	2.103	2.056
	8	a	2.143	2.5	2.079	2.028
		b	2.050	2.4	2.014	1.967

* Troxler NDG Bulk density values corrected using container method given in BS1377:Part 9:1990

** Dry density calculated using Troxler NDG gauge moisture content

TABLE B2

Granular Type 1 sub-base
Sections TTS5 - TTS8

Trench ID	Offset from Datum	Test ID	Troxler 3411B Values Bulk Density (Mg/m^3)	Moisture Content (%)	Corrected Bulk Density* (Mg/m^3)	Dry Density** (Mg/m^3)
	2	a	2.104	2.4	2.051	2.003
		b	2.089	1.8	2.041	2.005
TTS5	5	a	2.210	2.1	2.125	2.081
		b	2.201	2.1	2.119	2.075
	8	a	2.228	1.9	2.138	2.098
		b	2.187	2.1	2.109	2.066
	2	a	2.177	2.8	2.102	2.045
		b	2.109	2.5	2.055	2.005
TTS6	5	a	2.130	2.6	2.069	2.017
		b	2.056	2.5	2.018	1.969
	8	a	2.257	2.5	2.158	2.105
		b	2.094	2.5	2.044	1.995
	2	a	2.170	2.1	2.097	2.054
		b	2.075	2.1	2.031	1.989
TTS7	5	a	2.207	2.4	2.123	2.073
		b	2.217	2.2	2.130	2.084
	8	a	2.208	2.1	2.124	2.080
		b	2.113	2.2	2.058	2.013
	2	a	2.124	2.4	2.065	2.017
		b	2.032	2.4	2.001	1.954
TTS8	5	a	2.129	2.4	2.069	2.020
		b	2.067	2.2	2.026	1.982
	8	a	2.204	2.7	2.121	2.065
		b	2.175	2.6	2.101	2.048

* Troxler NDG Bulk density values corrected using container method given in BS1377:Part 9:1990
** Dry density calculated using Troxler NDG gauge moisture content

TABLE B3

Wet-mix macadam roadbase
Sections TTS1 - TTS4

Trench ID	Offset from Datum	Test ID	Troxler 3411B Values Bulk Density (Mg/m³)	Moisture Content (%)	Dry Density* (Mg/m³)
	2	a	2.208	3.6	2.131
TTS1	5	a	2.215	3.8	2.134
	8	a	2.194	4.0	2.110
	2	a	2.193	3.2	2.125
TTS2	5	a	2.233	3.6	2.155
	8	a	2.276	4.1	2.186
	2	a	2.063	4.8	1.969
TTS3	5	a	2.348	4.2	2.253
	8	a	2.233	3.8	2.151
	2	a	2.275	5.2	2.163
TTS4	5	a	2.209	4.3	2.118
	8	a	2.313	3.9	2.226

* Dry density calculated using Troxler NDG gauge moisture content

TABLE B4

Wet-mix macadam roadbase
Sections TTS5 - TTS8

Trench ID	Offset from Datum	Test ID	Troxler 3411B Values Bulk Density (Mg/m³)	Moisture Content (%)	Dry Density* (Mg/m³)
	2	a	2.121	4.9	2.022
TTS5	5	a	2.160	4.5	2.067
	8	a	2.150	4.3	2.061
	2	a	2.142	4.0	2.060
TTS6	5	a	2.185	4.3	2.095
	8	a	1.988	4.9	1.895
	2	a	2.095	5.4	1.988
TTS7	5	a	1.997	5.5	1.893
	8	a	2.336	4.6	2.233
	2	a	2.213	4.5	2.118
TTS8	5	a	2.367	3.8	2.280
	8	a	2.213	4.5	2.118

* Dry density calculated using Troxler NDG gauge moisture content

TABLE B5

Summary of bulk density results

Test Sections	Postion (m)	Measured Mean Bulk Density (Mg/m³) Dense Bitumen Macadam B/C	Measured Mean Bulk Density (Mg/m³) Hot Rolled Asphalt W/C
	2	2.335	2.261
TTS1	5	2.298	2.250
	8	2.294	2.246
	2	2.410	2.437
TTS2	5	2.426	2.453
	8	2.268	2.427
	2	2.368	2.468
TTS3	5	2.296	2.460
	8	2.297	2.435
	2	2.391	2.437
TTS4	5	2.279	2.407
	8	2.245	2.465
	2	2.356	2.394
TTS5	5	2.433	2.387
	8	2.333	2.429
	2	2.355	2.406
TTS6	5	2.280	2.442
	8	2.378	2.412
	2	2.373	2.458
TTS7	5	2.387	2.372
	8	2.223	2.396
	2	2.470	2.452
TTS8	5	2.338	2.311
	8	2.306	2.422

APPENDIX C: MATERIALS COMPLIANCE TESTING

TABLE C1

Sample testing - Type 1 granular sub-base
(Tested in accordance with BS812:Part 103:1989)

Sample No.	Sample Date	Supplier	Moisture Content (%)	Comments
1	15/10/92	COLCON Ltd	2.4	In specification.
2	15/10/92	COLCON Ltd	2.2	Not in specification. Graded coarse on 10mm, 5mm, and 600 micron sieves.
3	15/10/92	COLCON Ltd	2.0	Not in specification. Graded coarse on 5mm sieve.
4	15/10/92	COLCON Ltd	2.0	In specification.
5	15/10/92	COLCON Ltd	1.9	Not in specification. Graded coarse on 5mm sieve.

TABLE C2

Sample testing - Wet-Mix Macadam roadbase
(Tested in accordance with BS812:Part 103:1989)

Sample No.	Sample Date	Supplier	Moisture Content (%)	Comments
1	16/10/92	COLCON Ltd	3.7	Not in specification. Graded fine on 75 micron sieve.
2	16/10/92	COLCON Ltd	3.8	Not in specification. Graded fine on 5mm and 75 micron sieves.
3	16/10/92	COLCON Ltd	4.0	In specification.
4	16/10/92	COLCON Ltd	3.0	In specification.
5	16/10/92	COLCON Ltd	3.6	In specification.
6	16/10/92	COLCON Ltd	3.0	Not in specification. Graded coarse on 20mm sieve.
7	16/10/92	COLCON Ltd	3.5	In specification.
8	16/10/92	COLCON Ltd	3.6	In specification.

TABLE C3

Sample testing - 20mm Dense Macadam Basecourse
(Tested in accordance with BS 598:Part 102:1989 Clause 4.2 and BS 2000:Part 49:1983 & Part 58:1988)

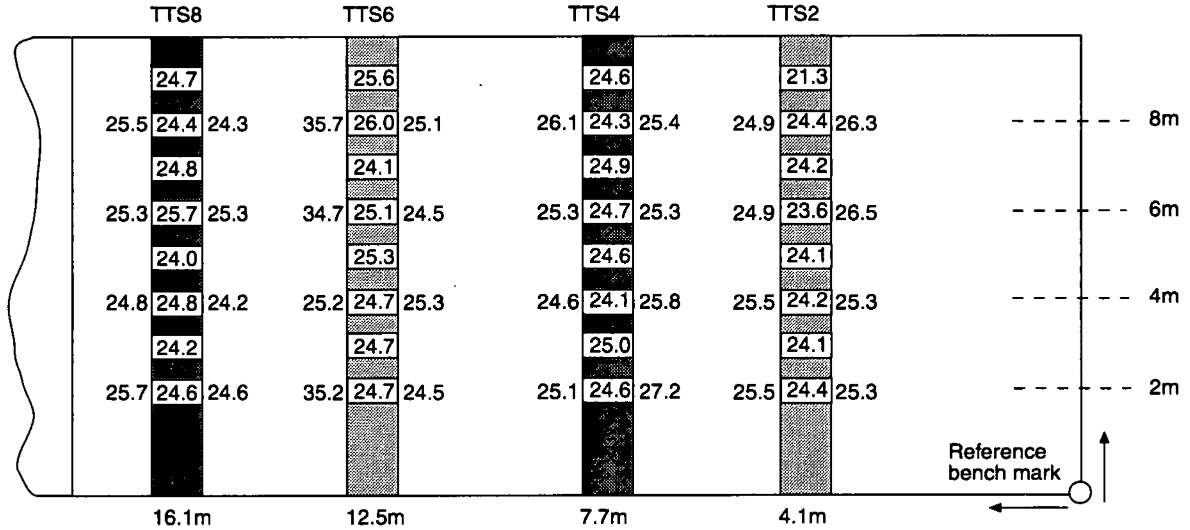
Sample No.	Sample Date	Supplier	Penetration	Softening Point (°C)	Comments
DBM 1	19/10/92	COLCON Ltd	51	49.8	Does not comply with specification. Binder Pen too low.
DBM 2	19/10/92	COLCON Ltd	51	48.6	Does not comply with specification. Binder Pen too low.
DBM 3	19/10/92	COLCON Ltd	58	47.4	Does not comply with specification. Binder Pen too low.
DBM 4	25/01/93	COLCON Ltd	53	49.8	Not in specification. Graded slightly coarse on 14mm sieve. Binder Pen too low.

TABLE C4

Sample testing - 30/14 HRA Wearing Course
(Tested in accordance with BS 598:Part 102:1989 Clause 4.2 and BS 2000:Part 49:1983 & Part 58:1988)

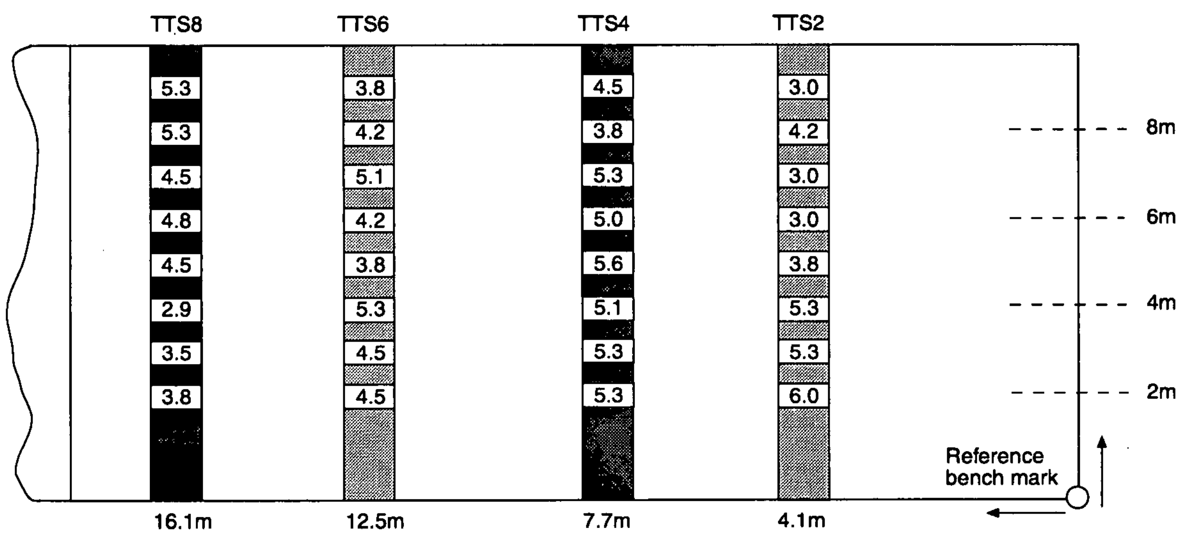
Sample No.	Sample Date	Supplier	Penetration	Softening Point (°C)	Comments
WC 1	20/10/92	COLCON Ltd	34	54.6	In specification.
WC 2	20/10/92	COLCON Ltd	35	53.4	Not in specification. Graded slightly coarse on 75 micron sieve
WC 3	20/10/92	COLCON Ltd	31	53.8	In specification.
WC 4	20/10/92	COLCON Ltd	37	53.0	In specification.
WC 5	25/01/93	COLCON Ltd	38	56.0	In specification.

APPENDIX D: SUBGRADE MOISTURE CONTENTS, CBRS AND SHEAR STRENGTHS



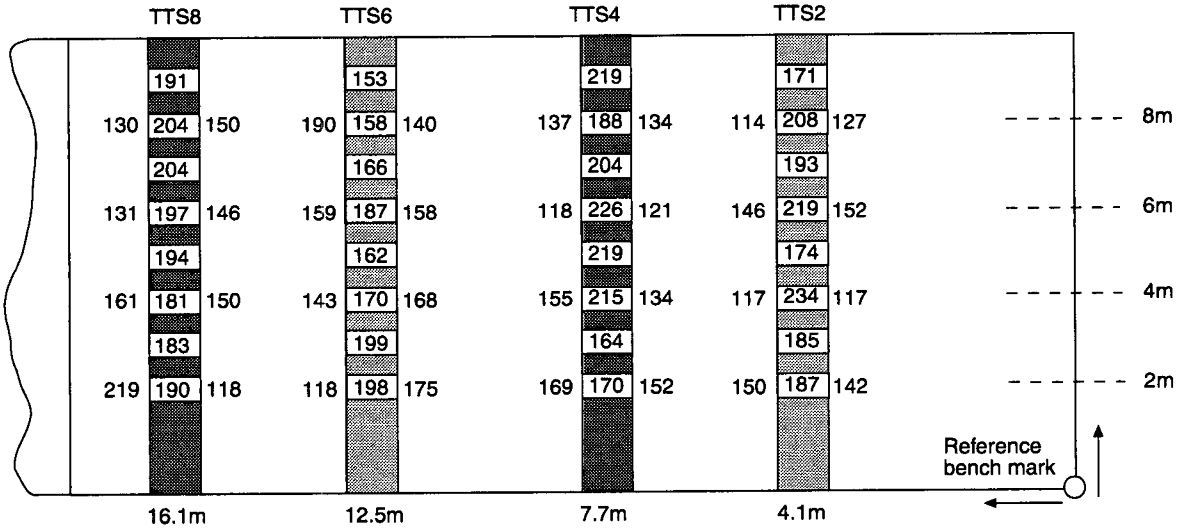
Note: Centre line numbers are percentage moisture contents for the trench bottom. Left and right numbers are percentage moisture contents for the respective side walls.

Fig. D1 Subgrade Moisture Contents (%)



Note: Readings taken at 50mm penetration depth CBR values shown are the corrected penetrometer CBRs. (See text for details).

Fig. D2 Corrected subgrade CBR values (%)



Note: Centre line numbers are shear strength values for the trench bottom. Left and right numbers are shear strength values for the respective side walls.

Fig. D3 Subgrade Shear Strengths Values (kPa)

APPENDIX E: BRITISH GAS PLC COMPACTION RESULTS FOR GRANULAR MATERIALS AND FOAMED CONCRETE

TABLE E1

Trench section TTS8 - Foamed Concrete
Bed - Finefill January 20/21 1993

Position	EXCAVATED BED Depth (mm)	Clegg (IV)*	Cone Pentr.(CBR)# 3"	6"	9"	Bed (1010mm) Clegg (IV)	FINEFILL Depth (mm)	Top Clegg (IV)**	Cone @ 3"
1	1150	3(25)	3	4	-	-	785	11	-
2	1155	3(25)	4	4	-	8	780	10	-
3	1160	4(20)	3	2	4	10	785	9	13+
4	1185	-	-	-	-	-	800	9	13+
5	1190	4(25)	4	4	4	-	810	7	-
6	1180	-	-	-	-	-	800	10	-
7	1190	2(25)	4	4	5	10	810	11	13+
8	1180	-	5	5	3	9	795	8	-
Average	1174	3(24)	4	4	4	9	796	9	13+

Material Placement and Compaction Procedure

1. No compaction of the Excavated Bed
2. Finefill placed to provide pipe bed at 1000mm nominal depth
3. 2 passes on Finefill pipe bed with BS45Y Vibrotamper + 2 extension legs
4. Load cells placed by TRL at 1050mm depth at positions 4 and 6
5. Finefill hand tamped adjacent to and over the load cells
6. Finefill removed underneath collar positions between 3 to 3.5 and 6.5 to 7 as requested by TRL to provide recess and minimise any preloading
7. Finefill placed up to pipe crown and hand tamped between pipe and trench wall
8. Finefill placed above pipe to give a nominal 800mm total depth of cover
9. 4 passes BS45Y Vibrotamper + extension leg
10. Finefill removed by TRL at positions 4 and 6 for load cells at nominally 850mm depth; Finefill hand tamped level

* Hammer indentation in brackets (mm)
\# Hand-held soil assessment cone penetrometer
** Clegg hammer 'bounce' due to resilience of subgrade

TABLE E2

Trench section TTS8 - Foamed Concrete
Foamed Concrete Backfill, Sub-Base & Roadbase January 22 1993
Surfacing January 25 1993

Position	FOAMED CONCRETE Depth to top of layer (mm)	FOAMED CONCRETE Clegg (IV)	BASECOURSE Depth to top of layer (mm)
0.5	126	15	-
1	120	-	65
1.5	126	-	-
2	126	-	60
2.5	125	-	-
3	125	18	56
3.5	130	-	-
4	125	-	61
4.5	126	-	-
5	122	-	56
5.5	124	-	-
6	120	25	60
6.5	123	-	-
7	120	-	56
7.5	117	-	-
8	122	-	56
8.5	122	19	-
Average	124	19	59
Layer Thickness (mm)	672		65

Placement and Compaction Procedure

Foamed Concrete

1. Poured and smoothed level with template to provide nominal 125mm cavity (non standard pouring procedure-transverse)
2. No compaction

Basecourse

1. Thickened Edge sealant (Ayton Asphalte) applied to B/C vertical edge
2. Delivery temperature check 123-147 Deg C
3. Placed to a 'nominal' 65mm compacted thickness to leave 'nominal' 60mm cavity-temperature check
4. Compaction 4 passes BS65Y Vibrotamper, setting 1 to nip-in material
   9 passes BS65Y Vibrotamper, setting 3 full compaction

Wearing Course

1. Tack coat applied on horizontal planed surface only (ie not on B/C)
2. Thickened Edge sealant (Ayton Asphalte) applied to W/C vertical edge
3. Delivery and placement temperature check
4. Compaction Twin-Drum Bomag BW120AD (1040 kg/m) - wider trench width
   Each edge nipped-in, then centre, all without vibration
   5 passes with vibration - full width

TABLE E3

Trench section TTS6 - Granular
Bed - Finefill January 20/21 1993

Position	EXCAVATED BED Depth to top of layer (mm)	Clegg (IV)*	Cone Pentr. (CBR)# 3"	6"	9"	FINEFILL Bed (1010mm) Clegg (IV) Depth (mm)	Clegg (IV)	Top Depth to top of layer (mm)	Clegg** (IV)*	Cone @ 3"
1	1060	3(25)	2	4	4	990	6(10)	780	11(8)	13+
2	1075	2(30)	4	5	5	1000	6(10)	780	10(10)	13+
3	1090	4(25)	5	6	6	985	6	775	12(7)	13+
4	1105	3(30)	5	5	6	985	-	800	13(7)	14
5	1110	2(30)	5	6	6	1005	6(20)	800	10(10)	14+
6	1110	4(25)	3	5	6	980	5	805	13(7)	14/13
7	1130	3(25)	5	6	6	995	5(20)	805	14(6)	14/13
8	1150	3(25)	5	5	3	1000	6(12)	805	12(8)	13/13
Average	1104	3(27)	4	5	5	993	6(14)	794	12(8)	13/13

Material Placement and Compaction Procedure

1. No compaction of the Excavated Bed
2. Finefill placed to provide pipe bed at 1000mm nominal depth
3. 2 passes on Finefill pipe bed with BS45Y Vibrotamper + 2 extension legs
4. Load cells placed by TRL at 1050mm depth at positions 4 and 6
5. Finefill hand tamped adjacent to and over the load cells
6. Finefill removed underneath collar positions between 3 to 3.5 and 6.5 to 7 as requested by TRL to provide recess and minimise any preloading
7. Finefill placed up to pipe crown and hand tamped between pipe and trench wall
8. Finefill placed above pipe to give a nominal 800mm total depth of cover
9. 4 passes BS45Y Vibrotamper + extension leg
10. Finefill removed by TRL at positions 4 and 6 for load cells at nominally 850mm depth; Finefill hand tamped level

* Hammer indentation in brackets (mm)
\# Hand-held soil assessment cone penetrometer
** Clegg hammer 'bounce' due to resilience of sub-grade

TABLE E4

Trench section TTS6 - Granular
Backfill - Roadbase January 20/21 1993

Position	BACKFILL Depth to top of layer (mm)	BACKFILL Clegg (IV)*	Layer 1 Depth to top of layer (mm)	Layer 1 Clegg (IV)*	Layer 2 Depth to top of layer (mm)	Layer 2 Clegg (IV)	Layer 3 Depth to top of layer (mm	Layer 3 Clegg (IV)
1	670	7	470	16	320	20	128	23
2	660	9	465	17	308	19	122	19
3	660	9	465	15	310	19	121	19
4	670	5	475	15	315	19	124	19
5	675	7	480	16	315	-	121	20
6	670	6	480	14	315	13	129	20
7	670	9	480	16	310	18	120	21
8	670	9	455	17	300	19	122	21
Average	668	8	471	16	312	18	123	20
Layer Thickness (mm)	126		197		159		189	

Material Placement and Compaction Procedure

Backfill
1. Excavated material acceptable as Class C (50% cohesive)
2. Backfill placed to provide nominal 130mm layer
3. Compaction Bomag BG100 with 240mm wide roll (setting 3) at 3600kg/m - 6 passes

Granular Sub-Base and Roadbase
1. Imported Type 1 in 3 layers total nominal thickness 570mm amended to 545mm for 125 cavity as agreed.
2. Compaction BG100 setting 3-8 passes per layer

* Clegg hammer 'bounce' due to resilience of sub-grade

TABLE E5

Trench section TTS6 - Granular
Roadbase - Surfacing January 25 1993

Position	ROADBASE Depth to top of layer (mm)	ROADBASE Clegg (IV)	BASECOURSE Depth to top of layer (mm)
0.5	(113)	-	-
1	128	-	54
1.5	125	-	-
2	122	25	45
2.5	124	-	-
3	121	-	49
3.5	126	-	-
4	124	29	59
4.5	121	-	-
5	121	-	52
5.5	124	-	-
6	129	24	54
6.5	121	-	-
7	120	-	49
7.5	121	-	-
8	122	-	49
8.5	130	-	-
Average	124	26	51
Layer Thickness (mm)			73

Placement and Compaction Procedure

Basecourse
1. Thickened Edge sealant (Ayton Asphalte) applied to B/C vertical edge
2. Delivery temperature check 123-147 Deg C
3. Placed to a 'nominal' 65mm compacted thickness to leave 'nominal' 60mm cavity - temperature check
4. Compaction 4 passes BS65Y vibrotamper, setting 1 to nip-in material
   9 passes BS65Y vibrotamper, setting 3 full compaction

Wearing Course
1. Tack coat applied on horizontal planed surface only (ie not on B/C)
2. Thickened Edge sealant (Ayton Asphalte) applied to W/C vertical edge
3. Delivery and placement temperature check
4. Compaction Twin-Drum Bomag BW120AD (1040 kg/m) - wider trench width
   Each edge nipped-in, then centre, all without vibration
   5 passes with vibration - full width

TABLE E6

Trench section TTS4 - Foamed Concrete
Bed - Finefill January 20/21 1993

Position	EXCAVATED BED Depth (mm)	Clegg* (IV)	Cone Pentr. (CBR)# 3"	6"	9"	FINEFILL Depth to top of layer (mm)	Top Clegg** (IV)*	Cone 3"
1	1050	3(20)	3	4	4	802	9(15)	-
2	1060	3(25)	3	4	5	802	10(10)	-
3	1060	3(20)	2	5	6	790	11(8)	-
4	1110	4(25)	3	5	6	795	10(10)	13+
5	1105	2(30)	4	5	6	800	11(8)	-
6	1110	3(25)	4	5	5	802	9(12)	13/13
7	1090	3(30)	4	5	6	802	9(12)	13/13
8	1070	3(25)	3	5	5	800	9(15)	13/13
Average	1082	3(25)	3	5	5	799	10(11)	13/13

Material Placement and Compaction Procedure

1. No compaction of the Excavated bed
2. Finefill placed to provide pipe bed at 1000mm nominal depth
3. 2 passes on Finefill pipe bed with BS45Y Vibrotamper + 2 extension legs
4. Load cells placed by TRL at 1050mm depth at positions 4 and 6
5. Finefill hand tamped adjacent to and over the load cells
6. Finefill removed underneath collar positions between 3 to 3.5 and 6.5 to 7 as requested by TRL to provide recess and minimise any preloading
7. Finefill placed up to pipe crown and hand tamped between pipe and trench wall
8. Finefill placed above pipe to give a nominal 800mm total depth of cover
9. 4 passes BS45Y Vibrotamper + extension leg
10. Finefill removed by TRL at positions 4 and 6 for load cells at nominally 850mm depth; Finefill hand tamped level

* Hammer indentation in brackets (mm)
\# Hand-held soil assessment cone penetrometer
** Clegg hammer 'bounce' due to resilience of sub-grade

TABLE E7

Trench section TTS4 - Foamed Concrete
Foamed Concrete Backfill, Sub-Base & Roadbase January 22 1993
Surfacing January 25 1993

Position	FOAMED CONCRETE Depth to top of layer (mm)	FOAMED CONCRETE Clegg (IV)	BASECOURSE Depth to top of layer (mm)
0.5	130	21	-
1	126	-	56
1.5	128	-	-
2	123	-	60
2.5	126	-	-
3	127	18	62
3.5	126	-	-
4	128	-	65
4.5	124	-	-
5	126	-	65
5.5	126	-	-
6	126	-	62
6.5	127	24	-
7	127	-	69
7.5	130	-	-
8	125	-	57
8.5	127	22	-
Average	127	21	62
Layer Thickness(mm)	672		65

Placement and Compaction Procedure

Foamed Concrete
1. Poured and smoothed level with template to provide nominal 125mm cavity (non standard pouring procedure-transverse)
2. No compaction

Basecourse
1. Thickened Edge sealant (Ayton Asphalte) applied to B/C vertical edge
2. Delivery temperature check 123-147 Deg C
3. Placed to a 'nominal' 65mm compacted thickness to leave 'nominal' 60mm cavity - temperature check
4. Compaction 4 passes BS65Y vibrotamper, setting 1 to nip-in material
   9 passes BS65Y vibrotamper, setting 3 full compaction

Wearing Course
1. Tack coat applied on horizontal planed surface only (ie not on B/C)
2. Thickened Edge sealant (Ayton Asphalte) applied to W/C vertical edge
3. Delivery and placement temperature check
4. Compaction Twin-Drum Bomag BW120AD (1040 kg/m) - wider trench width
   Each edge nipped in, then centre, all without vibration
   5 passes with vibration - full width

TABLE E8

Trench section TTS2 - Granular
Bed - Finefill January 20/21 1993

Position	EXCAVATED BED Depth (mm)	Clegg* (IV)	Cone Pentr. (CBR)# 3"	6"	9"	FINEFILL Depth to top of layer (mm)	Clegg ** (IV)*	Cone 3"
1	1040	2(25)	2	3	4	785	9(15)	13+
2	1040	3(20)	4	5	5	795	9(12)	-
3	1050	3(25)	4	5	6	795	14(8)	13+
4	1050	-	4	5	6	800	10(10)	_
5	1050	3(25)	3	6	5	810	11(10)	14+
6	1050	-	4	6	5	800	10(10)	14/14
7	1055	3(20)	3	5	5	795	9(12)	13+/13+
8	1055	3(25)	2	6	6	800	9(12)	13+/13+
Average	1049	3(23)	3	5	5	798	10(10)	13+/13+

Material Placement and Compaction Procedure

1. No compaction of the Excavated bed
2. Finefill placed to provide pipe bed at 1000mm nominal depth
3. 2 passes on Finefill pipe bed with BS45 Vibrotamper + 2 extension legs
4. Load cells placed by TRL at 1050mm depth at positions 4 and 6
5. Finefill hand tamped adjacent to and over the load cells
6. Finefill removed underneath collar positions between 3 to 3.5 and 6.5 to 7 as requested by TRL to provide recess and minimise any preloading
7. Finefill placed up to pipe crown and hand tamped between pipe and trench wall
8. Finefill placed above pipe to give a nominal 800mm total depth of cover
9. 4 passes BS45Y Vibrotamper + extension leg
10. Finefill removed by TRL at positions 4 and 6 for load cells at nominally 850mm depth; Finefill hand tamped level

* Hammer indentation in brackets (mm)
\# Hand-held soil assessment cone penetrometer
** Clegg hammer 'bounce' due to resilience of sub-grade

TABLE E9

Trench section TTS2 - Granular
Backfill - Roadbase January 20/21 1993

Position	BACKFILL Depth to top of layer (mm)	BACKFILL Clegg (IV)*	Layer 1 Depth to top of layer (mm)	Layer 1 Clegg (IV)*	Layer 2 Depth to top of layer (mm)	Layer 2 Clegg (IV)	Layer 3 Depth to top of layer (mm)	Layer 3 Clegg (IV)
1	665	7	475	16	302	19	(133)	19
2	670	9	460	17	288	17	126	23
3	675	9	460	15	293	19	128	16
4	670	5	470	15	303	17	134	21
5	680	7	460	16	290	17	124	23
6	660	6	465	14	291	18	132	19
7	680	9	460	16	298	19	129	21
8	680	9	460	17	297	19	126	22
Average	673	8	464	16	295	18	128	21
Layer Thickness (mm)	125		209		169		167	

Material Placement and Compaction Procedure

Backfill
1. Excavated material acceptable as Class C (50% cohesive)
2. Backfill placed to provide nominal 130mm layer
3. Compaction Bomag BG100 with 240mm wide roll (setting 3) at 3600kg/m - 6 passes

Granular Sub-Base and Roadbase
1. Imported Type 1 in 3 layers total nominal thickness 570mm amended to 545mm for 125 cavity as agreed.
2. Compaction BG100 setting 3-8 passes per layer

* Clegg hammer 'bounce' due to resilience of sub-grade

TABLE E10

Trench section TTS2 - Granular
Roadbase - Surfacing January 25 1993

Position	ROADBASE Depth to top of layer (mm)	ROADBASE Clegg (IV)	BASECOURSE Depth to top of layer (mm)
0.5	(127)	-	-
1	(133)	27	69
1.5	(133)	-	-
2	126	25	69
2.5	128	-	-
3	128	18	63
3.5	131	-	-
4	134	23	69
4.5	130	-	-
5	124	25	72
5.5	130	-	-
6	132	23	69
6.5	128	-	-
7	129	25	63
7.5	128	-	-
8	126	23	63
8.5	(135)	-	-
Average	129	24	67
Layer Thickness (mm)			62

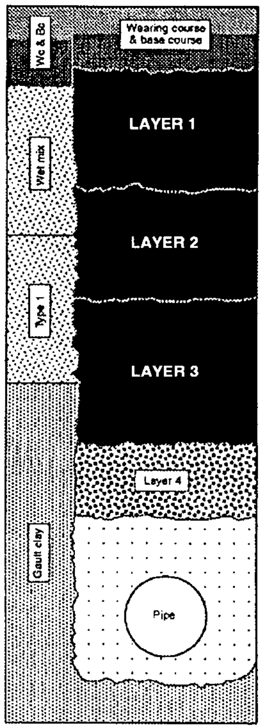
Placement and Compaction Procedure

Basecourse
1. Thickened Edge sealant (Ayton Asphalte) applied to B/C vertical edge
2. Delivery temperature check 123-147 Deg C
3. Placed to a 'nominal' 65mm compacted thickness to leave 'nominal' 60mm cavity - temperature check
4. Compaction 4 passes BS65Y vibrotamper, setting 1 to nip-in material
   9 passes BS65Y vibrotamper, setting 3 full compaction

Wearing Course
1. Tack coat applied on horizontal planed surface only (ie not on B/C)
2. Thickened Edge sealant (Ayton Asphalte) applied to W/C vertical edge
3. Delivery and placement temperature check
4. Compaction Twin-Drum Bomag BW120AD (1040 kg/m) - wider trench width
   Each edge nipped-in, then centre, all without vibration
   5 passes with vibration - full width

FIGURE F1

Compaction data for granular trench TTS2



Layer number	Material	Number of passes	(1) Compacted layer thickness (mm)	Moisture content (%)	(2) Test site	(3) Dry density (Mg/m^3)	Clegg Impact Value	(4) Relative Compaction (%)
					2 m	2.167	24	99
1	Type 1	4+4*	166	3.3	5 m	2.196	26	101
					8 m	2.200	24	101
					2 m	2.036	23	93
2	Type 1	4	171	3.1	5 m	2.093	24	96
					8 m	2.091	27	96
					2 m	2.031	20	93
3	Type 1	8	221	2.9	5 m	2.044	22	94
					8 m	2.100	17	96
					2 m	1.840	11	97
4	Class 4C	6	115	10.0	5 m	1.802	9	95
					8 m	1.833	10	96

Notes:
1. Compacted layer thickness is the average of three measurements taken along the trench.
2. Test site distances correspond to those shown in Figure 4.
3. Dry density results are NDG reading, corrected using the calibration by container method as specified in BS1377: Part 9: 1990.
4. Relative to maximum dry density achieved using BS1377: Part 4: 1990 (the vibrating hammer method).

* = number of passes after extra material added to achieve target layer depth

TABLE F1

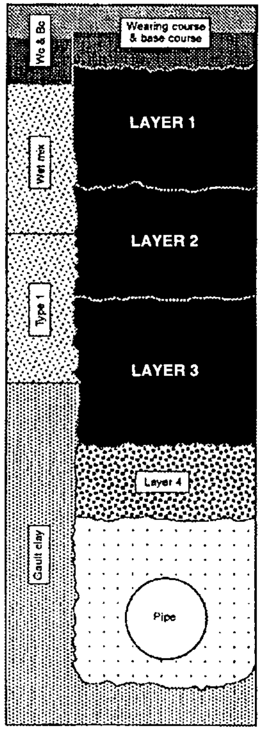
Moisture content and density data
Trench Section TTS2

Trench ID	Layer No	Material	Offset from Datum (m)	Test ID	NDG Bulk Density (Mg/m^{3})	Oven Dry Moisture Content (%)	Corrected Bulk Density* (Mg/m^{3})	Corrected Dry Density (Mg/m^{3})
			2	a	2.221		2.214	2.143
				b	2.265		2.262	2.190
TTS2	1	Type 1	5	a	2.261	3.3	2.258	2.186
				b	2.281		2.279	2.206
			8	a	2.283		2.282	2.209
				b	2.267		2.264	2.192
			2	a	2.144		2.130	2.066
				b	2.087		2.068	2.006
TTS2	2	Type 1	5	a	2.170	3.1	2.159	2.094
				b	2.169		2.157	2.092
			8	a	2.147		2.134	2.070
				b	2.187		2.177	2.112
			2	a	2.097		2.079	2.020
				b	2.116		2.100	2.041
TTS2	3	Type1	5	a	2.135	2.9	2.120	2.060
				b	2.103		2.086	2.027
			8	a	2.173		2.162	2.101
				b	2.170		2.159	2.098
			2	a	2.039		2.013	1.830
				b	2.057		2.034	1.849
TTS2	4	Class C	5	a	2.039	10.0	2.013	1.830
				b	1.985		1.951	1.774
			8	a	2.052		2.028	1.844
				b	2.032		2.005	1.823

*Troxler NDG bulk density values corrected using container method given in BS1377:Part 9:1990

FIGURE F2

Compaction data for granular trench TTS6



Layer number	Material	Number of passes	(1) Compacted layer thickness (mm)	Moisture content (%)	(2) Test site	(3) Dry density (Mg/m^3)	Clegg Impact Value	(4) Relative Compaction (%)
					2 m	2.130	24	98
1	Type 1	5+2*	192	2.8	5 m	2.144	22	98
					8 m	2.158	29	99
					2 m	2.050	22	94
2	Type 1	8	154	2.8	5 m	2.046	24	94
					8 m	na	na	na
					2 m	1.953	27	90
3	Type 1	8	209	3.2	5 m	na	na	na
					8 m	2.044	na	94
					2 m	1.794	9	94
4	Class C	3	128	9.7	5 m	1.811	11	95
					8 m	1.740	10	92

Notes:
1. Compacted layer thickness is the average of three measurements taken along the trench.
2. Test site distances correspond to those shown in Figure 4.
3. Dry density results are NDG reading, corrected using the calibration by container method as specified in BS1377: Part 9: 1990.
4. Relative to maximum dry density achieved using BS1377: Part 4: 1990 (the vibrating hammer method).

* = number of passes after extra material added to achieve target layer depth
na = not available

TABLE F2

Moisture content and density data
Trench Section TTS6

Trench ID	Layer No	Material	Offset from Datum (m)	Test ID	NDG Bulk Density (Mg/m^3)	Oven Dry Moisture Content (%)	Corrected Bulk Density* (Mg/m^3)	Corrected Dry Density (Mg/m^3)
			2	a	2.223		2.227	2.166
				b	2.164		2.152	2.093
TTS6	1	Type 1	5	a	2.214	2.8	2.206	2.146
				b	2.211		2.203	2.143
			8	a	2.248		2.243	2.182
				b	2.203		2.194	2.134
			2	a	2.117		2.101	2.044
				b	2.128		2.113	2.055
TTS6	2	Type 1	5	a	2.127	2.8	2.112	2.054
				b	2.111		2.094	2.037
			8	a	NA		NA	NA
				b	NA		NA	NA
			2	a	2.065		2.044	1.981
				b	2.013		1.988	1.926
TTS6	3	Type1	5	a	NA	3.2	NA	NA
				b	NA		NA	NA
			8	a	2.111		2.094	2.029
				b	2.138		2.124	2.058
			2	a	2.019		1.990	1.814
				b	1.981		1.947	1.775
TTS6	4	Class C	5	a	2.014	9.7	1.985	1.809
				b	2.018		1.989	1.813
			8	a	1.929		1.887	1.720
				b	1.966		1.930	1.759

* Troxler NDG Bulk density values corrected using container method given in BS1377:Part 9:1990
NA = Not available

APPENDIX G: STATIC SOIL PRESSURE RESULTS

TABLE G1

Cross-reference table of gauge and cell positions

Reference	Section TTS2	Section TTS4	Section TTS6	Section TTS8	Figs H13, H14
		Strain Gauges			
a	55	99	22	88	
b	56	100	23	89	
c	57	101	24	90	
d	58	102	25	91	
e	59	103	26	92	
f	60	104	27	93	5
g	61	105	28	94	6
h	62	106	29	95	7
i	63	107	30	96	8
j	64	108	31	97	9
k	65	109	32	98	
		Soil Pressure Cells			
l	40	34	28	22	11
m	39	33	27	21	10
n	42	36	30	24	13
o	41	35	29	23	12
p	44	38	32	26	15
q	43	37	31	25	14

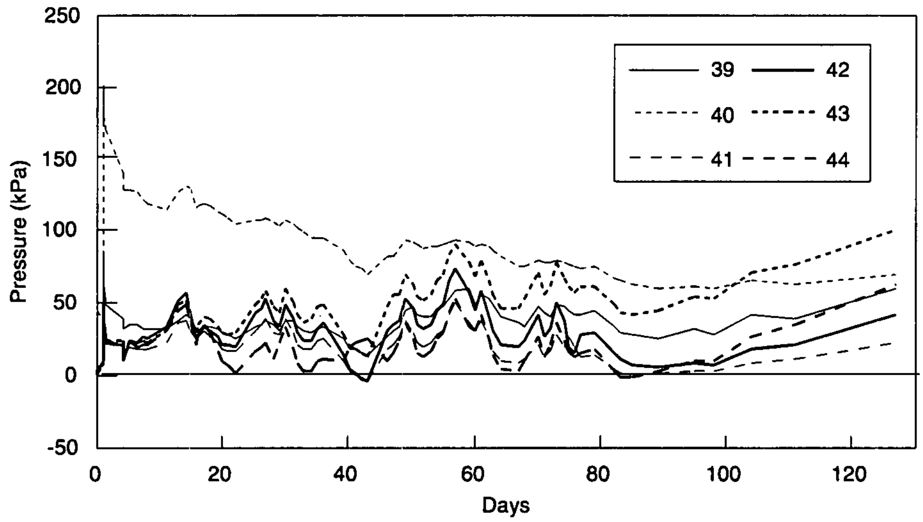


Fig.G1 Section TTS2 - Granular backfill static soil pressure (kPa)

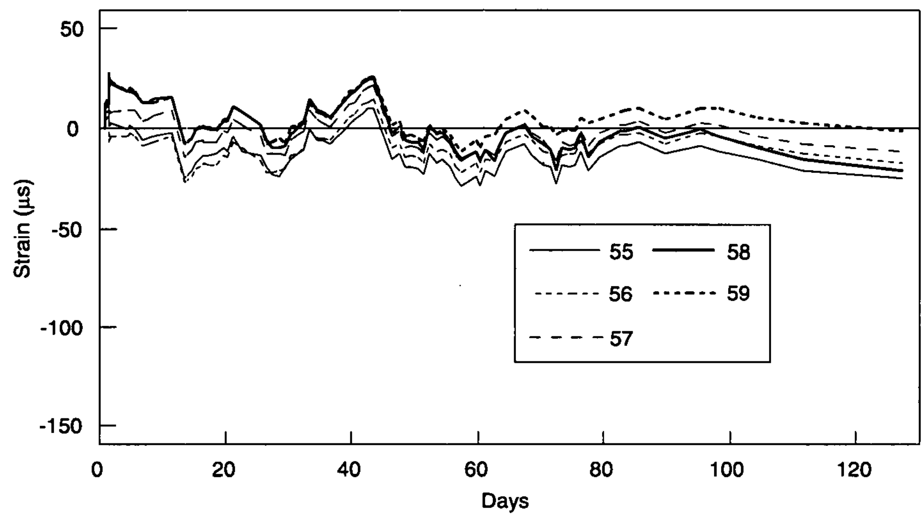


Fig.G2 Section TTS2 - Granular backfill static side pipe strain (+ ve tension)

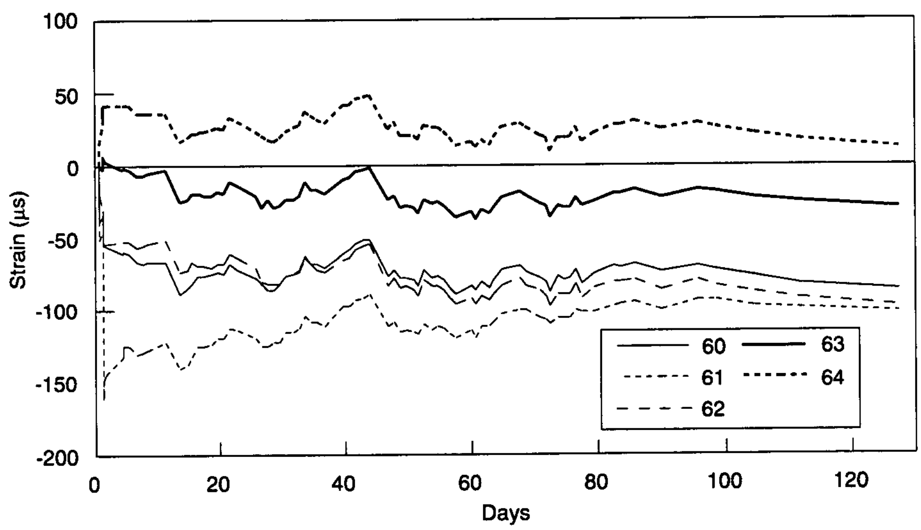


Fig.G3 Section TTS2 - Granular backfill static bottom pipe strain (+ ve tension)

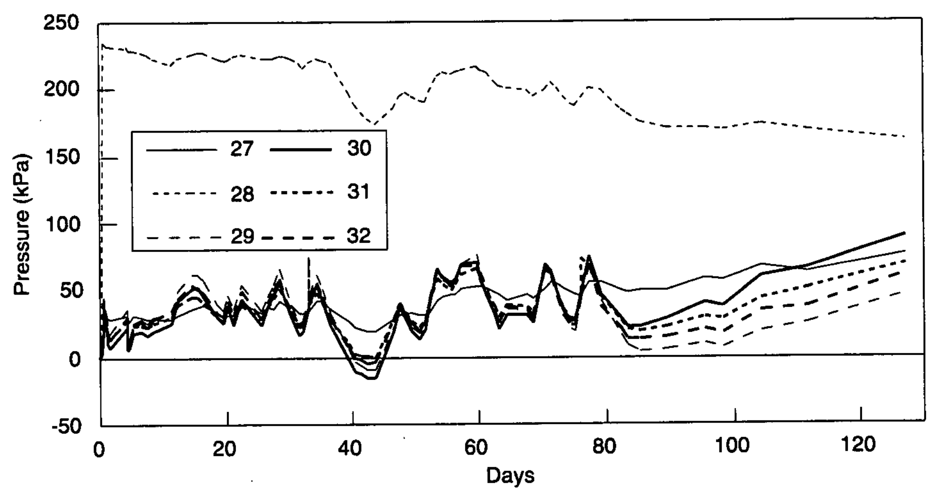


Fig.G4 Section TTS6 - Granular backfill static soil pressure (kPa)

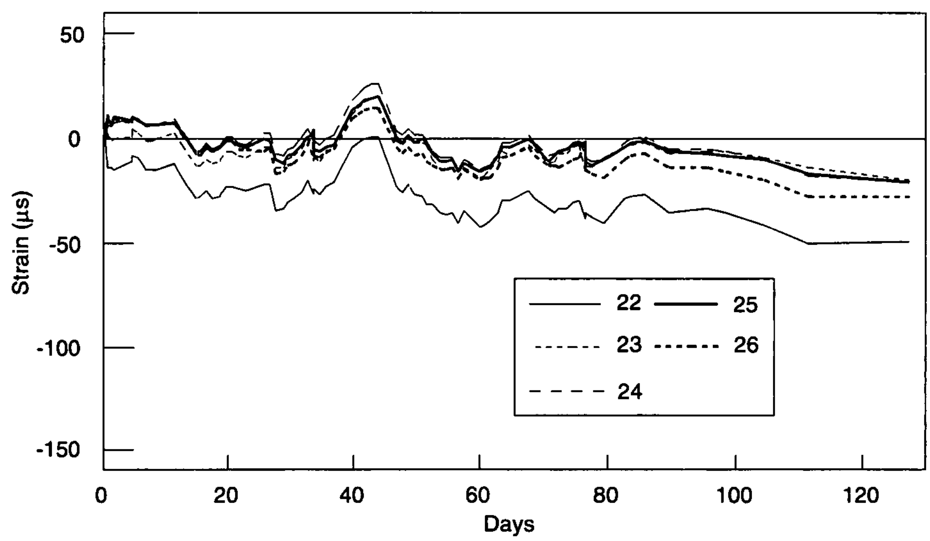


Fig.G5 Section TTS6 - Granular backfill static side pipe strain (+ ve tension)

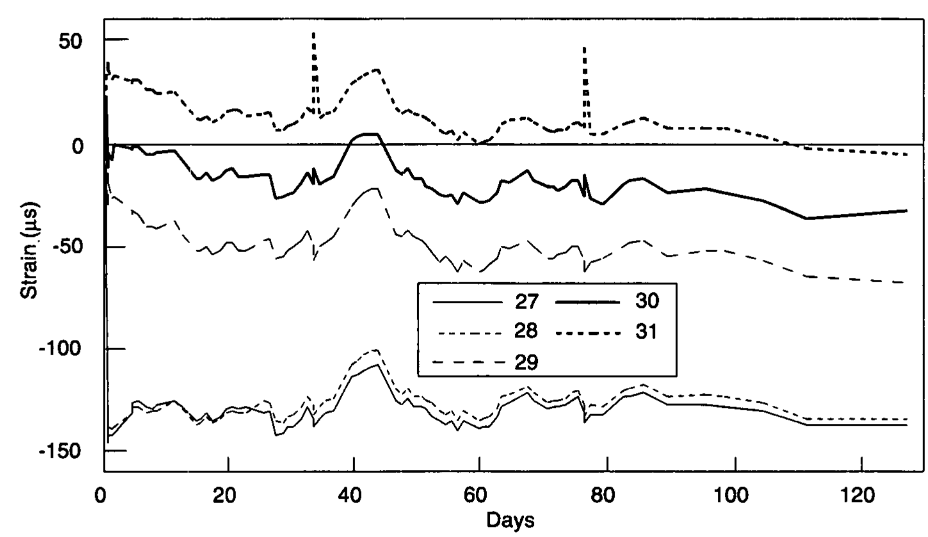


Fig.G6 Section TTS6 - Granular backfill static bottom pipe strain (+ ve tension)

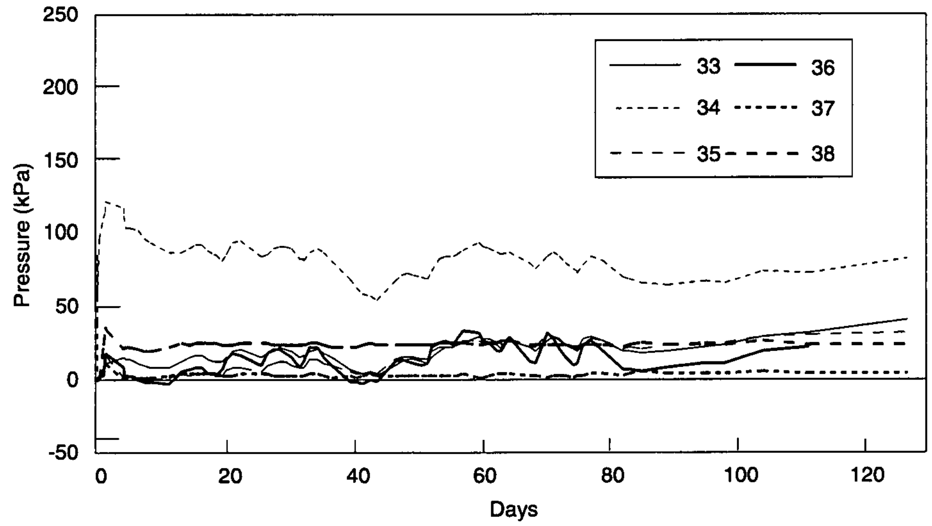


Fig.G7 Section TTS4 - Foamed concrete backfill static soil pressure (kPa)

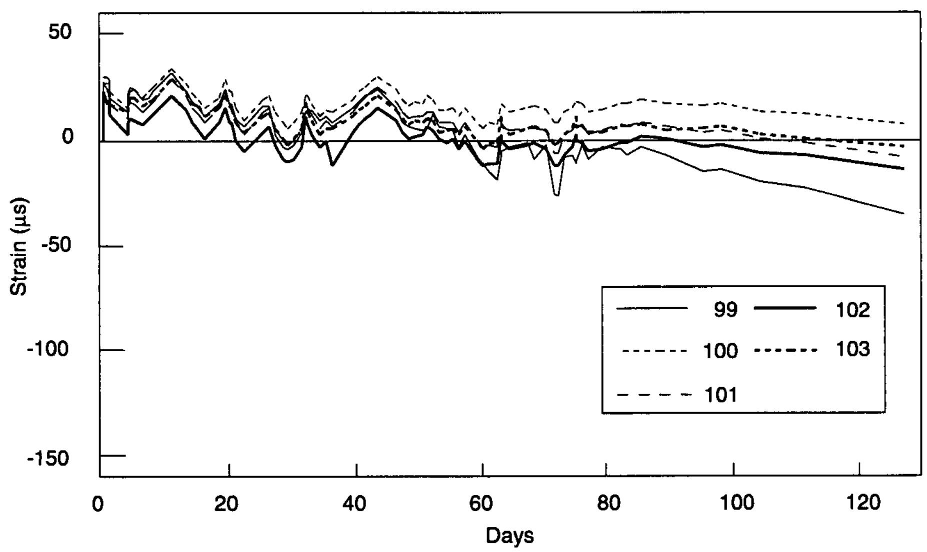


Fig.G8 Section TTS4 - Foamed concrete backfill static side pipe strain (+ve tension)

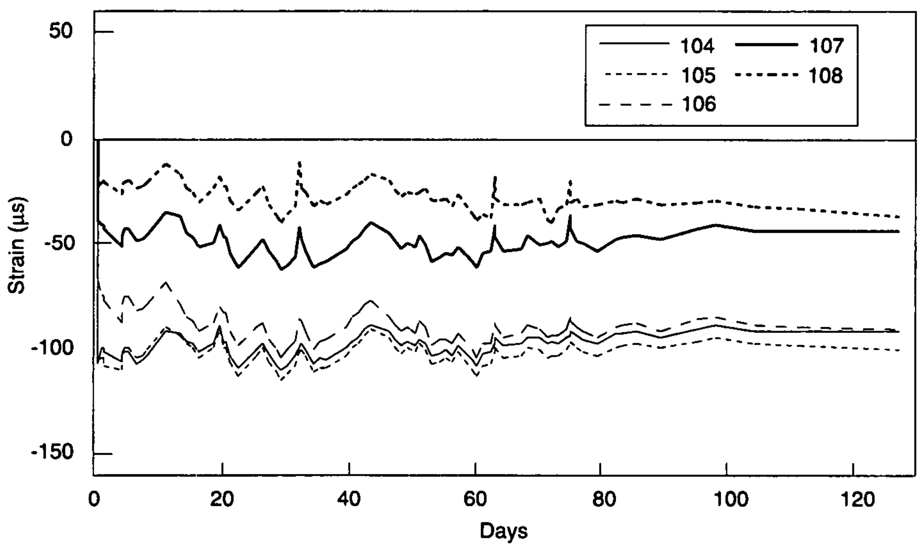


Fig.G9 Section TTS4 - Foamed concrete backfill static bottom pipe strain (+ve tension)

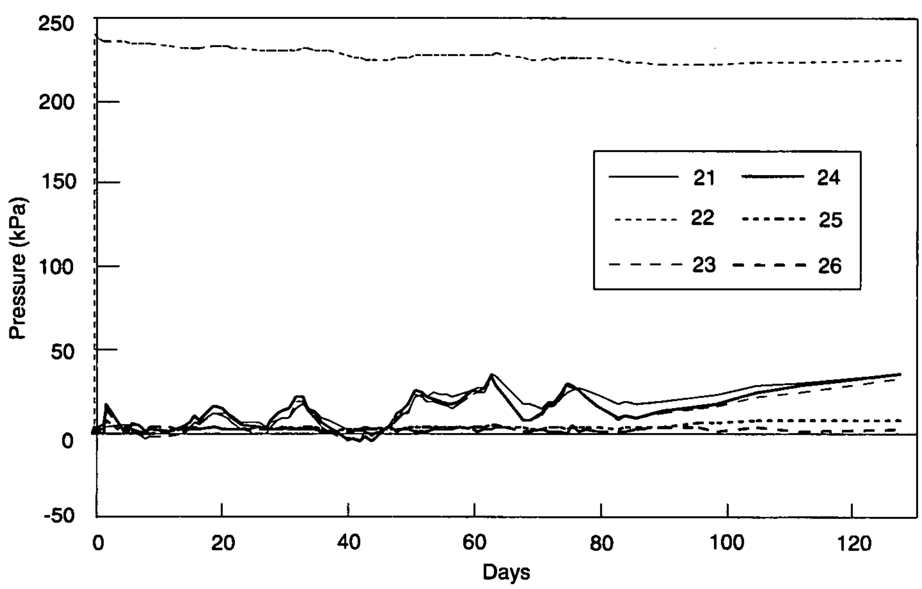


Fig.G10 Section TTS8 - Foamed concrete backfill static soil pressure (kPa)

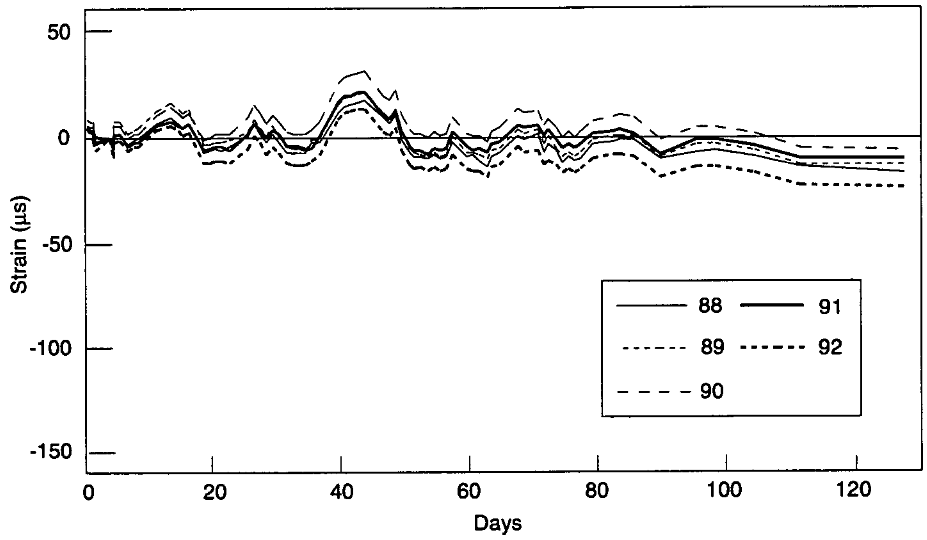


Fig.G11 Section TTS8 - Foamed concrete backfill static side pipe strain (+ve tension)

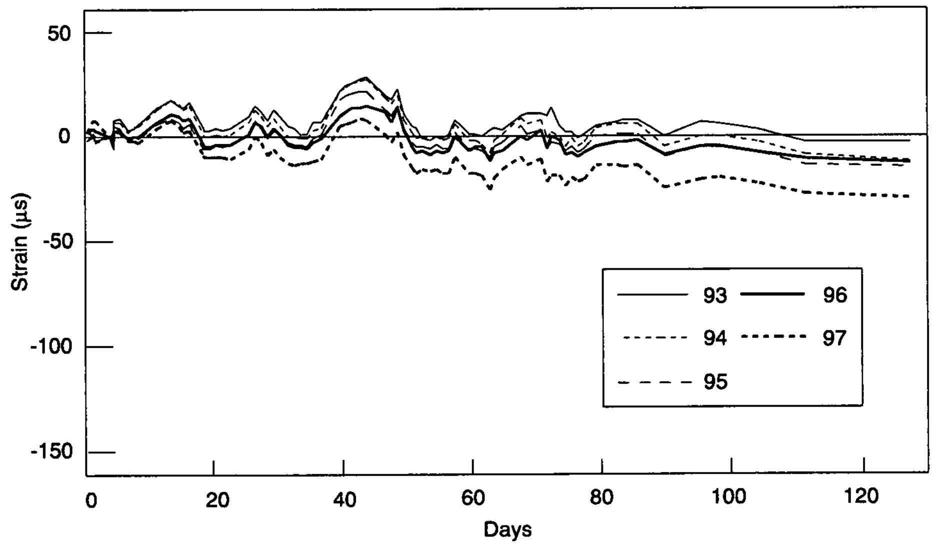


Fig.G12 Section TTS8 - Foamed concrete backfill static bottom pipe strain (+ve tension)

APPENDIX H: DYNAMIC SOIL PRESSURE RESULTS

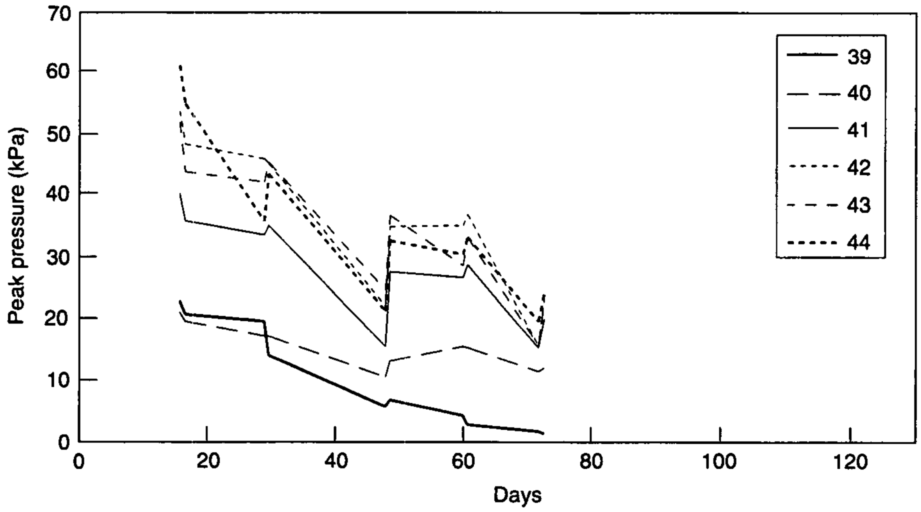


Fig.H1 Section TTS2 - Granular backfill dynamic soil pressure (kPa)

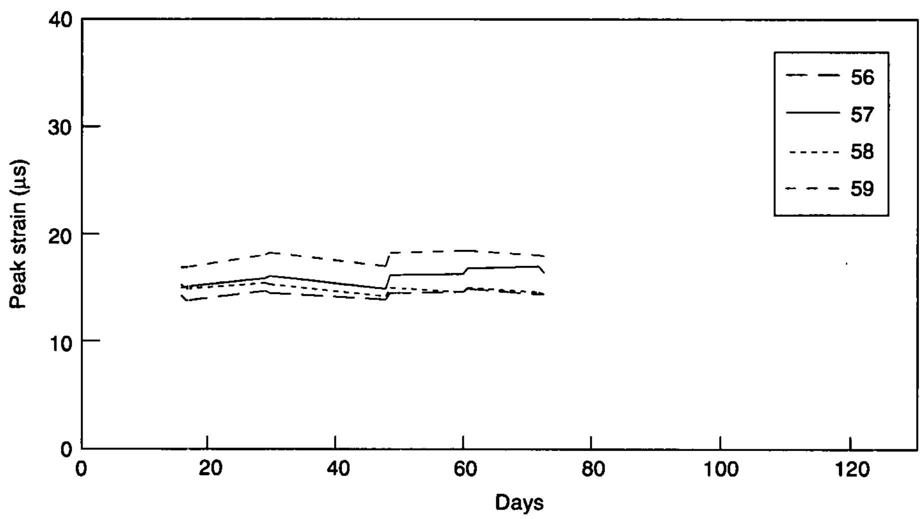


Fig.H2 Section TTS2 - Granular backfill dynamic side strain (+ve tension)

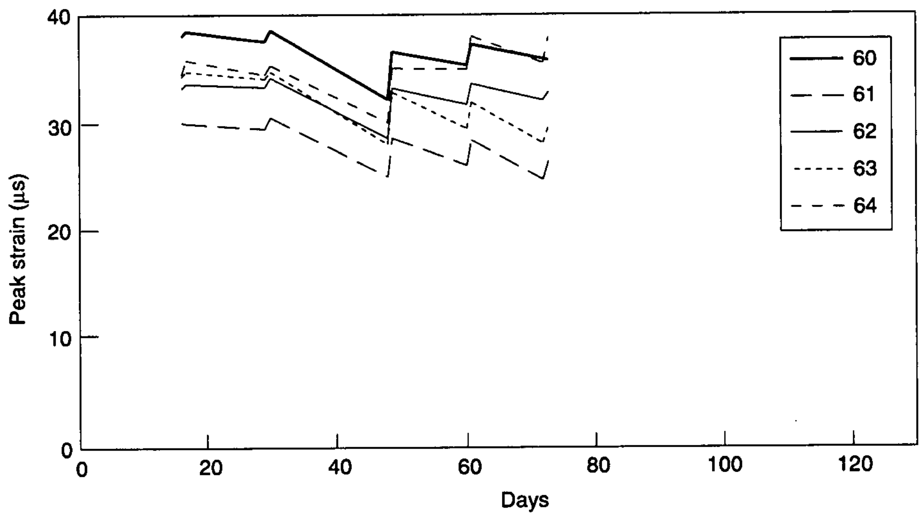


Fig.H3 Section TTS2 - Granular backfill dynamic bottom strain (+ve tension)

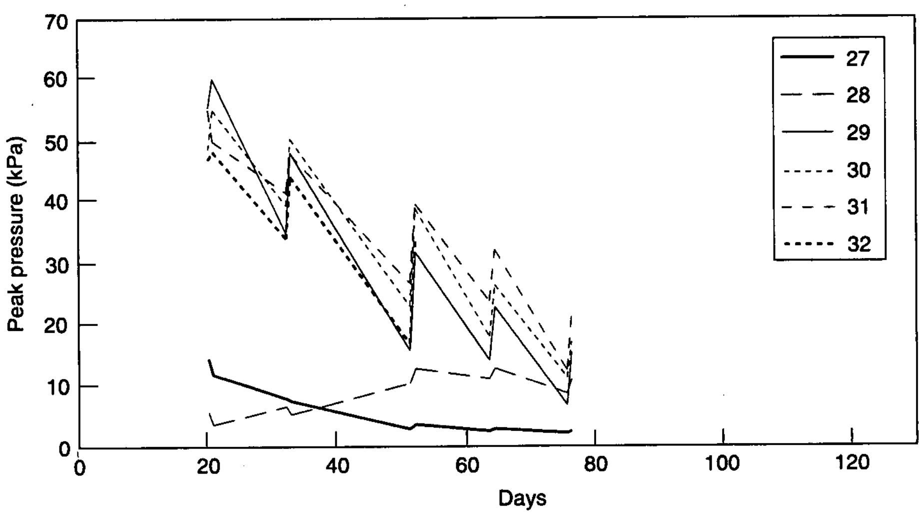


Fig.H4 Section TTS6 - Granular backfill dynamic soil pressure (kPa)

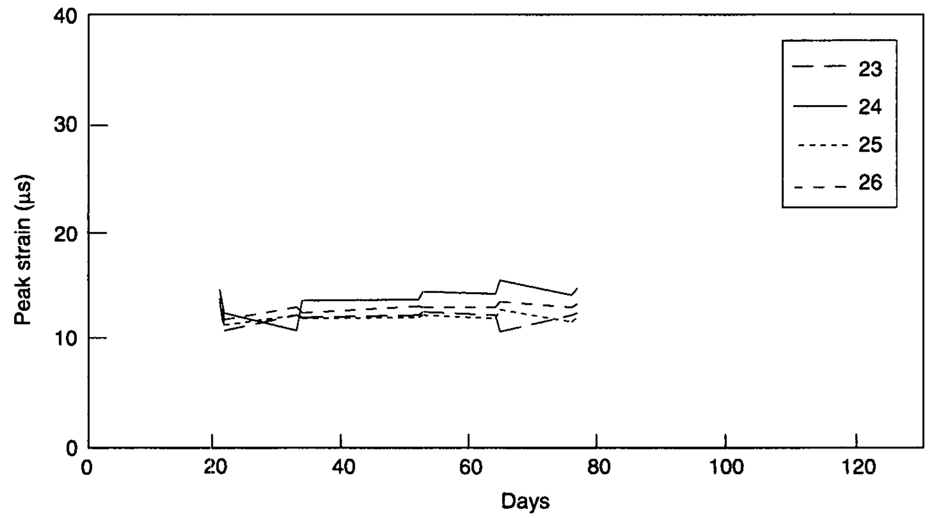


Fig.H5 Section TTS6 - Granular backfill dynamic side pipe strain (+ve tension)

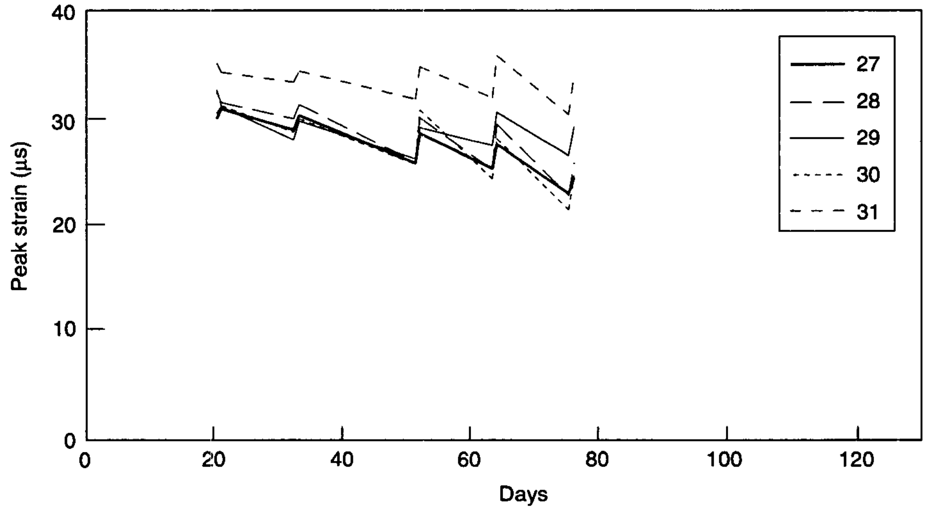


Fig.H6 Section TTS6 - Granular backfill dynamic bottom strain (+ve tension)

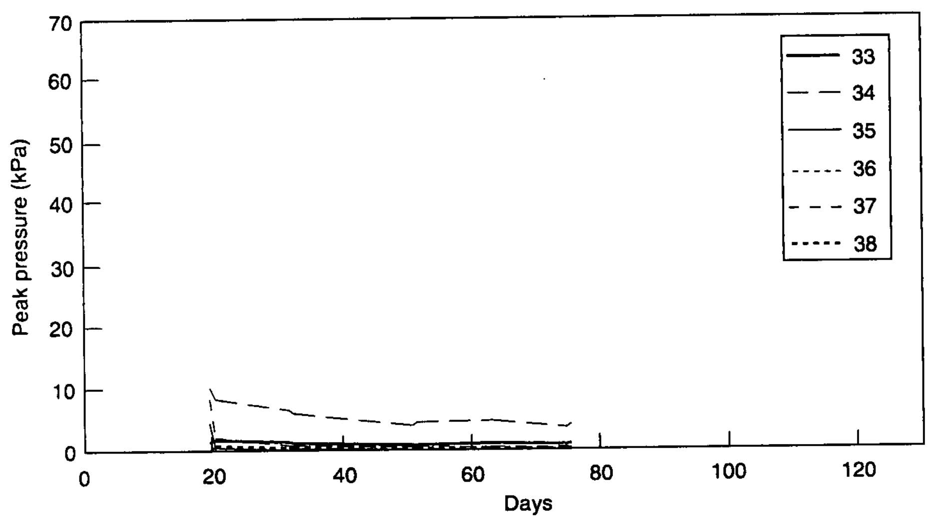


Fig.H7 Section TTS4 - Foamed concrete backfill dynamic soil pressure (kPa)

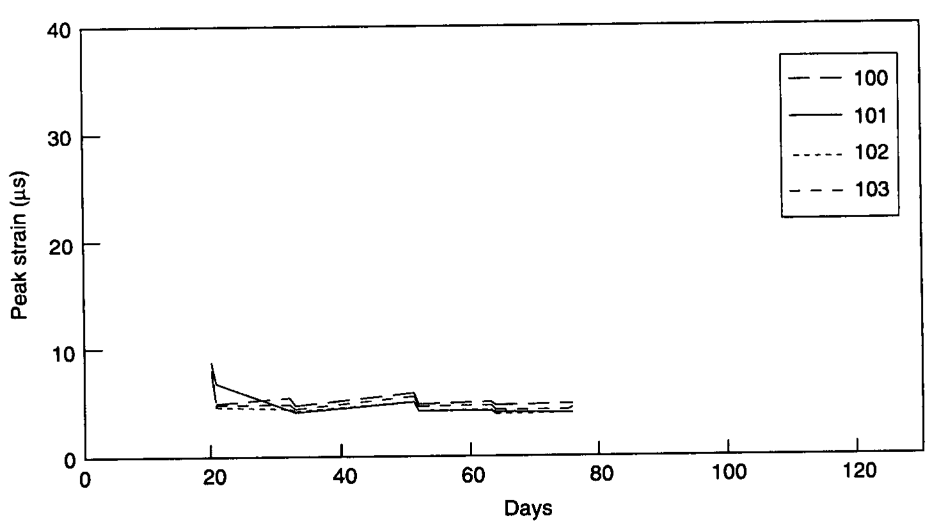


Fig.H8 Section TTS4 - Foamed concrete backfill dynamic side strain (+ve tension)

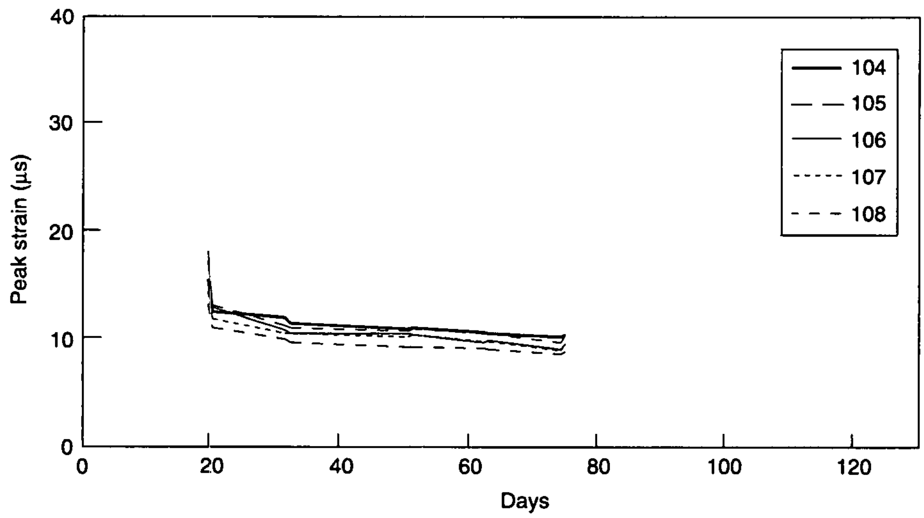


Fig.H9 Section TTS4 - Foamed concrete backfill dynamic bottom strain (+ve tension)

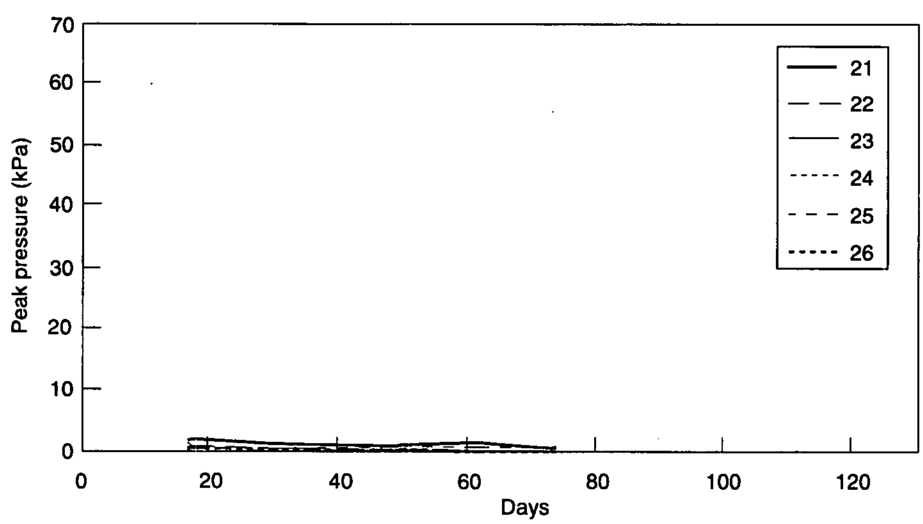


Fig.H10 Section TTS8 - Foamed concrete backfill dynamic soil pressure (kPa)

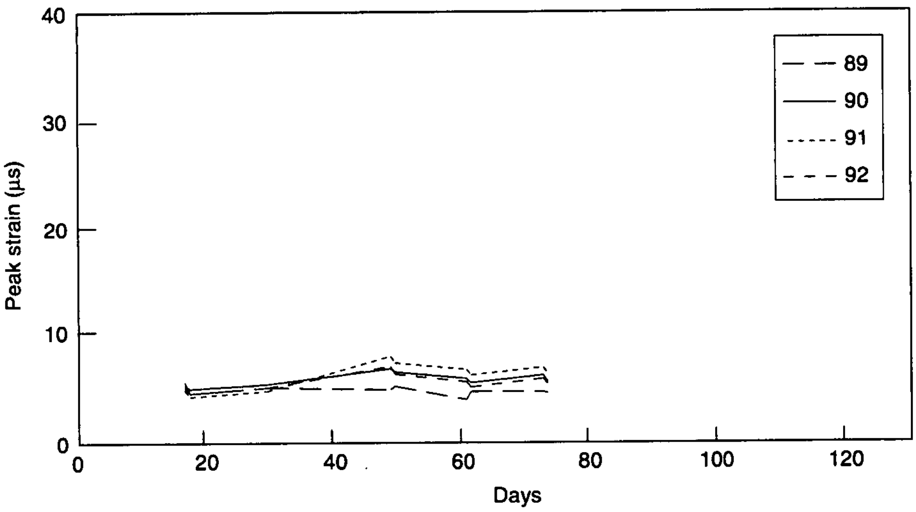


Fig.H11 Section TTS8 - Foamed concrete backfill dynamic side strain (+ve tension)

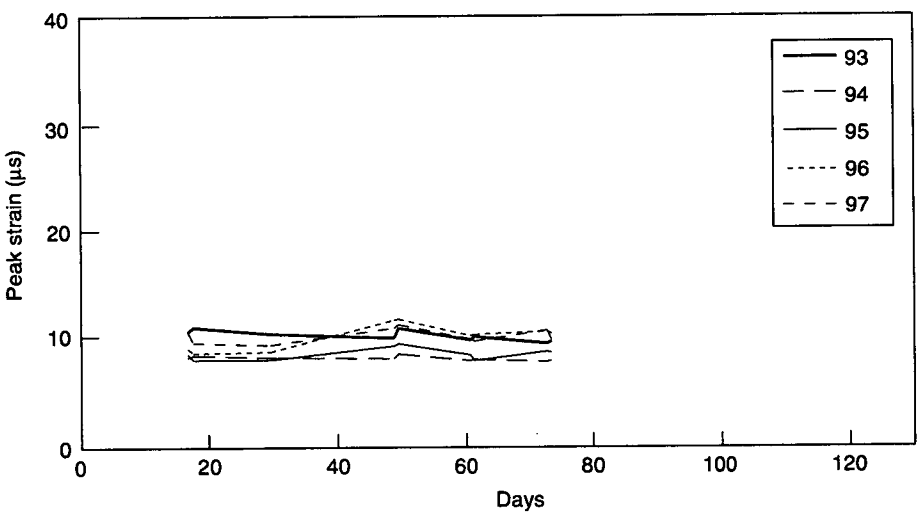
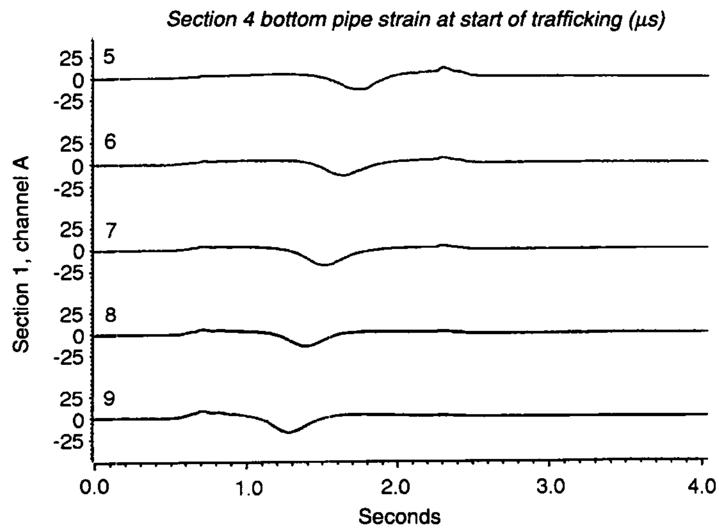


Fig.H12 Section TTS8 - Foamed concrete backfill dynamic bottom strain (+ve tension)



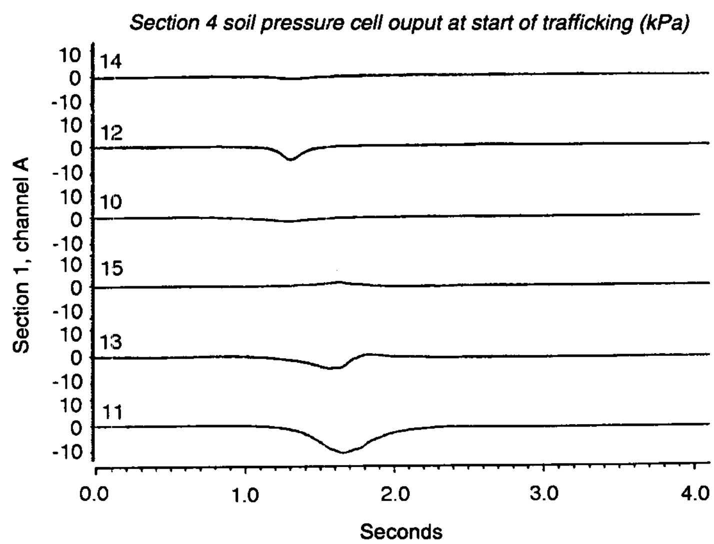
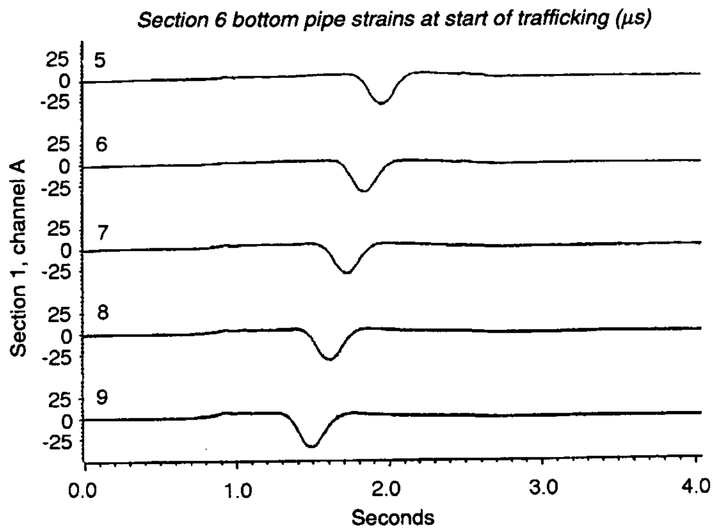


Fig.H13 Dynamic strain and soil pressure traces for foamed concrete



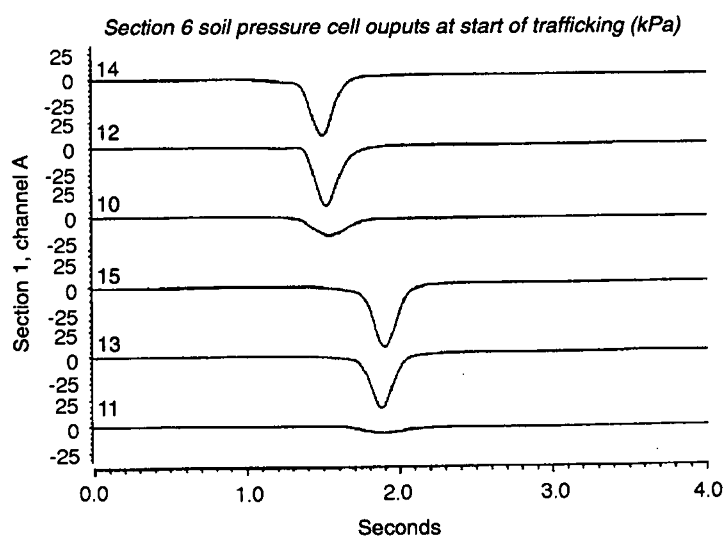


Fig.H14 Dynamic strain and soil pressure traces for granular backfill

APPENDIX I: COMPARISON OF INITIAL AND FINAL SUBGRADE MOISTURE CONTENTS, SHEAR STRENGTHS AND CBRS

TABLE I1

Comparison of the initial and final trench subgrade moisture contents for TTS4 andTTS8

Distance along trench (m)	TTS4 : Foamed concrete						TTS8 : Foamed concrete					
	Initial trench subgrade moisture content (%)			Final trench subgrade moisture content (%)			Initial trench subgrade moisture content (%)			Final trench subgrade moisture content (%)		
	Left sidewall	Trench bottom	Right sidewall	Left sidewall	Trench bottom	Right sidewall	Left sidewall	Trench bottom	Right sidewall	Left sidewall	Trench bottom	Right sidewall
2	25.1	24.6	27.2	24.5	25.8	25.5	25.7	24.6	24.6	24.6	24.9	25.2
3	-	25.0	-	-	26.2	-	-	24.2	-	-	26.0	-
4	24.6	24.1	25.8	24.8	26.0	25.1	24.8	24.8	24.2	25.5	26.8	25.9
5	-	24.6	-	-	26.6	-	-	24.0	-	-	26.7	
6	25.3	24.7	25.3	26.4	25.7	25.9	25.3	25.7	25.2	25.0	26.5	25.7
7	-	24.9	-	-	25.5	-	-	24.8	-	-	25.8	-
8	26.1	24.3	25.4	26.3	25.4	26.4	25.5	24.4	24.3	25.9	26.2	25.4
9	-	24.6	-	-	26.4	-	-	24.7	-	-	26.0	-
Mean	25.3	24.6	25.9	25.5	26.0	25.7	25.3	24.6	24.6	25.2	26.1	25.6

TABLE I2

Comparison of the initial and final trench subgrade vane shear strengths for TTS4 andTTS8

Distance along trench (m)	TTS4 : Foamed concrete						TTS8 : Foamed concrete					
	Initial trench subgrade vane shear strength (kPa)			Final trench subgrade vane shear strength (kPa)			Initial trench subgrade vane shear strength (kPa)			Final trench subgrade vane shear strength (kPa)		
	Left sidewall	Trench bottom	Right sidewall	Left sidewall	Trench bottom	Right sidewall	Left sidewall	Trench bottom	Right sidewall	Left sidewall	Trench bottom	Right sidewall
2	169	170	152	120	204	117	219	190	118	146	139	128
3	-	164	-	-	145	-	-	183	-	-	141	-
4	155	215	134	111	133	137	161	181	150	124	138	136
5	-	219	-	-	145	-	-	194	-	-	154	-
6	118	226	121	131	141	128	131	197	146	172	162	137
7	-	204	-	-	128	-	-	204	-	-	147	-
8	137	188	134	161	172	190	130	204	150	146	204	161
9	-	219	-	-	184	-	-	191	-	-	158	-
Mean	145	201	135	131	156	143	160	193	141	147	155	140

TABLE I3

Comparison of the initial and final trench penetrometer CBRs for TTS4 andTTS8

	TTS4 : Foamed concrete		TTS8 : Foamed concrete	
Distance along trench (m)	Initial trench subgrade penetrometer CBR (%)	Final trench subgrade penetrometer CBR (%)	Initial trench subgrade penetrometer CBR (%)	Final trench subgrade penetrometer CBR (%)
	Trench bottom	Trench bottom	Trench bottom	Trench bottom
2	5.3	4.5	3.8	3.5
3	5.3	3.6	3.5	4.5
4	5.1	3.9	2.9	2.4
5	5.6	3.3	4.5	2.3
6	5.0	3.3	4.8	2.3
7	5.3	3.0	4.5	4.8
8	3.8	5.7	5.3	3.6
9	4.5	4.8	5.3	4.4
Mean	5.0	4.0	4.3	3.5