The use of asphalt arisings as Type 4 sub-base

Prepared for Highways Agency

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Executive Summary

As a result of increasing demand for virgin aggregates, coupled with an escalating environmental awareness, the UK Government is committed to promoting more efficient use of natural resources. Part of this strategy involves developing methodologies for enhancing the effective use of substitutes for virgin aggregates, such as industrial waste, by-products and recycled materials.

In terms of highway construction applications, asphalt arisings were identified as a potential source of recycled materials for sub-base. Asphalt arisings are defined as either asphalt planings derived from the upper pavement layers of bituminous roads or waste bituminous bound aggregate, commonly referred to as granulated asphalt, that has been broken up by specialised plant. TRL Limited (TRL) has previously carried out research into the use of asphalt arisings as both sub-base and capping foundation layers in highway pavement applications. The early research concluded that asphalt arisings could be suitable for use as a sub-base and could give a performance comparable to conventional Type 1 crushed rock aggregates. However, it was recommended that before asphalt arisings were included in the Highways Agency’s Specifications and Standards, the Draft Specification should be tested in full-scale site trials under contractual conditions.

This report is a summary of the research into the use of asphalt arisings in sub-bases carried out by TRL over the past twelve years. In total, seven sub-base trials are reported, encompassing three initial specification development trials undertaken in the early 1990s, two specification validation trials undertaken under contractual conditions on the network in 1998 and 2000, and a small compaction trial undertaken at TRL in 2001. Data from the first full-scale site use of asphalt arisings as Type 4 sub-base in the UK, in 2001, are also reported. Other relevant data and comments, generated from the TRL research into the use of asphalt arisings in sub-base, capping and structural fill applications are also presented and discussed.

The report concludes that Type 4 asphalt arising sub-bases perform in a manner comparable to conventional Type 1 granular sub-bases. The principal conclusions are as follows:

1. Extensive trials have supported the view that a mean deformation of less than 30mm for trafficking to 1000 standard axles is a suitable performance criterion for Type 4 sub-base.
2. In terms of resistance to deformation, the performance of all sub-base materials is not only dependent on the material properties, but also on the characteristics of the formation on which the material is placed.
3. On the basis of TRL experience, typical values for Type 4 sub-base maximum dry density range from 1.860 Mg/m³ to 2.005 Mg/m³ and the corresponding optimum moisture content values range between 5.0% and 8.0%. Typical particle density values for asphalt arisings range between 2.35 Mg/m³ and 2.46 Mg/m³.
4. Type 4 sub-base, compacted in accordance with the Method Specification, may have mean in situ air void contents ranging from 10 to 15%, whilst Type 1 sub-base is expected to have mean in situ air void content of less than 10%. However, the Method Specification for both Type 1 and Type 4 materials has been shown to be appropriate and no major amendments are required, although a 200mm maximum compacted layer thickness is recommended for Type 4 sub-base applications, and the Specification should be amended accordingly.
5. Advice is presented on test procedures. In particular the advice addresses preparation for particle size distribution samples, and the interpretation of optimum moisture content data from the non-standard compaction curves that are frequently generated for Type 4 materials.
6. Further advice considers when asphalt arisings are used in thick layers, typically greater than 750mm. In such circumstances, asphalt arisings may exhibit compressibility problems that could result in large long-term settlements. Where potential settlement concerns are identified, an appropriate testing regime should be used to assess material compressibility characteristics.
7. Laboratory calibrations of the Dynamic Cone Penetrometer (DCP) versus California Bearing Ratio (CBR) can be significantly different for asphalt arisings and conventional granular materials. Accordingly, care must be taken when specifying and interpreting bearing capacity data such as Plate Bearing Test (PBT), CBR and DCP for materials with a visco elastic bituminous component.
1 Introduction

As a result of increasing demand for aggregate and heightened environmental awareness, the Highways Engineering Division of the Department of Transport (now the Highways Agency) sponsored research into the use of industrial waste, by-products and recycled materials as substitutes for virgin aggregate. A potential material identified for highway foundation applications was asphalt arisings, which are defined as either asphalt planings identified for highway foundation applications was asphalt arisings, which are defined as either asphalt planings derived from the upper pavement layers of bituminous roads, or waste bituminous bound aggregate that has been broken up by specialised plant and is often referred to as granulated asphalt.

During road construction, the road foundation is required to act as a haul road for traffic delivering foundation and roadbase material and only limited rutting in the wheelpaths is permitted. The foundation must also provide a firm substrate for good compaction of the roadbase and offer good support to the pavement throughout its life, to limit the tensile strains in the bound layer. To fulfil this function, the sub-base must form a stable layer that does not deform excessively within itself under construction traffic or a loaded paving machine. It must also have an adequate stiffness to reduce traffic stresses on the underlying weaker materials to magnitudes they can sustain. Additionally, all sub-base material within the top 450mm of the road surface must not be susceptible to frost.

Road foundation designs specified by the Department of Transport in 1987 included sub-bases of unbound granular materials, laid on soil subgrades. When the soil is weak, a capping layer of unbound granular material, or stabilised soil, is compacted on the subgrade before laying the sub-base. On heavily trafficked roads, only Type 1 sub-bases were specified and permitted materials encompassed crushed rock, crushed slag, crushed concrete or well burnt non-plastic shale. Although not included in the 1987 Specification, asphalt arisings were identified as a potential source of sub-base material.

As part of the initial development of a formal specification for asphalt arisings, TRL Limited (TRL) undertook three full-scale research trials of asphalt arisings as sub-base in the early 1990s. Trials were carried out at Hayes in Middlesex, on the A414 Cole Green Bypass in Hertfordshire and on the A74, near Gretna in Scotland. Using data and experience gained from these trials, and from other research investigating capping and fill applications, a Draft Specification for the use of asphalt arisings as sub-base was subsequently prepared. The Draft Specification was then tested in two full-scale site trials, under contractual conditions, on the A14 at Milton and the A27 at Arundel. Following these two specification validation trials, an additional compaction trial was carried out at TRL in 2001. This small trial was undertaken to confirm that appropriate states of compaction were being achieved, and to assist in the fine-tuning of Specification clauses. In the latter part of 2001, asphalt arisings were used extensively as Type 4 on the A2/M2 Cobham widening works; details of the Type 4 sub-base trial undertaken as part of the scheme are also described in this report.

The use of asphalt arisings is now firmly established in both the Series 600 (Earthworks) and Series 800 (Road pavements - unbound materials) of the Specification for Highway Works (SHW, MCHW 1). It is worth noting a minor difference in terminology; Series 600 refers to the use of bituminous planings as Class 6F3 capping, whilst Series 800 refers to the use of asphalt arisings as Type 4 sub-base. In terms of suitability and acceptability criteria, capping and sub-base requirements differ only marginally.

2 Specification development trials

Early research into the use of asphalt arisings involved the construction of trial foundations at three sites. The first Specification development trial was undertaken at Hayes in Middlesex, and was reported by Chaddock and Earland (1992). The second development trial, on the A414 Cole Green Bypass in Hertfordshire, was reported by Chaddock and Coyle, (1994). The third trial was undertaken on the A74 in Scotland near Gretna, and was reported by Coyle et al. (1995). In these trials, two material types were considered; granulated asphalt from a single source and asphalt arisings planed from three separate sources. These three Specification development trials covered a wide range of foundation strengths and environmental conditions, with the granulated asphalt being used in a strong foundation, whereas the planings were incorporated into much weaker foundations.

Full-scale installation, compaction and trafficking trials were undertaken at each site. In the absence of a dedicated Specification for asphalt arisings, the cold asphalt arisings were treated identically to control Type 1 materials. At each trial site, the asphalt arisings and the control Type 1 materials were placed using the same compactive effort. Each of the development trials is described in further detail in the following sections.

2.1 Hayes, Middlesex

The trial at Hayes, Middlesex took place between October and November 1991, and was located on a 50m long section of a two lane temporary road, associated with the construction of the Hayes bypass. The trial comprised granulated asphalt sub-base and Type 1 crushed rock sub-base, laid on a gravel capping (see Figure 2.1). The capping layer consisted of a 350mm thick layer of as-dug sand and gravel. The particle size distribution of the gravel was broadly within the limits for Type 6F1 capping specified in Series 600 of the SHW (MCHW 1).

2.1.1 Formation measurements

FWD tests

Previous studies (Chaddock and Blackman, 1989) suggested that deformation of foundations by traffic is related to stiffness, for foundations built of similar materials. Accordingly, the consistency of the formation along the site was evaluated using a Falling Weight Deflectometer (FWD). Prior to placing the sub-base material, formation stiffness at the top of the capping layer was measured and shown to be reasonably constant for just
over half the trial length. However, beyond chainage 27.5m, the formation stiffness decreased significantly. In order to compare the performance of the two sub-base materials, two trial sections with comparable formation stiﬀnesses, referred to as equivalent regions, were therefore selected (see Figure 2.1).

**DCP tests**
The top 250mm of capping was assessed using a Dynamic Cone Penetrometer (DCP) passing through the compacted sub-base. As the performance of both sub-bases was to be assessed by traﬃcking, DCP tests were carried out in the wheel-paths of a lorry prior to and on completion of traﬃcking. The substrate in the equivalent region below the granulated asphalt was found to be slightly stronger than that supporting the Type 1 crushed rock sub-base. The CBR of the capping, derived from the DCP, was an average of 30% and 40% under the granulated asphalt and Type 1 sub-base regions respectively, both before and after traﬃcking.

**2.1.2 Sub-base materials**
The granulated asphalt used in the trial at Hayes comprised bituminous bound materials recycled from roads under reconstruction and surplus material from an asphalt plant. These materials were combined, crushed and graded in mixing plant and the resulting granulated material stockpiled prior to use. The granulated asphalt supplied to site (by Talbot Aggregates, West Drayton) included occasional foreign matter. The control Type 1 sub-base material was crushed limestone.

**2.1.3 Particle size distribution of sub-base materials**
The particle size distribution of the Type 1 material was determined following the method speciﬁed in British Standard BS 812 (BSI, 1985), i.e. washing and decanting followed by oven drying and dry sieving. The Type 1 material was within the Type 1 grading envelope speciﬁed by the SHW (MCHW 1).

As oven drying after washing would have softened the bitumen and allowed particles to adhere to each other, which in turn would have resulted in an inaccurate grading, samples of the granulated asphalt were dry sieved. When treated in this manner, the granulated asphalt did not comply with the Type 1 grading (see Figure 2.2), the main deviations being either excessively coarse or insuﬃcient ﬁne material. One reason for the presence of excessively coarse material could have been adhesion between large particles. As ﬁne materials also tend to adhere to larger particles, they may therefore not have been recorded by the dry sieving method used. It is possible that the washing and sieving method used for the Type 1 may have resulted in greater compliance of the granulated asphalt.

**2.1.4 Construction of sub-bases**
A vibratory roller was used to compact both sub-bases. Following compaction, the *in situ* mean dry density of the Type 1 sub-base was 92% of the peak value (i.e. relative compaction), determined from the compaction test speciﬁed in the relevant British Standard (BS 5835, BSI, 1980). Only two-thirds of the number of passes of the roller required by the Speciﬁcation were applied. The *in situ* density of the granulated asphalt sub-base was not evaluated, as techniques had not yet been developed for carrying out rapid *in situ* density tests and laboratory compatibility tests on unbound granular materials containing bitumen. The thickness of the compacted sub-bases was speciﬁed as 150mm but the mean thickness after construction was found to be 140mm for the granulated asphalt and 116mm for the Type 1.

**2.1.5 Trafficking of sub-bases**
The foundations were traﬃcked by a rigid four-axle lorry, of total mass 10.8 tonnes, ﬁtted with dual wheels on the rear two axles. The axle loads of the lorry were converted into an equivalent number of standard 80kN axles using the fourth power law (Liddle, 1962).

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**Figure 2.1** Layout of Hayes trial
The lorry was driven down the middle of the trial foundations and care was taken to minimise lateral variation in the wheel-paths from pass to pass. At intervals, trafficking was stopped and the height of the sub-base surface was measured at fixed locations to allow calculation of the deformations. During trafficking, the lorry was initially driven in a circuit, traversing the trial area in one direction. After 400 standard axles, the lorry was driven forwards as before but then reversed over the trial sections to increase the rate of trafficking. This change in the nature of trafficking appears to have increased the rate of deformation of the Type 1 crushed rock section. The increase in mean deformation of the foundations with cumulative traffic is shown in Figure 2.3.

The foundation deformations after 1000 standard axles were predicted to be 13mm for the Type 1 crushed rock section and 16mm for the granulated asphalt section. As a comparison, Powell et al. (1984) recommended a maximum sub-base deformation of 40mm under this trafficking.

**Figure 2.2** Hayes trial: Granulated asphalt particle size distribution

**Figure 2.3** Hayes trial: Deformation of trial sections
2.1.6 Summary of Hayes trial

Measurements from the two sub-bases in the Hayes trial are summarised in Table 2.1. Although the granulated asphalt sub-base was thicker than the Type 1 sub-base, deformation of the granulated asphalt section was slightly more than that of the Type 1 section. Notwithstanding, the trial demonstrated that on a stiff substrate, granulated asphalt could perform as well as a traditional Type 1 sub-base.

Table 2.1 Hayes trial: Foundation parameters and performance

<table>
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<tr>
<th></th>
<th>Type 1 crushed rock</th>
<th>Granulated asphalt</th>
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<tbody>
<tr>
<td>Substrate stiffness (MPa)</td>
<td>166</td>
<td>155</td>
</tr>
<tr>
<td>Sub-base thickness (mm)</td>
<td>116</td>
<td>140</td>
</tr>
<tr>
<td>Sub-base CBR (DCP) before trafficking (%)</td>
<td>14</td>
<td>44</td>
</tr>
<tr>
<td>Sub-base CBR (DCP) after trafficking (%)</td>
<td>23</td>
<td>76</td>
</tr>
<tr>
<td>Deformation after 200 lorry passes (mm)</td>
<td>11</td>
<td>14</td>
</tr>
<tr>
<td>Predicted deformation after 1000 standard axles (mm)</td>
<td>13</td>
<td>16</td>
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The research from the Hayes trial concluded that:
- on a stiff substrate, granulated asphalt could perform as well as a traditional Type 1 sub-base;
- further work was needed to develop in situ tests that rank granulated asphalt and Type 1 crushed rock in the same order as did the trafficking trial;
- producers needed to establish quality control procedures to ensure the consistency of granulated asphalt;
- the Specification for unbound aggregates, and the dependent tests, needed to be adapted for materials containing bitumen.

2.2 A414 Cole Green Bypass

This trial at the A414 Cole Green Bypass was carried out in December 1993. The location of the trial was on an 80m long section that comprised part of the westbound carriageway of the new A414 two lane, dual carriageway Cole Green Bypass. In the region of the trial, the finished road was in a cutting of 4m depth. The trial comprised a section of asphalt arisings re-used as unbound sub-base and a section of Type 1 crushed rock sub-base. The design required that the sub-bases were laid to a nominal thickness of 150mm on capping of 350mm nominal thickness.

The subgrade in the region of the trial was a glacial till of silty clay, with sand pockets and occasional gravel. The subgrade strength was very variable, with areas having CBRs of 5% or less, hence the requirement of capping in the design. The capping was an as-dug clayey sand and gravel, obtained from the construction site, and had a liquid limit of 84%, a plastic limit of 30% and a plasticity index of 54%. The particle size distribution of the capping complied with the specification limits of Class 6F1 capping (Series 600, MCHW 1), with between 8 and 14% of material passed the 0.063mm sieve. As a consequence of this significant fines content and associated susceptibility to moisture, the rain that fell prior to laying the sub-base resulted in softening of the top of the prepared capping.

2.2.1 Measurements on the formation

The layout of the A414 Cole Green trial is shown in Figure 2.4. Prior to placing the sub-bases, the consistency of the formation (capping) stiffness along the site was assessed using a FWD. The formation stiffness was shown to be reasonably constant between chainage 4600m and chainage 4560m. To permit compaction on adjacent equivalent regions, the boundary between the two sub-base types was selected at chainage 4580m. The mean formation stiffness was 169MPa in the planings section and 142MPa in the Type 1 section.

2.2.2 Sub-base materials

The planings used at the A414 site were sourced from the planing operations on the northbound carriageway of the...
A1(M), between Junctions 6 and 7. The material, supplied by Redlands Aggregates Ltd., was taken from a continuously graded dense roadbase, a gap graded basecourse and a gap graded wearing course. The control Type 1 sub-base, comprising crushed granite, was supplied by Redlands Aggregates from Mountsorrel in Leicestershire.

2.2.3 Particle size distribution of sub-base materials
The particle size distribution of the Type 1 sub-base was determined according to BS 812 (BSI, 1985) and complied with the specification requirements in Series 800 (MCHW 1).

For the planings sub-base, to reduce the tendency of the bitumen to soften, oven drying was carried out at a temperature of 50°C instead of the specified 105°C. Despite this precaution, the planings adhered slightly but the material was crumbled by hand before sieving. The particle size distribution of the planings was determined before compaction and again after the trafficking trial. As shown in Figure 2.5, the planings did not comply with the Type 1 specification limits; the average grading of the planings was coarse of the lower boundary of the Type 1 grading envelope for particles finer than about 10mm.

The particle size distribution of the aggregate recovered from the planings after extraction of the bitumen was also determined (see Figure 2.6) and showed that the particle size distribution of the recovered aggregate was much finer than that measured for the unprocessed planings. This confirmed that the individual particles of the planings were aggregations of finer particles bound with bitumen.

2.2.4 Compaction of sub-bases
A vibratory roller was used to compact both sub-bases and the minimum number of passes of the roller required in the Specification (Series 800, MCHW 1) was exceeded by one third. To compact any loose material on top of the sub-bases, a final pass of the roller was applied without vibration. Whilst the compacted thickness of the sub-bases was specified as 150mm, the average compacted thickness of the materials was found to be 113mm for the asphalt arisings sub-base and 151mm for the control Type 1 sub-base.

Both in situ and laboratory tests were performed to assess the quality of compaction and the results are presented in Table 2.2. The in situ mean dry densities for the Type 1 and the asphalt planings sub-bases were both 102% of the maximum dry densities determined from vibrating hammer compaction tests, carried out in accordance with BS 1377 (BSI, 1990) and it was therefore evident that both sub-base materials were well compacted.

Table 2.2 A414 Cole Green: Compaction data

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<thead>
<tr>
<th></th>
<th>Type 1</th>
<th>Planings</th>
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<tr>
<td>Crushed rock</td>
<td>5.5</td>
<td>3.0</td>
</tr>
<tr>
<td>Maximum dry density (Mg/m³)</td>
<td>2.220</td>
<td>1.946</td>
</tr>
<tr>
<td>Mean in situ moisture content (%)</td>
<td>3.0</td>
<td>2.6</td>
</tr>
<tr>
<td>Mean in situ dry density (Mg/m³)</td>
<td>2.266</td>
<td>1.983</td>
</tr>
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2.2.5 Trafficking of sub-base
The foundations were trafficked with a rigid three-axle lorry, of total mass 24 tonnes, fitted with dual wheels on the rear two axles. The axle loads of the lorry were converted into an equivalent number of standard 80kN axles using the fourth power law (Liddle, 1962). The lorry was driven forward and then reversed over the test
sections, with care being taken to direct the driver along the same straight marked track, so that the wheel paths of successive lorry passes were superimposed. At intervals, trafficking was stopped and the height of the sub-base surface was measured at fixed locations to allow calculation of the deformations of the sub-bases.

The increase in mean deformation with cumulative traffic is shown in Figure 2.7. The trafficking trial ranked the performance of the control Type 1 sub-base and the planings sub-base as being similar, despite the planings sub-base being placed in a considerably thinner layer. Both sub-base materials failed to meet the 40mm maximum deformation requirement under trafficking.

2.2.6 Excavation following trafficking
Following the trafficking trial, the sub-bases were carefully excavated to expose the surface of the capping. The deformation of the capping was measured and varied from being almost undetectable to consisting of large sub-surface ruts, whose adjacent heave disrupted the overlying

![Figure 2.6](image)

**Figure 2.6** A414 Cole Green: Particle size distribution of recovered aggregate

![Figure 2.7](image)

**Figure 2.7** A414 Cole Green: Deformation of trial sections
sub-base. It was also noted that, as a result of compaction and internal shear deformation, both sub-bases had become thinner in the trafficking wheelpaths.

### 2.2.7 Laboratory CBR tests on planings

Laboratory CBR tests to BS 1377 (BSI, 1990) were carried out at different temperatures on planings which had been compacted at room temperature. The results of the CBR tests, presented in Table 2.3, clearly showed that the CBR was affected by temperature. When compacted to near identical dry densities, the CBR almost doubled for a 7°C reduction in temperature. As the individual particles of planings are themselves aggregations of smaller particles bound by bitumen, it was considered that increasing temperature was likely to increase the deformation of the planings particles and could also ease the relative movement of adjacent particles. It was concluded that temperature dependency of the structural properties of asphalt planings was significant. Also noted was the fact that the laboratory CBR values of the planings were less than the 30% required for Type 1 sub-base (HD 25/94, DMRB 7.1.2).

### Table 2.3 A414 Cole Green: Laboratory CBR tests on planings

| Dry density (Mg/m³) | 1.898 | 1.909 |
| Average test temperature (°C) | 19 | 12 |
| CBR (%) | 13 | 25 |

**Mean of five tests**

### 2.2.8 Summary of A414 Cole Green trial

Measurements from the sub-bases in the A414 Cole Green trial are summarised in Table 2.4.

### Table 2.4 A414 Cole Green: Foundation parameters and performance

| Stiffness measured on surface of capping (MPa) | 142 | 169 |
| Class 6F1 Capping CBR (%) | 46 | 34 |
| Sub-base thickness (mm) | 151 | 113 |
| Deformation after 100 lorry passes (300 standard axles) (mm) | 41.4 | 36.8 |
| Predicted deformation after 1000 standard axles (mm) | >40 | >40 |

The research from the A414 Cole Green trial concluded that:

- the asphalt planings sub-base performed similarly to the Type 1 crushed rock sub-base, despite being placed in a thinner layer;
- significant problems are caused by wetting of the formation prior to laying sub-base;
- the tests specified for unbound Type 1 sub-base materials were not directly applicable to asphalt arisings (oven drying of test samples resulted in particles adhering together);
- the structural properties of asphalt planings are temperature dependent.

### 2.3 A74 Gretna

This trial was carried out in October 1994 as part of the works to upgrade 6.4km of the A74 near Gretna (between Eaglesfield and Kirkpatrick-Fleming) in Southwest Scotland. The design of the trial is presented in Figure 2.8. The trial was constructed on two adjacent 150m lengths (Lane 1 and Lane 2) on the northbound carriageway of the A74. At the location of the trial, the road was formed on top of a new embankment, approximately five to six metres in height. This embankment consisted of both Class 1A and Class 2C granular fill, as described in the SHW (MCHW 1). Each lane incorporated a 50m control section of Type 1 granular sub-base material and two 50m sections of asphalt road planings. The planings, referred to as Planings A and Planings B, were obtained locally from different sources. Lane 1 was designed as a sub-base layer 150mm thick overlying 350mm of Class 6F2 capping. The capping material was naturally occurring gravel won on site. Lane 2 was designed as a 350mm thick sub-base placed directly on the granular fill of the embankment.

#### 2.3.1 Measurements on the formation

Prior to placing the sub-bases, the consistency of the substrate to the sub-base layer along the site was assessed using a FWD. The formation stiffness of Lane 1 ranged between 40MPa and 130MPa. For Lane 2, the stiffness also ranged between 40MPa and 130MPa. Equivalent regions were selected from the three trial sections and their location and stiffnesses are detailed in Table 2.5.

### Table 2.5 A74 Gretna: Equivalent regions

<table>
<thead>
<tr>
<th>Sub-base material</th>
<th>Chainage (m)</th>
<th>Average formation stiffness (MPa)</th>
<th>Standard deviation (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Lane 1</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Planings A</td>
<td>5700-5730</td>
<td>102.1</td>
<td>15.0</td>
</tr>
<tr>
<td>Planings B</td>
<td>5740-5770</td>
<td>78.8</td>
<td>27.7</td>
</tr>
<tr>
<td>Type 1</td>
<td>5780-5830</td>
<td>96.1</td>
<td>24.9</td>
</tr>
<tr>
<td><strong>Lane 2</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Planings A</td>
<td>5700-5725</td>
<td>116.0</td>
<td>22.1</td>
</tr>
<tr>
<td>Planings B</td>
<td>5735-5770</td>
<td>64.4</td>
<td>25.2</td>
</tr>
<tr>
<td>Type 1</td>
<td>5780-5830</td>
<td>110.1</td>
<td>20.0</td>
</tr>
</tbody>
</table>

Using the DCP, the CBR of the Lane 1 foundation was also assessed. Using the Kleyn DCP/CBR relation described in Overseas Road Note 8 (1990), the average CBR was 44%.

#### 2.3.2 Sub-base materials

Planings A were derived from a recently laid Dense Bitumen Macadam, planed from the new southbound carriageway of the A74 and rejected due to a poor longitudinal profile caused by incorrect adjustment of the paver. Planings B were planed from the old northbound carriageway of the A74 and consisted of either Hot Rolled Asphalt or Dense Bitumen Macadam, which had been overlain a lean mix concrete roadbase. The control Type 1 granular sub-base was a crushed limestone aggregate.
Figure 2.8 Layout of A74 trial

2.3.3 Particle size distribution of sub-base materials
The particle size distribution of the crushed limestone was determined according to BS 812 (BSI, 1985) and complied with the Specification for Type 1 sub-base (Series 800, MCHW 1). Whilst no grading envelope was specified for the planings in the contract, the particle size distribution was determined using a dry sieving procedure. The grading of Planings A was coarse of the lower boundary of the Type 1 Specification, for particles finer than 0.6mm, and the grading of Planings B was coarse of the lower boundary of the Type 1 Specification, for particles finer than 10mm.

2.3.4 Compaction and testing of sub-bases
Compaction was carried out using either a Bomag BW 212D-2 vibratory roller with a mass per metre width of 3010kg/m or a Dynapac CA251, with a mass per metre width of 2347kg/m. Both compactors were articulated self-propelled machines, driven by two pneumatically tyred wheels at the rear, and had a single smooth vibratory roll at the front. Compaction speeds during operation were between 2 and 3km/h. The Bomag complied with the ‘over 2900kg to 3600kg per metre/width’ compactor category of the SHW Table 6/4 (MCHW 1), whilst the Dynapac complied with the ‘over 2300kg to 2900kg per metre width’ category. Compaction was carried out in accordance with Method 6 of the SHW (MCHW 1). Using an optical levelling system, the thicknesses of the placed sub-bases were determined along the anticipated wheel paths of the test lorry. As shown in Table 2.6, the site control of sub-base thickness was poor.
Prior to trafficking, the FWD was used to determine the foundation stiffness of each lane after compaction of the sub-bases. The FWD results are presented in Table 2.7 and indicate that the placement and compaction of the sub-bases had not significantly changed the stiffness of the foundation.

The strengths of the sub-bases were also assessed using the DCP and CBR values were calculated using the Kleyn calibration (Overseas Road Note 8, 1990). The calculated equivalent CBR values ranged between 30% and 66% for Planings A, between 30% and 50% for Planings B and between 27% and 41% for the Type 1 control. As discussed in Section 7.4.1, laboratory calibrations of DCP versus CBR can be significantly different for planings and granular materials (Toombs et al., 1994, Steele and Snowdon, 1995 and MacNeil and Steele, 2002). The effect is primarily due to the plastic deformation that can occur during CBR tests on materials containing bitumen. The magnitude of the difference is indicated in Figure 2.10.

Retrospectively applying a laboratory DCP versus CBR calibration for the A74 planings (from Toombs et al., 1994) results in CBR values for the planings on site ranging between 4% and 20%. Whilst similar density data were recorded for samples A1 and A2, significant differences between samples B1 and B2 were measured.

The results of the laboratory compaction tests are presented in Figure 2.11 and Figure 2.12 for Planings A and Planings B respectively. Air voids curves have been generated using measured particle densities (Pd). Two air void curves are shown in Figure 2.12, reflecting the different values of Pd obtained for samples B1 and B2. For all tests, the dry density increased approximately linearly with moisture content, up to near the saturation point, where the tests were terminated. In terms of temperature, the overall effect of compacting the planings at the lower temperature was to slightly increase the dry density at the lower moisture contents (with the exception of sample B2, which showed an increase in density at the higher moisture contents). Testing at the lower temperature also produce shallower gradients on the compaction curves.

A summary of the Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) data for laboratory compaction of the bituminous planings are presented in Table 2.9.

<table>
<thead>
<tr>
<th>Description</th>
<th>Temperature (°C)</th>
<th>Optimum moisture content (%)</th>
<th>Maximum dry density (Mg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Planings A</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>200mm layer</td>
<td>20</td>
<td>8.5</td>
<td>1.970</td>
</tr>
<tr>
<td>150mm layer</td>
<td>5-10</td>
<td>8.7</td>
<td>1.924</td>
</tr>
<tr>
<td>Planings B</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>150mm layer</td>
<td>20</td>
<td>7.3</td>
<td>1.931</td>
</tr>
<tr>
<td></td>
<td>5-10</td>
<td>7.8</td>
<td>1.895</td>
</tr>
<tr>
<td>200mm layer</td>
<td>20</td>
<td>7.5</td>
<td>1.998</td>
</tr>
<tr>
<td></td>
<td>5-10</td>
<td>7.5</td>
<td>2.080</td>
</tr>
</tbody>
</table>
Figure 2.10 Laboratory CBR and DCP results for Planings A and B
Figure 2.11 A74 Gretna: Density data for Planings A
Figure 2.12 A74 Gretna: Density data for Planings B
2.3.5 Trafficking of sub-bases

The foundations were trafficked with a rigid three-axle lorry of total mass 24 tonnes, fitted with dual wheels on the rear two axles. The axle loads of the lorry were converted into an equivalent number of standard 80kN axles using the fourth power law (Liddle, 1962). The lorry was driven forwards and then reversed over the test sections, with care being taken to restrict the lorry to canalised wheel paths. At intervals, trafficking was stopped to permit the measurement of the deformation of the sub-bases at fixed locations. The increases in mean deformation of the foundations with cumulative traffic are presented in Figure 2.13 and Figure 2.14, for Lanes 1 and 2 respectively.

With the exception of the Type 1 on Lane 1, the deformation of the sub-base materials was lower than the maximum deformation of 40mm recommended by Powell et al. (1984). Following the 30mm deformation after 1000 standard axles sub-base stability criteria recommended by Earland and Pike (1985), Planings B sub-base on Lane 1 would be classified as borderline, whilst the Type 1 control sub-base failed dramatically. The poor performance of the Type 1 on Lane 1 may have resulted from saturation of the capping prior to laying the sub-base. It is also probable that, due to wet weather, the moisture content of the planings increased between the time of compaction and commencement of the trafficking trials. As a consequence of carriageway crossfall, Lane 1 would have become wetter.

**Figure 2.13** A74 Gretna: Deformation of Lane 1 trial sections

**Figure 2.14** A74 Gretna: Deformation of Lane 2 trial sections
that the SHW (MCHW 1) was amended to allow the use of slag bound material, and this amendment occurred prior to the introduction of Type 4 sub-base. As such, the clause numbers in the Draft Specification do not directly relate to the current amendments.

Before asphalt arisings were introduced in the Department of Transport Specifications and Standards, in order to test validity and robustness, it was recommended that the Draft Specification be tested in full-scale site trials under contractual conditions. The two specification validation trials took place on the A14 and A27, and the findings are reported in Section 4. In order to validate the performance of materials identified for use in the main works, the principal requirement for these validation trials was a trafficking trial, similar to those previously carried out.

4 Specification validation trials

The specification validation trials were not only required to test the validity and robustness of the Draft Specification, but also to assess the performance of the materials installed under contractual conditions. Data collected during construction and in-service monitoring of the trials were also intended to be used to facilitate the preparation of an End Performance Specification for the use of asphalt arisings as Type 4 sub-base.

4.1 A14 Milton

The validation trial on the A14 at Milton, near Cambridge, was carried out in August 1998. This trial represented the first application of the Draft Specification under normal contractual conditions. The scheme entailed the strengthening of approximately 1.2km of the dual 2-lane Milton to Fen Ditton stretch of the A14. As part of the scheme, it was decided to fully reconstruct the westbound carriageway. The contract, including a trial area of Type 4 sub-base, was undertaken using a lane rental scheme, and was awarded to Lafarge Redland Aggregates in July 1998.

The objectives of the validation trial were to:

- monitor the construction of the trafficking trial area which incorporated asphalt planings as a Type 4 sub-base, in accordance with the draft;
- check, via the trafficking trial, that the Type 4 sub-base met the performance requirements detailed in the Draft Specification;
- monitor the construction of a control area which used Type 1 (crushed rock) sub-base;

3 Draft specification for asphalt arisings

Following the three specification development trials in which the trial sub-base materials performed similarly to existing Type 1 sub-base materials, it was concluded that both granulated asphalt and asphalt arisings were suitable for use as sub-base materials. However, due to the particular material characteristics of asphalt arisings, which is discussed in further detail in Section 7, it was considered that a separate material type was required in the Series 800 of the SHW (MCHW 1). To progress the use of asphalt arisings as sub-base, a Draft Specification and associated Notes for Guidance were developed by Ellis and Earland (1998). A copy of the Draft Specification is presented in Appendix A. For reference, it should be noted

2.3.6 Summary of A74 trial

The results from the A74 trial are summarised in Table 2.10. Overall, the research from the A74 trial concluded that:

- the performance of the planings sub-base materials were broadly comparable and met the recommended 30mm deformation limit per 1000 standard axles trafficking criteria (with the exception of Planings B on Lane 1, which was borderline);
- in terms of in situ dry densities and air void contents, the moisture contents of the planings at the time of compaction were too low to ensure adequate states of compaction were achieved;
- the BS 1377 vibrating hammer test produced similar dry densities to those achieved in situ, for compaction of planings at low moisture contents. The laboratory compaction testing also demonstrated that had the planings been compacted within a range of OMC-2% to OMC, acceptable states of compaction would have been achieved;
- due to the effect of loading rate on strength tests for planings materials, the CBR test may not be appropriate for recycled unbound planings materials;
- where in situ CBR values are calculated from DCP testing, a material specific DCP/CBR calibration should be used;
- the effect of temperature on both laboratory and in situ CBR and density testing of planings was significant.

Table 2.10 A74 Gretna: Summary of foundation parameters and performance data

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Lane 1</th>
<th>Lane 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type 1</td>
<td>Planings A</td>
</tr>
<tr>
<td>Average stiffness measured on surface of capping (MPa)</td>
<td>96</td>
<td>102</td>
</tr>
<tr>
<td>Average stiffness measured on surface of sub-base (MPa)</td>
<td>73</td>
<td>107</td>
</tr>
<tr>
<td>Sub-base thickness (mm)</td>
<td>130</td>
<td>160</td>
</tr>
<tr>
<td>Average dry density (Mg/m³)</td>
<td>2.029</td>
<td>1.864</td>
</tr>
<tr>
<td>Air voids content (%)</td>
<td>6.5</td>
<td>15.9</td>
</tr>
<tr>
<td>Deformation after 1000 standard axles (mm)</td>
<td>&gt;40</td>
<td>18</td>
</tr>
</tbody>
</table>
assess, on completion of the trafficking trial and the acceptance of the asphalt planings as Type 4 sub-base, the long term performance of the nearside lane of the westbound carriageway over the length of the reconstruction.

4.1.1 Construction of trial areas
The majority of the A14 carriageway under reconstruction had been constructed on a Gault Clay embankment that ranged from natural ground level to between five and six metres in height. The area designated for the trafficking trial was on the A14 westbound carriageway. This location corresponded to the very western end of the scheme, where the embankment tapered down to natural ground level.

Preparation for the trafficking trial involved planing out a length of the carriageway, encompassing the offside lane (Lane 2) and a small percentage of the near side lane (Lane 1). The in situ pavement material was removed to a depth of approximately 610mm, which resulted in the surface of the Gault Clay subgrade being exposed. The subgrade was graded and then the surface sealed by compacting with a roller.

In total, a plan area of 5.3m wide by approximately 90m long was planed out. A minimum length of 10m lead in/lead out section was included at each end of the trial area to allow the trafficking lorry to park and line up. This left a length of 60m for the trafficking trial for the asphalt planings. The layout of the trafficking trial site and cross-section of the trial area are shown in Figure 4.1.

4.1.2 Assessment of subgrade
The subgrade strength was assessed using a MEXE cone penetrometer, fitted with a small cone. Cone Index (CI) values were recorded over the exposed subgrade area and converted to California Bearing Ratio (CBR) using CI divided by 20 (Black, 1979). The CBR of the subgrade in the planings trial area was 3 to 4%, while that of the subgrade in the Type 1 control area was 5%.

Subgrade stiffness was also assessed using the FWD, using a dynamically loaded plate of 0.45m diameter, which was considered to significantly stress a depth of up to 0.7m. Within the trafficking trial section, the stiffness of the subgrade was consistent between chainage 1360m and 1400m and was within the range 11 to 24MPa in both near side and offside wheel paths. To the east of chainage 1400m, the subgrade stiffness increased in both wheel paths. The increase was most marked in the near side wheel path. This stiffer region of subgrade corresponded to the first 10 metres of the trafficking trial site area (see Figure 4.1) and the remainder of the trial area was considered to be constructed on subgrade of equivalent stiffness.

The subgrade stiffness in the Type 1 control area was significantly more variable and stiffer than the subgrade on which the trafficking trial was founded. The subgrade underlying the control area had stiffness in the range 19MPa to 135MPa.

![Figure 4.1 Layout of A14 trial](image-url)
4.1.3 Sub-base material and construction

The asphalt arisings used in the trial were planed from the A14 westbound carriageway, at the eastern end of the site. It was evident that this carriageway had undergone many repairs, as the planed wearing course materials showed considerable variations. Also evident in the planed material was a considerable amount of sand and rounded gravel - according to site personnel, this was consistent with practices used when the carriageway was originally constructed.

Due to a lack of on-site testing facilities, particle size data were not available when decisions to accept or reject loads of planings were taken. Using experience gained from previous TRL research, TRL personnel were able to judge when the size and frequency of the large lumps, typically in excess of 100mm, were unacceptable. Using this judgement, and with the assistance of the contractor, sufficient homogeneous planed material was sourced for the trafficking trial area.

An initial nominal 200mm layer of Type 4 sub-base was delivered by tipper lorries and spread, using a crawler dozer, to a target loose layer thickness of 250mm. During this process, the loose material was trafficked by the lorries using the uncompacted layer to deliver material to the opposite end of the trial area. The layer was then wetted-up using a pump driven tank and hose. Due to the low output of the hose and pump system, three complete wetting-up passes were required to increase the moisture content of the material until samples of the road planings appeared to be close to saturation. The material was compacted with a vibratory roller and the compacted layer thickness measured using a semi-automatic optical level. The mean thickness of the first placed layer was 202mm. On completion of wetting-up, compaction and FWD testing of this first sub-base layer, a second nominally 150mm thick layer of Type 4 sub-base was placed, at a target loose layer depth of 190mm, and processed in the same manner. This resulted in a mean compacted layer depth of 155mm.

Both sub-base layers were compacted using a Bomag BW161 AD vibratory roller with a mass per metre width of 2757kg. This is a smooth twin drum, self-propelled vibratory compactor, which complied with the ‘2300kg to 2900kg per metre/width’ category of the SHW Table 8/1 (MCHW 1). Compaction was carried out in accordance with Table 8/1 of the SHW (MCHW 1); the initial 200mm layer was compacted with 6 machine passes, and the second 150mm layer was compacted with 3 machine passes.

Weather conditions on site were good throughout the trial. During compaction, the ambient air temperature varied between 19°C and 21°C and the temperature of the planings varied between 20°C and 26°C.

4.1.4 Materials testing

On behalf of Redlands Aggregates, and acting on instruction from TRL, Flexitec (a commercial materials test house) undertook sampling and initial site testing of the material used for the trial. The scope of the testing included:

- microwave moisture content prior to compaction;
- BS 1377 (vibrating hammer) optimum moisture content and maximum dry density (BSI, 1990);
- particle size distribution.

Details of the Flexitec compaction data are presented in Table 4.1 and Figure 4.2.

<table>
<thead>
<tr>
<th>Moisture content (%)</th>
<th>Dry density (Mg/m³)</th>
<th>Particle density (Mg/m³)</th>
<th>Air voids (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9</td>
<td>1.589</td>
<td>21.9</td>
<td></td>
</tr>
<tr>
<td>2.6</td>
<td>1.625</td>
<td>27.0</td>
<td></td>
</tr>
<tr>
<td>3.8</td>
<td>1.714</td>
<td>20.9</td>
<td></td>
</tr>
<tr>
<td>4.9</td>
<td>1.687</td>
<td>20.2</td>
<td></td>
</tr>
<tr>
<td>5.3</td>
<td>1.598</td>
<td>23.8</td>
<td></td>
</tr>
<tr>
<td>6.5</td>
<td>1.598</td>
<td>21.9</td>
<td></td>
</tr>
</tbody>
</table>

The results of a subsequent laboratory compaction test, undertaken by TRL following BS 1377: Part 4: Method 3.7 (vibrating hammer) (BSI, 1990), and the associated particle density test, are presented in Table 4.2 and Figure 4.2. The moisture contents of all the TRL test samples were determined using an oven set at 45°C. For moisture contents in excess of 4.5%, compaction resulted in saturated fines flowing out of the bottom of the mould. As a consequence, test levels 5 and 6 resulted in similar moisture contents, and low dry densities. In an attempt to achieve a higher moisture content, the mould bottom was sealed and the test repeated with a sample at a moisture content of approximately 7%. Linear regression analysis of the data resulted in a very poor correlation between moisture content and dry density.

<table>
<thead>
<tr>
<th>Moisture content (%)</th>
<th>Dry density (Mg/m³)</th>
<th>Particle density (Mg/m³)</th>
<th>Air voids (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>1.856</td>
<td>18.4</td>
<td></td>
</tr>
<tr>
<td>3.4</td>
<td>1.816</td>
<td>17.8</td>
<td></td>
</tr>
<tr>
<td>4.6</td>
<td>1.808</td>
<td>16.0</td>
<td></td>
</tr>
<tr>
<td>4.5</td>
<td>1.829</td>
<td>2.39</td>
<td>15.2</td>
</tr>
<tr>
<td>4.6</td>
<td>1.783</td>
<td>17.2</td>
<td></td>
</tr>
<tr>
<td>5.1</td>
<td>1.774</td>
<td>16.7</td>
<td></td>
</tr>
<tr>
<td>7.6</td>
<td>1.877</td>
<td>7.2</td>
<td></td>
</tr>
</tbody>
</table>

The TRL laboratory compaction testing concluded that the optimum moisture content (OMC), taken to be the intercept of the linear regression and the 10% air void line, was 6.9%. Consequently, the moisture content range for acceptability of the asphalt planings was 4.9% to 6.9%. The maximum dry density (MDD) corresponding to the OMC was 1.847Mg/m³.

On each of the two Type 4 sub-base layers, two sand replacement density tests were carried out in accordance with BS 1377: Part 4: Method 2.2 (BSI, 1990), using the large cylinder method for hole depths of 150mm. These sand replacement tests were undertaken as checks for adequate compaction of the two layers, and were not part of the Draft Specification. The moisture contents of the samples removed for sand replacement tests, were determined by Flexitec, using a microwave on a low power setting. The sand replacement data are presented in Table 4.3.

For the first two sand replacement tests on layer 1, the moisture contents are estimates and it is assumed that the
Figure 4.2 A14: Density data
moisture content of the layer, at the time of compaction, was within the range OMC to OMC minus 2%, as required by the Draft Specification. Significantly, the lower 20mm of the material removed from the Test 1 hole was completely dry; this is probably a reflection on the method used to apply moisture prior to compaction. The resulting mean dry density was equivalent to 100% relative compaction, with a mean air void content of 13%.

For the second layer, the two tests were undertaken some 20 hours after compaction, and due to drying out, the measured moisture contents should be considered to be considerably less than the moisture contents at the time of compaction. Accordingly, the data were recalculated using the mean estimated moisture contents of samples of the layer prior to compaction, obtained by Flexitec. These data are referred to as Tests 3 and 4, which used the mean moisture content at time of compaction. Tests 3A and 4A were carried out some 20 hours after compaction - the measured moisture contents are likely to be significantly less than the moisture contents at the time of compaction. Tests 3A and 4A used the mean moisture content at time of compaction.

For the second layer, the two tests were undertaken some 20 hours after compaction, and due to drying out, the measured moisture contents should be considered to be considerably less than the moisture contents at the time of compaction. Accordingly, the data were recalculated using the mean estimated moisture contents of samples of the layer prior to compaction, obtained by Flexitec. These data are referred to as Tests 3 and 4, which used the mean moisture content at time of compaction. Tests 3A and 4A were carried out some 20 hours after compaction - the measured moisture contents are likely to be significantly less than the moisture contents at the time of compaction. Tests 3A and 4A used the mean moisture content at time of compaction.

4.1.5 Assessment of sub-base layers

FWD tests carried out in the trafficking trial section indicated that placing the first sub-base layer of asphalt planings increased the foundation stiffness to between 20 and 46MPa. The foundation stiffness was further increased to approximately 55MPa when the second sub-base layer was placed. The foundation stiffnesses measured on the Type 1 control sub-base, following compaction of the first layer, resulted in less variation, but there was little absolute increase in foundation stiffness. Tests carried out on the surface of the finished Type 1 foundation indicated that the compaction of the second layer had continued to reduce the variability but had only marginally increased the foundation stiffness to a typical value of 55MPa, the same as was attained for the asphalt arisings.

4.1.6 Trafficking trial

The layout of the trafficking trial is shown in Figure 4.1. Trafficking was conducted using a loaded, 3 axle, rigid tipper lorry which had a gross vehicle weight of 24.4 tonnes. During the trial, the lorry was restricted to canalised wheel paths in the existing offside and nearside wheel paths of Lane 2 of the westbound carriageway. Each pass of the lorry applied the equivalent of 3.51 standard axles (Liddle, 1962).

A grid for measuring the deformation was marked on the sub-base. The grid comprised five lines: lines A and B corresponded with the centre of the two offside rear tyre-paths, line C corresponded with the centreline, and lines D and E corresponded with the centre of the two nearside rear tyre-paths. Along the length of the trafficking trial, measurements of deformation were taken every 5 metres.

The lorry was driven forward at a steady speed of approximately 4 kilometres per hour and then reversed back along the trial section in the same wheel paths. In total 384 passes of the lorry were made with deformation measurements being taken after 6, 12, 24, 48, 96, 192 and 384 passes. At intervals, measurements were taken along the carriageway to assess any potential heave from the trafficking of the lorry. During the trafficking, it was observed that after only 192 passes, equivalent to 673 standard axles, the rate of rutting increased considerably and the materials in the wheel paths showed signs of distress. Throughout trafficking, the weather was fine and dry and the temperatures recorded during trafficking, at 50mm depth, varied between 13°C and 18°C.

The results of the trafficking trial are presented in Figure 4.3. At 1000 standard axles, the mean deformations were 31.0mm, 19.0mm, 43.9mm, and 53.8mm for lines A, B, D and E respectively. There was a slight crossfall across the trial area at the start of the trial, of approximately 80mm from line A to E. The crossfall was broadly similar to the camber on the original carriageway. It is likely that this resulted in unequal loading across the axles, with lines D and E receiving

<table>
<thead>
<tr>
<th>Sub-baselayer</th>
<th>Test</th>
<th>Location</th>
<th>Bulk density (Mg/m³)</th>
<th>Moisture content (%)</th>
<th>Dry density (Mg/m³)</th>
<th>Air voids (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (nominal 200mm)</td>
<td>1</td>
<td>52.5m offside</td>
<td>1.923</td>
<td>5.0</td>
<td>1.832</td>
<td>14.2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>27.5m nearside</td>
<td>1.977</td>
<td>5.0</td>
<td>1.882</td>
<td>11.8</td>
</tr>
<tr>
<td>2 (nominal 150mm)</td>
<td>3</td>
<td>26m centrelne</td>
<td>1.789</td>
<td>2.0</td>
<td>1.754</td>
<td>23.1</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>47m centrelne</td>
<td>1.865</td>
<td>2.2</td>
<td>1.825</td>
<td>19.6</td>
</tr>
<tr>
<td>2 – recalculated (nominal 150mm)</td>
<td>3A</td>
<td>26m centrelne</td>
<td>1.789</td>
<td>5.0</td>
<td>1.704</td>
<td>20.2</td>
</tr>
<tr>
<td></td>
<td>4A</td>
<td>47m centrelne</td>
<td>1.865</td>
<td>5.0</td>
<td>1.776</td>
<td>16.8</td>
</tr>
</tbody>
</table>

1 Moisture contents for tests 1 and 2 are estimates (incomplete Flexitec test data showed approx. 4%).
2 Tests 3 and 4 were carried out some 20 hours after compaction - the measured moisture contents are likely to be significantly less than the moisture contents at the time of compaction. Tests 3A and 4A used the mean moisture content at time of compaction.
3 Particle density = 2.39Mg/m³, following test method in BS 1377 (BSI, 1990).
the larger loads. This resulted in greater mean deformations for lines D and E compared to lines A and B.

The overall mean deformation following 285 passes of the lorry (equivalent to 1000 standard axles) was 36.9mm. The Draft Specification for using asphalt arisings as Type 4 sub-base required that the mean vertical deformation after 1000 standard axles be less that 30mm. The asphalt planings used for this trial failed to meet this performance criterion and were therefore unsuitable for use as Type 4 sub-base.

4.1.7 Discussion

The asphalt arisings used as sub-base in the trial complied with Draft Specification for Type 4 sub-base (Ellis and Earland, 1998) in terms of their method of production and particle size distribution. Visual examination of the material clearly indicated that the bitumen content was well below the maximum 10% allowable.

The limited number of moisture content determinations brought into question whether the asphalt planings were compacted within the moisture content range specified. In all other respects, the material was placed and compacted in accordance with the Draft Specification. Previous research on the use of asphalt planings as capping (Steele and Snowdon, 1995) and structural backfill (MacNeil et al., 1997) has shown that mean air void contents of up to 10% can be anticipated for well compacted asphalt planings layers. However, the mean air void contents for the asphalt planings used in this trial were considerably higher than this value, indicating that the states of compaction achieved were poor. Such poor states of compaction may seem to be at odds with the high relative compaction achieved for both layers, but the laboratory compaction plot shows that, for this particular material, high air void contents can be expected over the moisture content acceptability range.

A number of reasons may have contributed to the poor compaction performance of the asphalt planings:

1 Poor grading.

Whilst all the sampled material met the grading requirement, it was evident that the delivered material had lumps greater than 100mm in size. These lumps tended to fall to the base of the loads tipped by the delivery lorries, and after spreading, were to be found at the bottom of each placed layer.

2 Water addition.

Using a water tank, pump and hose proved to be an ineffective and time consuming method of applying moisture to asphalt planings. Ideally, the trial area would have been wetted up with a towed water bowser, and compaction could have been completed shortly afterwards. The laboratory compaction test also highlighted the difficulty in achieving high moisture contents; this is a possible reflection on the high sand and low bitumen content of the material.

3 Angularity.

It is likely that the principal reason for the failure of the material to meet the performance criteria for Type 4 sub-base, as measured by the trafficking trial, was due to the poor mechanical interlock between the larger particles. As indicated by the relatively flat compaction curve for the asphalt planings used in this trial, achieving low air void contents is not possible when compacting a material with a significant amount of rounded particles. The sand and gravel component of the planings was considerable, and the gravel tended to be rounded. Problems with rounded capping materials, typically as dug sand and gravels, have been reported by MacNeil and Snowdon (1996), who

![Figure 4.3 A14: Deformation of Type 4 sub-base trial area](image-url)
concluded that trafficking trials were the only reliable method of identifying materials with stability problems. MacNeil and Snowdon also suggested highlighting possible problem materials and restricting their use locally.

During the trial, the on-site staff were under pressure to ensure that the main works were not being unduly held up. It is likely that this contributed to deviations in the methodology set out in the Draft Specification, e.g. insufficient material acceptability data and an inadequate technique for wetting-up prior to compaction.

The conclusion reached as a result of the A14 validation trial were:
- Asphalt planings can meet the material properties specified for Type 4 sub-base material.
- Acceptability data must be available prior to commencing construction. It is vital to ensure that that the grading and moisture content of the asphalt arisings complies with that given in the Draft Specification for Type 4 sub-base material.
- The results of the A14 trial indicate that it is essential to conduct a preliminary trafficking trial prior to incorporating Type 4 sub-base into the main works.
- Any further validation trials carried out for research purposes should not be undertaken within a live lane rental scheme, where the contractor is facing heavy financial penalties if the project is delayed.

4.2 A27 Arundel

The A27 Arundel works were carried out in October 2000 and entailed the strengthening of approximately 4km of the dual 2-lane Avisford to Arundel stretch of the A27. As part of the scheme, it was decided to fully reconstruct part of the westbound carriageway. The main objectives of the validation trial were to:
- monitor the construction of the trafficking trial area which incorporated asphalt planings as a Type 4 sub-base, in accordance with the Draft Specification;
- check, via the trafficking trial, that the Type 4 sub-base met the performance requirements detailed in the Draft Specification.

4.2.1 Construction

The initial phase of the work involved planing out an approximately 600m long length of the westbound carriageway (immediately before the junction with Yapton Lane), encompassing Lane 1 and an area between Lane 1 and the verge. The in situ pavement material was removed to a depth of approximately 450mm, which resulted in the surface of a capping layer being exposed. The capping layer, comprising different materials (mostly consisting of large, single sized), had been originally placed in various thicknesses. During planing of the first 100m of the reconstruction area, the capping layer was found to be very thin and was therefore removed. In some locations in this initial 100m, the exposed clay subgrade was weak (CBRs being estimated as less than 4%), and had to have remedial treatment. For the remainder of the reconstruction area, the capping layer was simply graded and sealed by compacting with a roller. The area designated for the trafficking trial was selected to encompass a section of the reconstruction where the capping was thicker and appeared to be intact.

In total, a plan area of 6m wide by approximately 90m long was designated for the trial (see Figure 4.4). At each end of the trial, lead in/lead out areas were marked out to allow the trafficking lorry to park and line up. This left a length of 60m for the asphalt arisings trafficking trial area.

![Figure 4.4 Layout of A27 trial](image-url)
The construction methods used for the trafficking trial and control sections were identical. The asphalt arisings used in the trial were planed from the westbound carriageway, and it was evident that the carriageway had undergone repairs, as the planed wearing course materials showed minor variations. Following planing, the contractor carried out laboratory testing to provide acceptability data. Using these data and experience gained from previous TRL studies, sufficient homogeneous planed material was identified for use in the trafficking trial area. The asphalt arisings were transported using tipper wagons and then spread and levelled using a 360° wheeled excavator. The target loose layer thickness was 135mm. In an attempt to achieve a suitable moisture content for the asphalt arisings in the first placed layer, water was added using the spray bars of a vibratory roller (roller passes were completed without the use of vibration). On completion of wetting up, the layer was then compacted (with vibration) and the compacted layer thickness measured using a semi-automatic optical level. The mean layer thickness of the first layer was 127.4mm, which closely matched the target compacted thickness value of 125mm. The surface stiffness of the compacted first sub-base layer was then evaluated using the FWD. Subsequent examination of areas where samples of asphalt arisings were removed revealed that water had only penetrated the top 30 to 50mm of the layer. For this reason, it was decided to add water prior to delivery of each load to be used in the second layer. This was achieved using a lorry mounted water bowser, which sprayed the asphalt arisings as they were being loaded into the tipper wagons. Following the same procedure as used for the first layer, the asphalt arisings were spread and compacted. The mean compacted thickness of the second layer was 101.3mm, which resulted in an overall mean Type 4 sub-base thickness of 228.7mm. On completion of levelling, the surface stiffness of trial area was again evaluated with the FWD.

All compaction was carried out in accordance with Table 8/1 of the SHW (MCHW 1), using a Bomag BW161 AD vibratory roller with a mass per metre width of 3042kg. Both the first and second nominal 125mm sub-base layers were compacted with 3 machine passes. During construction of the trial, the weather conditions were fair and the air temperature varied between 10°C and 14°C.

4.2.2 Materials testing

After compaction of each layer, density measurements were taken using a NDG in the direct transmission mode. The density tests were intended to be used as a check for the trafficking trial area. The asphalt arisings were intended to be used as a check for acceptance data. Using these data and experience gained from previous TRL studies, sufficient homogeneous planed material was identified for use in the trafficking trial area. The asphalt arisings were transported using tipper wagons and then spread and levelled using a 360° wheeled excavator. The target loose layer thickness was 135mm. In an attempt to achieve a suitable moisture content for the asphalt arisings in the first placed layer, water was added using the spray bars of a vibratory roller (roller passes were completed without the use of vibration). On completion of wetting up, the layer was then compacted (with vibration) and the compacted layer thickness measured using a semi-automatic optical level. The mean layer thickness of the first layer was 127.4mm, which closely matched the target compacted thickness value of 125mm. The surface stiffness of the compacted first sub-base layer was then evaluated using the FWD. Subsequent examination of areas where samples of asphalt arisings were removed revealed that water had only penetrated the top 30 to 50mm of the layer. For this reason, it was decided to add water prior to delivery of each load to be used in the second layer. This was achieved using a lorry mounted water bowser, which sprayed the asphalt arisings as they were being loaded into the tipper wagons. Following the same procedure as used for the first layer, the asphalt arisings were spread and compacted. The mean compacted thickness of the second layer was 101.3mm, which resulted in an overall mean Type 4 sub-base thickness of 228.7mm. On completion of levelling, the surface stiffness of trial area was again evaluated with the FWD.

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4.2.2 Materials testing

After compaction of each layer, density measurements were taken using a NDG in the direct transmission mode. The density tests were intended to be used as a check for adequate compaction of the two layers, and were not part of the Draft Specification.

<table>
<thead>
<tr>
<th>Sub-base layer</th>
<th>NDG bulk density (Mg/m³)</th>
<th>Corrected NDG moisture content (%)</th>
<th>Dry density (Mg/m³)</th>
<th>Air voids (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.988</td>
<td>2.8</td>
<td>1.934</td>
<td>14.0</td>
</tr>
<tr>
<td>2</td>
<td>2.009</td>
<td>5.1</td>
<td>1.913</td>
<td>10.8</td>
</tr>
</tbody>
</table>

Moisture contents given by the NDG were corrected to take account of the bituminous content of the asphalt arisings. The correction was carried out in accordance with the findings from previous TRL studies Yuille (1996) (a reliable estimate of the true in situ moisture content could be obtained by subtracting the percentage bitumen content from the moisture content measured using the NDG). As the bitumen content reported by the contractor was 5.6%, this value was subtracted from all measured NDG moisture contents.

Particle size distribution tests were carried out on the asphalt arisings and indicated that the material complied with the relevant grading limits in the Draft Specification. The results of laboratory compaction testing, undertaken by TRL following BS 1377: Part 4: Method 3.7 (vibrating hammer) (BSI, 1990), are presented in Table 4.4 and Figure 4.5. The moisture contents of all the test samples were determined using a microwave oven on a low power setting. The resulting OMC was 5.9%, and therefore the moisture content range for acceptability of the asphalt arisings was 3.9% to 5.9%. The MDD corresponding to the OMC was 1.990Mg/m³.

<table>
<thead>
<tr>
<th>Moisture content (%)</th>
<th>Dry density (Mg/m³)</th>
<th>Assumed particle density (Mg/m³)</th>
<th>Air voids (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6</td>
<td>1.886</td>
<td>23.2</td>
<td>20.2</td>
</tr>
<tr>
<td>3.0</td>
<td>1.972</td>
<td>12.0</td>
<td>12.0</td>
</tr>
<tr>
<td>4.1</td>
<td>1.908</td>
<td>2.40</td>
<td>12.7</td>
</tr>
<tr>
<td>5.5</td>
<td>2.006</td>
<td>5.4</td>
<td>5.4</td>
</tr>
<tr>
<td>6.5</td>
<td>1.921</td>
<td>7.4</td>
<td>7.4</td>
</tr>
</tbody>
</table>

For the first sub-base layer, the in situ moisture content estimated using the NDG was approximately 3%, which was less than the lower acceptability limit. The associated mean relative compaction was 97% relative compaction. For the second layer (where an improved method of wetting the asphalt arisings prior to delivery was used), the in situ moisture content was approximately 5% and the associated mean relative compaction was 96%. Whilst the improvement in control of moisture did not produce improvements in relative compaction, probably due to shallow gradient of the compaction line (see Figure 4.5), improvements in the state of compaction, as measured from air void data, were realised.

4.2.3 Subgrade and sub-base

The results of the FWD tests carried out on the subgrade in the near side wheel path of the trafficking trial area showed that the subgrade stiffnesses ranged between 17 to 53Mpa. The corresponding offside wheel path showed considerable variations, with stiffness values ranging between 15 and 300Mpa. The reason for such variations was unclear, but was likely to be due to the coarse nature of the existing formation in these areas.

The FWD tests carried out on the compacted surface of the first sub-base layer in the trafficking trial section resulted in foundation stiffness values broadly similar to...
those measured for the subgrade but with reduced variability in the offside wheel path. Measurements taken on the compacted surface of the second Type 4 sub-base layer resulted in typical foundation stiffness values approaching 50MPa. Due to time constraints and equipment availability, it was not possible to carry out FWD tests on the Type 1 sub-base control area.

4.2.4 Trafficking trial
The trafficking trial was conducted using a loaded, 3 axle, rigid tipper lorry (see Figure 4.6) which had a gross vehicle weight of 23.9 tonnes. Each pass of the lorry applied the equivalent of 2.92 standard axles (Liddle, 1962). In the same manner as the trafficking conducted at the previous A14 trial, the lorry was restricted to canalised wheel paths which coincided with the existing offside and nearside wheel paths for Lane 1 of the westbound carriageway. To assist the lorry driver, the outer edge of the two offside rear tyre paths was marked on the completed sub-base over the length of the trial.

A grid for measuring the deformation was also marked on the sub-base. The grid comprised five lines: lines A and B corresponded with the centre of the two offside rear tyre-paths, line C corresponded with the centreline, and lines D and E corresponded with the centre of the two nearside rear tyre-paths (see Figure 4.4). A grid spacing of 5m

Figure 4.5 A27: Density data
intervals along the 60m length of the trial was used, resulting in a total of thirteen measurement points along each of the lines.

For the first pass, the lorry was driven backwards over the 60m test area, and stopped in the lead out area. Both the lead out and lead in lengths were selected to ensure that the lorry’s suspension had stabilised after moving off, and that the lorry was travelling at a steady speed of approximately 3.2 km/h prior to entering the test area. The lorry was then driven forwards over the trial section and the deformation caused by these 2 passes measured, for all grid points on Lines A and B, D and E, using a semi-automatic optical level. The deformation at each point was defined as the change from the original level, as measured prior to the commencement of trafficking. This process was repeated after 6, 12, 24, 48, 96, 192, 350 and 400 passes of the lorry.

The results of the trafficking trial are presented in Figure 4.7. At 1022 standard axles, the mean deformations were 18.5mm, 13.5mm, 17.9mm, and 23.5mm for lines A, B, D and E respectively.

The overall mean deformation following 350 passes of the lorry (equivalent to 1022 standard axles) was 18.3mm. As the Draft Specification for Type 4 sub-base required that the mean vertical deformation after 1000 standard axles be less than 30mm, the asphalt arisings used for this trial met the performance requirement for Type 4 sub-base.

4.2.5 Discussion

In terms of method of production, particle size distribution and resistance to deformation, the asphalt arisings used as sub-base on the A27 trial complied with the requirements of the Draft Specifications for Type 4 sub-base (Ellis and Earland, 1998). As a result of this compliance, the Type 4 sub-base was incorporated into the scheme. The tight schedule for completion of the scheme left only sufficient time to carry out the Type 4 trafficking trial before placing of the bound pavement layers. For this reason, Type 1 sub-base was used on the remaining sections.

The A27 trial, and the previous A14 trial, highlighted the benefits of carrying out a trafficking trial prior to the commencement of the main works. Such a procedure would reduce the impact of asphalt arisings failing to meet the trafficking requirements, allow better planning, and reduce delays in completion of the works. However, it is recognised that a pre-works trial may not be a viable option on small schemes. If a pre-works trial is carried out, as recommended, adequate engineering supervision would be required to ensure that the asphalt arisings used in the main works were similar to that being evaluated in the trafficking trial. When evaluating acceptability data for Type 4 sub-base for both the trafficking trial and the main works, experience suggests that it is essential that the

![Figure 4.6 A27: Trafficking of Type 4 sub-base](image)

![Figure 4.7 A27: Trafficking trial data](image)
grading and moisture content of the asphalt arisings comply with the requirements of the Specification.

Following validation trials of the Draft Specification, Type 4 sub-base material was included in the current version of Series 800 in the SHW (MCHW 1).

5 Type 4 compaction trial

5.1 TRL Type 4 compaction trial

To confirm that appropriate states of compaction were being achieved using the Method Specification described in Series 800 of the SHW (MCHW 1, 2001), a compaction trial of Type 4 sub-base was carried out at TRL in September 2001. At the same time, a small trafficking trial was carried out to provide additional evidence of the equivalence of the performances of Type 4 and Type 1 sub-bases.

A forest track on the TRL site, comprising a silty, sandy-clay subgrade, was prepared by grading a trial length of 120m by 5m to formation level. After grading, the formation stiffness was assessed using the FWD and equivalent regions were selected for the trafficking trial. Within the trafficking trial section, the average formation stiffness was 39MPa in the area designated to have Type 1 sub-base area and 36MPa in that where the Type 4 sub-base was to be placed. The formation was also assessed using the DCP, which indicated a CBR of 8% in the area selected for the Type 1 sub-base area and 9% in the area selected for the Type 4 sub-base. A 200mm compacted layer thickness of sub-base was therefore specified for the trial.

The Type 4 sub-base was supplied by Tarmac Limited, from their site at Hayes, Middlesex. It was sourced from their stockpile of asphalt arisings and processed to meet the requirements of Type 4, Clause 806 (MCHW 1). The bitumen content was 5.3% and the grading was in the middle of the specified grading envelope for Type 4. The optimum moisture content of the asphalt arisings was determined using the BS 1377 vibrating hammer test (BSI, 1990), and was found to be 5.7% at a maximum dry density of 1.892Mg/m³. The Type 1 sub-base was crushed limestone, supplied by Hanson Aggregates in Frome, Somerset.

5.1.1 Trafficking trial

Both sub-bases were compacted using a Benford TV1700 tandem roller, with mass per metre width of 1998kg. The Method Specification for this category of compaction plant is 8 machine passes. The compacted thickness averaged 190mm for the Type 1 sub-base and 206mm for the Type 4 sub-base.

The in situ bulk density was measured using a Troxler 3440 NDG in the direct transmission mode. The NDG bulk density calibration was checked, using samples of the Type 1 and 4 sub-base materials, via compaction into a large box of known dimensions (equivalent of one of the five calibration points described in BS 1377 calibration process). The results indicated that the NDG bulk density measurement was within 1% of the calculated value.

At the trial site, bulk density was measured at locations in each test zone, with the NDG being operated in two orientations (rotated through 180°) at each location. Depth of operation of the NDG was 101.6mm (i.e. 4 inches). Dry densities were calculated from bulk density data, using the in situ moisture contents determined by oven drying of samples of the materials recovered from the area adjacent to each NDG test. The mean compacted in situ dry density of the Type 1 was 2.081Mg/m³. For the Type 4, the mean compacted in situ dry density was 1.88Mg/m³, equivalent to 99.4% relative compaction.

FWD measurements on the compacted sub-bases showed the surface modulus of the Type 1 and Type 4 foundations to be 62MPa and 51MPa respectively (an increase over the subgrade surface modulus of 60% for the Type 1 and 42% for the Type 4).

A standard trafficking trial was carried out and the average deformation of both sub-bases, after 1000 standard axles, was 35mm. Whilst both sub-bases therefore exceeded the 30mm target in the Specification, it was significant to note that the Type 4 sub-base performed similarly to the standard Type 1 sub-base during trafficking.

5.1.2 Compaction trial

The compaction trial area (as shown in Figure 5.1) was divided into compaction zones of 2, 4, 6 and 8 passes of the Benford TV1700 tandem roller.

<table>
<thead>
<tr>
<th>8 Passes</th>
<th>6 Passes</th>
<th>4 Passes</th>
<th>2 Passes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5.1 Layout of TRL compaction trial
The compacted thickness achieved in the nearside and offside wheel paths is shown in Table 5.1. The \textit{in situ} bulk density for the compaction trial was measured following the same procedure used for the assessing the trafficking trial area. Bulk density was measured at three locations in each test zone with the NDG being operated in two orientations (rotated through 180°) at each location, resulting in six readings in total. The depth of operation of the NDG was 152.4mm (i.e. 6 inches). The measurements were averaged separately for 2, 4, 6 and 8 roller passes and are presented in Table 5.2.

Table 5.1 TRL Compaction trial: Type 4 sub-base thicknesses

<table>
<thead>
<tr>
<th>Wheelpath</th>
<th>Nearest</th>
<th>Offside</th>
<th>Combined</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average thickness (mm)</td>
<td>195</td>
<td>205</td>
<td>200</td>
</tr>
<tr>
<td>Standard deviation (mm)</td>
<td>12.2</td>
<td>10.4</td>
<td>11.8</td>
</tr>
</tbody>
</table>

Table 5.2 TRL Compaction trial: Summary data

<table>
<thead>
<tr>
<th>Number of roller passes</th>
<th>Bulk density (Mg/m³)</th>
<th>Moisture content (%)</th>
<th>Dry density (Mg/m³)</th>
<th>Relative compaction (%)</th>
<th>Air voids (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1.887</td>
<td>4.7</td>
<td>1.803</td>
<td>95.3</td>
<td>17.7</td>
</tr>
<tr>
<td>4</td>
<td>1.954</td>
<td>4.8</td>
<td>1.864</td>
<td>98.5</td>
<td>14.6</td>
</tr>
<tr>
<td>6</td>
<td>2.010</td>
<td>5.2</td>
<td>1.925</td>
<td>101.8</td>
<td>11.1</td>
</tr>
<tr>
<td>8</td>
<td>2.044*</td>
<td>4.9*</td>
<td>1.948*</td>
<td>103.0*</td>
<td>10.6*</td>
</tr>
</tbody>
</table>

* Sample size reduced by removing 2 outliers.

Relative compactions typically range from 85% to 105% for loose to well compacted crushed aggregates (Parsons, 1992). As such, it is not uncommon for field density data to be in excess of laboratory MDD data (i.e. to achieve relative compaction values in excess of 100%), as the compactive effort in the field is typically applied in a less controlled environment.

All six dry density data sets are presented in Figure 5.2. As shown from the free-hand best fit curve, and confirmed from the air void contents in Table 5.1, the state of compaction of the Type 4 sub-base was improved considerably between 2 and 6 machine passes (the mean air void content was reduced from 17.7 to 11.1%). Between 6 and 8 passes, the improvement was greatly reduced (the mean air void content was only reduced from 11.1 to 10.6%). Accordingly, it is likely that there would be no further significant improvement in the state of compaction for further passes of the specific roller used in the trial.

In terms of moisture content, the Type 4 sub-base used in this trial was placed on a firm subgrade and compacted typically at 5% moisture content, i.e. within 1% of OMC. As such, the states of compaction achieved for the Type 4 in this trial should be viewed as approaching the upper end of those achievable for the specified underlying site conditions and the specific compactive effort afforded by the tandem roller. Conversely, where weaker subgrades are encountered, and Type 4 sub-bases are compacted at moisture contents approaching the lower limit specified (OMC - 2%), poorer states of compaction will be produced. However, even with these more difficult conditions, it is still considered likely that relative compactions around 100% would be achievable for 8 machine passes.

In general terms, the TRL compaction trial supported the view that the current Method Specification is appropriate for the compaction of Type 4 sub-base, when construction is in accordance with the requirements of the SHW (MCHW 1).

![Figure 5.2 TRL compaction trial: \textit{In situ} dry densities](image-url)
6 Full-scale use of Type 4 sub-base

6.1 A2/M2 Cobham
The A2/M2 Cobham widening in Kent, carried out under a design and build contract by a Costain-Skaneka-Mowlem joint venture (CSMjv), was the first full-scale use of asphalt arisings as Type 4 sub-base in the UK. The total length of the scheme was 17km and work commenced in November 1999. The project encompassed the construction of a new London-bound carriageway and the reconstruction of existing carriageways to form a new coast-bound carriageway. The coast-bound works initially involved the removal of redundant sections of A2/M2 carriageway. Planing operations produced site-won asphalt arisings, which were used in construction phases as Class 6F3 capping and Type 4 sub-base. A CSMjv Type 4 sub-base site trafficking trial was undertaken at the site in November 2001. Data originally reported by Davies and Boulton (2001) and Boulton (2002) are reproduced in the following text.

6.1.1 Sub-base materials and construction
The materials used in the trafficking trial consisted of a Type 1 and a Type 4 sub-base. The Type 1 control material was crushed granite imported from Jelsa Quarry, Norway, and was supplied in accordance with SHW Clause 803 (MCHW 1). The Type 4 material comprised site-won asphalt arisings.

The formation on which the trafficking trial sections were constructed consisted of good quality, intact chalk. The sections were constructed adjacent to each other in a length of mainline carriageway known as Victoria Close, between chainages 10740 to 10810. The layout of the trial sections is presented in Figure 6.1. In total, a plan area 70m long by 19m width was made available for the trial. A width of 9.5m, corresponding to the hard shoulder (safety lane) and Lane 1 of the new London-bound carriageway, was selected for the Type 4 sub-base section and the remaining 9.5m, corresponding to Lanes 2 and 3, selected for Type 1 sub-base section.

The stockpiled asphalt arisings were delivered via tipper trucks and were spread in an approximately 170mm thick loose layer by a 360° tracked excavator. In accordance with the Method Specification (Table 8/1 of the SHW), the material was compacted with five passes, using a Bomag 161 AD twin-drum vibrating roller, which has a mass per metre width of 2680 kg. The same plant and procedure was used to place and compact the Type 1 sub-base control section. The mean temperature of the asphalt arisings during construction was 9.2°C and the mean air temperature was 10.2°C. Once construction of the demonstration area had been completed, all site traffic was prohibited from trafficking the trial areas.

![Diagram of A2/M2 Cobham trial](Figure 6.1 Layout of A2/M2 Cobham trial)
6.1.2 Particle size distribution

Two particle size distributions of the Type 4 asphalt arisings used in the trial, undertaken by the CSMjv laboratory in accordance with BS 812: Part 103 (1985), are presented in Figure 6.2. Both gradings were located towards the fine limit of the Type 4 grading envelope. There was a slight non-compliance on the 20mm particle size for one of the samples.

6.1.3 Compaction data

The compaction requirements for Type 4 sub-base are based on Method Specification, which does not rely on in situ acceptability testing to be undertaken on completion of compaction. However, in order to validate the states of compaction achieved using Method Specification, it was decided that, for both trial sections, six sand replacement density tests would be performed. These tests were carried out by CSMjv laboratory staff, in accordance with the procedure specified in BS 1377 (BSI, 1990). Test data are presented in Table 6.1 for both the Type 4 and Type 1 trial sections.

Prior to the demonstration, various samples of asphalt arisings were taken for laboratory compaction testing. The tests were undertaken following BS 5835: Part 1 (1980) or BS 1377: Part 4: Method 3.7 (1990). The moisture contents of all test samples were determined using an oven set between 45 and 50°C. The resulting data are presented in Table 6.2; materials identified as A, B or C relate to asphalt arisings used for the trial area, whilst materials D and E relate to asphalt arisings mixed with ballast or Type 1.

The compaction data for the trial area asphalt arisings, presented in Figure 6.3, show that the dry density versus moisture content plots did not always exhibit the classical peaks that enable MDD and associated OMC to be readily specified. This is often the case for asphalt planings and has been reported previously (Toombs et al., 1994; MacNeil et al., 1997; MacNeil and Steele, 2002). In the absence of classical peaks, best fit straight lines are used, and the point at which the regression line intersects the 5% air voids line can be taken to be the optimum for that particular material. If 5% air voids cannot be attained, then the intersection with the 10% air voids line is used.

According to MacNeil et al. (1997), typical MDD values for asphalt arisings range between 1.860 to 2.005Mg/m³ and the corresponding OMC values range between 5.0% and 8.0%. Typical particle density values range between 2.35Mg/m³ and 2.46Mg/m³. The site data from the A2/M2 trial (Table 6.1) indicate agreement with these ranges.

<table>
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<th>Sub-base material</th>
<th>Chainage (m)</th>
<th>Offset (m)</th>
<th>Moisture content (%)</th>
<th>Dry density (Mg/m³)</th>
<th>Relative compaction (%)</th>
<th>Air void content (%)</th>
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</table>

Type 4: Maximum dry density = 1.955Mg/m³, Particle density = 2.42 Mg/m³
Type 1: Maximum dry density = 2.220Mg/m³, Particle density = 2.68 Mg/m³

Table 6.1 A2/M2 Cobham: Sand replacement data for Type 4 and Type 1 sections

Figure 6.2 A2/M2 Cobham: Particle size distribution for asphalt arisings
Davies and Boulton (2001) also reported that the overall mean OMC and MDD values, achieved in accordance with BS 5835 (BSI, 1980), for the trial and for further works using asphalt arisings blended with other materials, was 6.0% and 2.013 Mg/m³ respectively. It should be noted that, compared to the compaction data for unblended asphalt arisings, the effect of blending high quality crushed rock aggregates with asphalt arisings would be to increase the MDD without significantly altering the OMC.

According to the Method Specification, the moisture content requirement for Type 4 sub-base is OMC to OMC minus 2%. Accepting an OMC of 5.8% (based on Sample A data and the overall average), results in the majority of the sand replacement data (see Table 6.1 and Figure 6.3) falling outside this acceptability range. A number of aspects could contribute to this, including insufficient or non-uniform water addition prior to compaction, drying out of the material prior to sampling, or sampling errors due to the coarse grained nature of the material allowing moisture to drain away. As such, it is possible that the measured moisture contents may have been slightly less than the true in situ moisture contents at the time of compaction. As a consequence of this, for the full scale works, Davies and Boulton (2001) recommended following the method advocated by Berg et al. (1992) and Yuille (1996). This rapid and reliable method uses the fact that an approximation to the oven dried moisture content can be obtained by subtracting the bitumen content from the NDG moisture content of asphalt planings (the over reading of moisture content is due to the presence of hydrogen molecules in bitumen binders).

### 6.1.4 Assessment of formation

As part of the CSMJV Specification for the works, it was a requirement that the surface CBR of the foundation was greater than 32% for locations where sub-base was laid directly on chalk that has a surface CBR in excess of 8%. As part of routine testing for the project, Plate Bearing Tests (PBT) were undertaken to determine equivalent CBR values for the formation of the trial area prior to laying sub-base. The results of all PBT tests indicated that the equivalent CBR values for the formation were greater than 50%.

### 6.1.5 Assessment of sub-base layers

PBT tests were also performed on the as laid sub-base materials, and were undertaken on completion of compaction and prior to trafficking. The results of these PBTs (Davies and Boulton, 2001) are presented in Table 6.3 for both the Type 4 and Type 1 sections. All PBTs gave

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**Table 6.2 A2/M2 Cobham: Laboratory compaction data**

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<th>Test</th>
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<th>Moisture content (%)</th>
<th>Dry density (Mg/m³)</th>
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back-calculated sub-base CBR values in excess of 50%, indicating that both the Type 4 and Type 1 site trial sections satisfied the CSMjv Specification requirement. The mean moduli of surface reaction were 366.5kN/m² for the Type 4 sub-base section and 465.3kN/m² for the Type 1 sub-base section, though the exclusion of test A515 data reduces the Type 1 mean to a more representative value of 405.2kN/m². The difference between the Type 1 and Type 4 mean values may have been due to difference in foundation stiffness in the trial sections, or reflect the different response of asphalt arisings under loading. Toombs et al. (1994) discussed examples of poor correlations between PBT and CBR, which were believed to be due to the viscous nature of asphalt arisings and different loading duration for a PBT compared to a CBR test. During a PBT, the asphalt arisings would be under load for 40 to 50 minutes, whereas for a CBR test, loading duration would be less than 10 minutes. It might therefore be expected that much more plastic deformation would occur during a PBT compared to a CBR test. For the A2/M2 trial, it is also reasonable to assume that, had the surface temperatures been significantly higher, such as 30°C compared to less than 10°C measured on site, the reported PBT values may have been considerably lower and conceivably outside the CSMjv lower limit of 32% for sub-

**Figure 6.3** A2/M2 Cobham: Density data for Type 4 demonstration area
base. As a result of the temperature dependent nature of asphalt arisings, TRL recommends that appropriate caution be used when specifying bearing capacity requirements.

TRL undertook portable Prima FWD tests on the compacted surfaces of both of the trial sub-base sections. The resulting surface modulus data are presented in Table 6.4. For the Type 4 sub-base section, the mean surface modulus was 202.3 MPa, whilst the mean surface modulus for the Type 1 sub-base section was 125.5 MPa (of note, there are considerable variations in both sets of data). It was concluded that whilst the Type 4 sub-base section was stiffer than Type 1 sub-base section, the extent of the difference would not significantly impact on the validity of the trafficking trial.

### 6.1.6 Trafficking trial

The trafficking trial was conducted using a loaded 4-axle tipper truck, which had a gross vehicle weight of 31.14 tonnes. As it was not possible to weigh all four axles individually using a weighbridge, the front two and the rear two axles were measured separately. The mass of the front and rear axles was 11.76 and 19.38 tonnes respectively. These were converted to an equivalent number of standard 80kN axles using the fourth power wear law (Liddle, 1962). Each pass applied the equivalent of 4.527 standard axles.

During trafficking, the lorry was restricted to canalised wheel paths. To assist the lorry driver, the outer edges of the offside rear tyre paths were marked on the compacted sub-base over the length of each trial section. Each trial length consisted of 50m designated as the trafficking trial area, with an additional 10m at each end designated as a run off area. Before trafficking commenced, a grid for measuring the deformation induced by the loaded lorry was marked on the sub-base. The grid comprised five points at 10m intervals on each of the rear tyre-paths. A survey of initial surface levels was carried out prior to trafficking. The survey was then repeated after 50, 100, 150, 200 and 225 passes. The trafficking trial results are presented in Figure 6.4 and Table 6.5 for both the Type 4 and the Type 1 trial area. At the equivalent of 1019 standard axles, the mean deformations for the Type 4 and Type 1 sections were 7.8 and 5.8 mm respectively.

### 6.1.7 Discussion

The asphalt planings used in the A2/M2 Cobham site trial easily met the Specification requirement (MCHW 1) that trafficking of Type 4 sub-base should result in mean vertical deformation of less than 30 mm after 1000 standard axles. The site trial also highlighted that the Type 4 sub-base performed in a broadly comparable manner to the control Type 1 sub-base. Due to full compliance with the Specification, Type 4 sub-base was used extensively throughout the works.

### 7 Use of asphalt arisings

The nature and behaviour of asphalt arisings differ from conventional crushed rock aggregates used in highway construction. For this reason, most suitability and acceptability criteria for the conventional materials, and the techniques used to generate such data, required modification prior to their application for asphalt arisings. As part of this process, the research carried out by TRL over the past decade has evaluated the performance of asphalt arisings in applications encompassing sub-base, capping and structural fill. In general terms, the research has concluded that, subject to certain limitations, there are no technical reasons why asphalt arisings could not be used in sub-base and capping applications. Based on these findings, Class 6F3 capping and Type 4 sub-base have now been implemented in the current version of the SHW (MCHW 1).

Asphalt arisings can be produced from a wide variety of bituminous source materials, which in turn can impact on
the quality and performance of a foundation constructed using asphalt arisings. Factors that influence the engineering properties of asphalt arisings include:

- the method of planing (including temperature);
- aggregate characteristics (type, grading, effective particle size, particle shape, texture and hardness etc.);
- bitumen binder characteristics (quantity, chemical composition, aging via oxidation, loss of volatiles and hardening).

The performance of the bitumen binder with respect to temperature is a particularly significant aspect in the performance of asphalt arisings. At lower compaction temperatures, corresponding to lower viscosities of the bitumen binders, considerably more friction may be generated between the aggregate particles. This may limit the effectiveness of applied compactive effort, which, in turn, may contribute to an excessive air void content, a high compressibility potential and poor load bearing ability of the compacted layer.

Many of the engineering problems faced when producing, specifying and using asphalt arisings were directly addressed in the Specification development and validation work described in this report. Relevant findings on a number of these topics are summarised in the following sections.

7.1 Production

7.1.1 Planing

Asphalt arisings consist of materials derived from the asphalt layers of the pavement, which are produced using a mobile machine fitted with milling cutters. The quality of the asphalt arisings is therefore partially dependent on the effectiveness of the planing operation, in terms of producing material with the correct particle size distribution. In general, the overall grading usually lies within the limits set for capping and sub-base materials in the SHW (MCHW 1). However, problems can occur with ‘slabbing’ of material during the initial pass, which can lead to large lumps and slabs of material being produced. Such excessively sized particles (often greater than 100mm) need to be removed or further processed before the asphalt arisings can be used.

7.1.2 Bitumen content

In terms of compaction and strength (CBR), early research carried out by Toombs et al. (1994) found no particular effect of bitumen on the performance of planings materials. However, at the time, it was considered that high bitumen contents might occur with granulated asphalt that contained reject mixes or waste materials from the end of a batch. To ensure that this did not occur, the authors recommended an upper limit on bitumen content of 10% for Class 6F3 capping materials in Series 600 of the SHW (MCHW 1). As bitumen contents ranging up to 8% were also found to limit the effectiveness of set laboratory compactive efforts, it was also recommended that the Specification requirements limit the compacted layer depth of Class 6F3 to 200mm.
Following on from this early research, MacNeil and Steele (2002) subsequently identified a lower bitumen content of 2% which would enable granular mixtures containing bituminous planings to be classified into a new capping category, namely Class 6F4. Through extensive laboratory and pilot-scale testing, it was found that granular mixtures with bitumen contents below 2% performed like conventional granular materials. Due to the reduction in maximum bitumen content proposed for Class 6F4, it was recommended that the compacted layer depth restriction of 200mm need not apply to the proposed Class 6F4.

7.1.3 Environmental considerations
Environmental considerations regarding the use of asphalt arisings were reported by MacNeil et al. (1997). Health concerns with asphalt and tarmacadam related to the content of polycyclic aromatic hydrocarbons (PAHs, some of which are carcinogenic) and metals. A review of published research had shown that problems with leachates occurred only if tar is present in asphalt arisings. As tar contains significant amounts of carcinogenic PAHs, the Specification requirements for both Class 6F3 and Type 4 sub-base do not permit materials with any tar or tar bitumen content.

Whilst tar is used in significant amounts in Europe, the level of tar used in pavement construction in the UK is believed to be minimal. The amount of tar bound pavement materials used since the mid-1970s on UK motorways and trunk roads is probably less than 0.1%, and should not therefore cause a pollution problem in the use of recycled planings (Toombs et al., 1994). Because of their resistance to oil, small volumes of tar binders may still be used in paving vehicle lay-bys. MacNeil et al. (1997) reported on the analysis of contaminants and PAH content of four sources of bituminous planings materials used in previous TRL research. The results showed that the levels of PAH contents and contaminants, apart from the slightly higher cadmium contents in two samples, fell well below the DOE Interdepartmental Committee for the Reclamation of Contaminated Land (DoE, 1987) threshold trigger concentration values, and therefore should not present a contaminant problem.

7.2 Laboratory and site testing

7.2.1 Moisture content
Normal methods of moisture content determination for soils and granular materials involve drying samples in an oven at between 105 and 110°C for a period of up to 24 hours. For asphalt arisings, this method is impractical, as bitumen has a low melting point, and volatile constituents could be potentially lost, resulting in an inaccurate measurement of moisture content. Experience has shown that, for asphalt arisings, moisture content determinations can be effectively carried out using an oven set at a reduced temperature of 45 to 50°C, or using a microwave oven on its lowest power setting. When using a microwave oven, it is recommended that trials be carried out to determine optimum sample size and drying time.

7.2.2 Determination of moisture content using a nuclear density gauge
In normal circumstances, the use of a NDG on site can be highly beneficial, as it permits the rapid determination of the moisture content of placed and compacted materials. However, the NDG is unable to record the moisture contents of bituminous materials accurately because the NDG’s moisture monitoring system relies on the slowing down of neutrons by the hydrogen molecules in water. As bitumen binders also contain hydrogen molecules, the NDG records erroneously high moisture contents, compared to the equivalent oven dried moisture contents. Research has shown that an approximation to the oven dried moisture content can be obtained by subtracting the bitumen content from the NDG moisture content of asphalt arisings (Berg et al., 1992 and Yuille, 1996).

For more reliable estimates of moisture content, undertaken as part of compliance checks, it is recommended that site/gauge specific calibrations are undertaken that relate bitumen content, oven dried moisture content and NDG moisture content for similar source material. If any doubt exists as to the accuracy of the NDG moisture content measurements, then standard laboratory determinations should be carried out.

7.2.3 Determination of density using a nuclear density gauge
Whilst not implicitly required in the Specification for Type 4 sub-base, densities are often measured during trafficking trials to permit comparisons with laboratory compaction data (relative compaction) and to permit the calculation of air void contents. Where an NDG is being used to measure bulk density, it should be noted that, for all soils, the recorded NDG bulk densities must be corrected using a field calibration (Troxler, 1984). The field calibration involves comparing sand replacement bulk densities with those measured by the NDG (typically for 5 discrete densities) and producing a material specific calibration. In situ NDG bulk densities can be corrected to equivalent sand replacement values, and then converted to dry densities using the moisture content of a sample adjacent to the test location.

7.2.4 Particle density
In general, no particular problems have been encountered with the measurement of particle density for asphalt arisings, when using the procedure specified in BS 1377: Part 2 Method 8.2 (BSI, 1990). Particle densities of asphalt arisings are typically within the range 2.35 to 2.47 Mg/m³, with the actual value being dependent on the constituent material types contained within the particular source.

7.2.5 Grading
The particle size distribution of asphalt arisings is mainly dependent on the grading of the aggregate within the planed layer, but is also partially dependent on the planing technique and equipment used for recovery. The majority of asphalt planings encountered during the TRL research fell within the grading envelopes detailed in the SHW
as the BS 5835 test is more complex and requires non-bituminous materials, is believed to give more developed specifically for graded aggregates and, for such (BSI, 1980). The BS 5835 vibrating hammer test was accordance with the vibrating hammer method in BS 5835 requirement is based on an OMC determined in accordance with the vibrating hammer (BSI, 1990). The BS 1377 test is commonly used in soil testing for 3.7 (vibrating hammer test) (BSI, 1990). The BS 1377 (BSI, 1990) As described previously in Section 6, dry density versus moisture content plots for asphalt arisings often do not exhibit the classical peak which enables a MDD and associated OMC to be assigned to these materials. Current advice is that in the absence of classical peaks, best fit straight lines may be used and the point at which the regression line intersects the 5% air voids line can be taken to be the optimum for that particular material or mixture. If 5% air voids cannot be attained, the intersection with the 10% air voids line may used.

On the basis of current TRL experience, typical values for MDD asphalt arisings range from about 1.860 Mg/m³ to 2.005Mg/m³ and the corresponding OMC values range between 5.0% to 8.0%. The actual level of MDD is partially dependent on the ratio and particle densities of the materials that make up the bulk sample. Typical particle density values for asphalt arisings range between 2.35Mg/m³ and 2.46Mg/m³.

7.3 Compaction requirements

The state of compaction achieved in a compacted soil or aggregate is a function of particle size distribution, particle characteristics (composition, angularity, hardness etc.), moisture content, compactive effort (compactor mass, speed, operating frequency etc.), layer thickness and underlying ground conditions (Parsons, 1992). For most material Classes within the SHW, a Method Specification will have been developed to ensure that appropriate states of compaction are achieved for different categories of compactors and layer thicknesses. Where a material is highly variable, or further site control is required, an End Performance Specification is implemented.

Compared to standard Type 1 aggregates, the bitumen component gives asphalt arisings unique compaction characteristics. In a relatively dry condition, interparticle friction is high and asphalt arisings therefore exhibit considerable resistance to compaction. This normally results in the compacted material having high air void contents, typically up to 20%.

The compaction requirements for Class 6F3 capping (see Table 6/4, MCHW 1) were developed primarily from the findings of UK capping trials undertaken in the early 1990s (Toombs et al., 1994). The research concluded that, due to the higher compactive effort being required to ensure adequate compaction at the bottom of the layer, a maximum compacted layer thickness of 200mm should be specified from Class 6F3 materials (the maximum layer thicknesses for other non-bituminous Classes of capping were, and continue to be, 250mm). As Class 6F3 and Type
4 materials are indistinguishable, it is recommended that the maximum compacted layer thickness of 200mm also be specified for Type 4 sub-base applications.

7.3.1 Combined layer thickness
Due to the nature of asphalt arisings, even when correctly compacted, Type 4 materials may have mean air void contents of 10 to 15% (well compacted Type 1 materials would be expected to have lower mean air void values). Air void contents of this magnitude are not normally considered to be too problematic for the thin layer applications, where bearing capacity improvements are being sought. However, for thicker layer applications, the combination of bitumen coated particles and high air void contents may result in mobilisation via a creep mechanism and produce settlement in the longer term. It is likely that the magnitude of the settlement will depend on the effect of the seasonal temperature cycle on the bitumen, the loading (due to static overburden and dynamic traffic loading) sustained by the sub-base and the combined thickness of the compacted asphalt arisings layer.

MacNeil et al. (1997) and MacNeil and Steele (2002) reported on investigations into the use of asphalt arisings in both laboratory compressibility tests (using a Rowe cell) and a full scale instrumented abutment. A summary of the settlement data is presented in Figure 7.1. For a Class 6F2 crushed rock capping (which was very similar to Type 1), the settlement ranged in a linear manner from 0 to 1% across the overburden range of 10 to 40kPa. At 20kPa, the settlement was around 0.3%. For the asphalt arisings, the lower and upper bounds on settlement were 0 to 1.5% at 10kPa, increasing to 2 to 5% at 40kPa. At 20kPa, the average settlement was around 3%, some 10 times greater than for the Type 1 equivalent. Whilst a 3% settlement may not be too excessive for a well compacted thin layer (e.g. 4.5mm settlement for a 150mm layer) of Type 4 sub-base at an overburden of 20kPa, higher settlements for thicker layers, with associated higher overburdens, have the potential to cause serious long-term problems. In order of severity, the problems identified with asphalt arisings placed in thick layers were the:

i. excessive lengths of settlement periods;
ii. considerable magnitudes of the associated settlement;
iii. possibility of differential settlements due to structural geometry;
iv. potential for further settlements when a structure is loaded;
v. maintenance of the creep mechanism by the seasonal temperature cycle.

In highway applications such as capping and sub-base, the overburdens are much lower than for structural fill, though the contribution from dynamic traffic loading to the total stress will be greater. The design of foundations are covered in HA 44 (DMRB 4.1.1) for capping and in HD 25/94 (DMRB 7.1.2) for capping and sub-base thickness designs. Following the guidance in HD 25/94, the potential combined thicknesses of Class 6F3 capping and Type 4 sub-base that could be formed using asphalt arisings range from 750mm for a weak subgrade below 2% CBR, to 150mm (no capping element required) for a subgrade of 20% CBR.

In terms of the design of foundations on weak subgrades, HD 25/94 advises:

*When a subgrade has a CBR sufficiently below 2% such that it becomes unsuitable as a pavement foundation, (a subgrade would tend to deform and ‘wave’ under construction traffic), then a number of options are available.*

---

Figure 7.1 Summary plot of settlement versus overburden
and:

The material can be removed and replaced by more suitable material; if the depth is small, all can be replaced but it may only be necessary to replace the top layer. The thickness removed will typically be between 0.5 and 1.0m. Although the new material may be of good quality, the subgrade should be assumed to be equivalent to one of a CBR value just under 2% (i.e. 600mm capping), in order to allow for movements in the soft underlying material. A total construction thickness about 1.5m thick will often result.

Should a pavement foundation constructed on such a weak subgrade comprise a total thickness of 1.5m of asphalt arising, the potential long-term settlement, within the arisings, would be approximately 60mm (assuming a 4% settlement for a 30kPa overburden on a 1.5m layer). Long-term settlements of this order would not normally be acceptable in highway construction in the UK. As such, it is recommended that, where asphalt arisings are used in combined layers greater than 750mm, or where concerns have been identified with combined thicknesses less than 750mm, an appropriate testing regime should be used to assess material compressibility characteristics. An engineering judgement, based on the material performance, would then be required to decide on material suitability.

7.4 Structural performance

7.4.1 Bearing capacity

In the past, plate bearing tests (PBTs) have been used to assess the in situ performance of asphalt arisings. Toombs et al. (1994) reported that a major problem with asphalt arisings was the wide variation in measured CBR values, which showed a dependency on the test method used. In particular, on some occasions there was found to be no agreement between the CBR as measured in situ using the DCP, the PBT or from a laboratory CBR determination.

The reason for this difference is that bitumen stiffness is very sensitive to both duration of loading and variations in temperature. Pell (1982) showed that the stiffness modulus of a typical asphalt basecourse mix is reduced by a factor of about 25 if the loading time is increased from 1 to 1000 seconds. Similarly, the stiffness of the mix at a temperature of 30°C is about 15 times less than at 15°C. During a PBT, the asphalt arisings are under load for a period of 40 to 50 minutes, whilst for a laboratory CBR determination, the period of load is approximately 8 minutes. During a DCP test, the loading period is the duration of a blow, which is virtually instantaneous. It might therefore be expected that much more plastic deformation of the test sample would occur during a PBT than during a laboratory CBR or a DCP test. For this reason, CBR values derived from a PBT would be lower than those obtained from the laboratory CBR test or derived from the DCP test. Following the same reasoning, laboratory CBRs might be expected to be lower than the DCP derived values.

The Kleyn DCP/CRB correlations (1975) for cohesive soils and granular soils do not readily apply to asphalt arisings, and the difference between the Kleyn relation and the asphalt arisings relation can generate considerable errors in the estimation of CBR from DCP testing. Both Toombs et al. (1994) and MacNeil and Steele (2002) highlighted the differences (see Figure 2.10). For example, a DCP value of 10mm per blow is equivalent to a CBR of 18% using the Kleyn calibration whereas the asphalt arisings calibrations give an equivalent CBR that ranges from 4.5 to 8.5%. Whilst such low CBR values would suggest that asphalt arisings are unsuitable for use as capping (where the target CBR is 15%, DMRB 4.1.1) or sub-base (where the inferred SHW minimum is 30% CBR), it may be more appropriate to think in terms of resistance to penetration. A DCP value of 14mm per blow gives an equivalent Kleyn CBR of 15%, and it is likely that such a DCP value for asphalt arisings will also result in acceptable bearing capacity, though this should not be relied upon without further corroboration. Additionally, the temperature dependent nature of the performance of asphalt arisings may also serve to increase the discrepancy between the Kleyn and the asphalt arisings calibrations, especially when surface temperatures are high.

In general, care must be taken when specifying and interpreting PBT/DCP/CRB data associated with asphalt arisings. Furthermore, when undertaking a comparison of bearing capacity between conventional Type 1 and Type 4 sub-base materials, the contribution of the visco elastic behaviour of asphalt arisings must be recognised.

7.5 Trafficability

According to HD 25/94 (DMRB 7.1.2), the main purpose of the foundation is to distribute the applied vehicle loads to the underlying subgrade, without causing distress in the foundation layers or in the overlying layers. This is required both during construction and during the service life of the pavement. Following the Earland and Pike (1985) recommendation that a mean deformation value of 30mm after 1000 standard axles is an appropriate measure of sub-base stability, the Draft Specification and the current SHW Notes for Guidance require that a controlled trafficking is used to determine trafficability. A convenient test vehicle is a 3-axle tipper lorry loaded to a gross mass of 24 tonnes (1 pass is equivalent to 3 standard axles), though other equivalent standard axle loads may be readily calculated (Liddle, 1962). The selection of the trafficking vehicle should reflect actual site conditions. During the trial, the trafficking vehicle is restricted to canalised wheelpaths and the deformations from initial levels are measured at various points as the trial progresses. Due to the asymptotic nature of the collective passes versus deformation plot, it is worth gathering data at regular intervals early on in the sequence. To have full confidence in the mean level of deformation, it is often useful to continue beyond 1000 standard axles.

In terms of resistance to deformation, the performance of sub-base materials is not only dependent on the material properties, but also on the characteristics of the formation on which the material is placed. Accordingly, when comparing the performance of a Type 4 sub-base to that of a Type 1 sub-base, it is important to select, within the limitations of site operations, trial areas that have comparable characteristics in terms of material composition and strength or stiffness.
Research over the past decade, the finding of which are now implemented in the UK’s national Specification for Highway Works, has shown that Type 4 asphalt arising sub-bases perform in a manner comparable to conventional Type 1 granular sub-bases. The finding of the many different studies used to develop the Specification for Type 4 are reported and summarised in this document.

The principal conclusions are as follows:

1. Extensive trials have supported the view that a mean deformation of less than 30mm for trafficking to 1000 standard axles is a suitable performance criterion for Type 4 sub-base.
2. In terms of resistance to deformation, the performance of all sub-base materials is not only dependent on the material properties, but also on the characteristics of the formation on which the material is placed.
3. On the basis of TRL experience, typical values for Type 4 Maximum Dry Density range from 1.860 Mg/m$^3$ to 2.005Mg/m$^3$ and the corresponding Optimum Moisture Content values range between 5.0% and 8.0%. Typical particle density values for asphalt arisings range between 2.35Mg/m$^3$ and 2.46Mg/m$^3$.
4. Type 4 sub-base, compacted in accordance with the Method Specification, may have mean in situ air void contents ranging from 10 to 15%, whilst Type 1 sub-bases are expected to have mean in situ air void contents of less than 10%. However, the Method Specification for both Type 1 and Type 4 materials has been shown to be appropriate and no major amendments are required. A 200mm maximum compacted layer thickness is recommended for Type 4 sub-base applications, and the Specification should be amended accordingly.
5. Advice is presented on test procedures. In particular the advice addresses preparation for particle size distribution samples, and the interpretation of optimum moisture content data from the non standard compaction curves that are frequently generated for Type 4 materials.
6. Further advice considers when asphalt arisings are used in thick layers, typically greater than 750mm. In such circumstances, asphalt arisings may exhibit compressibility problems that may result in large long-term settlements. Where potential settlement concerns are identified, an appropriate testing regime should be used to assess material compressibility characteristics.
7. Laboratory calibrations of DCP versus CBR can be significantly different for asphalt arisings and conventional granular materials. Accordingly, care must be taken when specifying and interpreting bearing capacity data (PBT, CBR, and DCP) for materials with a visco elastic bituminous component.


HA44 Earthworks: Design and preparation of contract documents. (DMRB 4.1.1).  


Draft specification ASPHALT ARISINGS AS SUB-BASE

The specification below is based on the clauses of Series 800 of the Specification for Highway Works. The following amendments/additions to the 800 Series are proposed for use in the trials.

# 801 Unbound materials for sub-bases
i Amend Sub-Clause 3 to limit the thickness of asphalt arisings that may be spread in one layer so that after compaction the maximum total thickness permitted is 200mm. The minimum compacted layer thickness shall remain as 110mm.
ii Add Sub-Clause 4 to limit the total compacted thickness of asphalt arisings that shall be permitted in the construction of the combined capping and sub-base to 600mm.

# 802 Compaction
No change

# 803 Granular sub-base material Type 1
No change

# 804 Granular sub-base material Type 2
No change

Add Clause:

# 805 Granular sub-base material Type 4

i Type 4 granular sub-base material shall be derived from asphalt arisings. The asphalt arisings shall be either asphalt road planings or granulated asphalt, but excluding materials containing tar or tar-bitumen binders. Asphalt planings are defined as materials derived from the asphalt layers of the pavement using a mobile machine fitted with milling cutters.
Granulated asphalt is defined as asphalt bound material recycled from roads under reconstruction or surplus asphalt material destined for bound pavement layers, but unused, which has been granulated.
Type 4 granular sub-base material shall have an upper limit on recovered bitumen content of 10% when tested in accordance with BS 598: Part 102.
Type 4 granular sub-base material shall, at the time of placing, lie within the lump size grading envelope of Table 8/4 and not be gap graded.

<table>
<thead>
<tr>
<th>BS sieve size</th>
<th>Percentage by mass passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>75 mm</td>
<td>100</td>
</tr>
<tr>
<td>37.5 mm</td>
<td>85-100</td>
</tr>
<tr>
<td>20 mm</td>
<td>60-90</td>
</tr>
<tr>
<td>10 mm</td>
<td>30-70</td>
</tr>
<tr>
<td>5 mm</td>
<td>15-45</td>
</tr>
<tr>
<td>600 micron</td>
<td>0-22</td>
</tr>
<tr>
<td>75 micron</td>
<td>0-10</td>
</tr>
</tbody>
</table>

The lump size distribution shall be determined either by the washing and sieving method or by the dry sieving method of BS 812: Part 103:1985 (see Note 1).

Note 1: The planings should be oven dried at a temperature of 45 to 50°C to reduce the tendency of the bitumen to soften and particles to adhere to each other.
2 The material shall be transported, laid and compacted at a moisture content within the range optimum moisture content to 2% below the optimum moisture content determined in compliance with BS 5835 and without drying out or segregation.

Measurement of moisture content both for control purposes and for optimum moisture content determination shall be according to BS 812 Part 109 using a conventional oven (Paragraph 6) on a reduced temperature setting of 45 to 50°C; or by using a micro-wave oven (Paragraph 7.3) provided that the lowest power setting is employed and that trials are carried out to determine the optimum sample size and drying time.

3 When required by Appendix 7/1, the Contractor shall construct a trial area incorporating the Type 4 granular sub-base material proposed for use in the Works. The trial area shall be constructed, trafficked and assessed in accordance with the procedure described in Clause NG805 of the Notes for Guidance on the Specification for Highway Works, Road Pavements - Unbound Materials, Series NG 800. The mean vertical deformation after 1000 standard axles shall be less than 30mm when measured in accordance with the stated procedure.

4 A brief performance report on the behaviour of the Type 4 granular sub-base is required.

Publications referred to in the Specification

BS 598  Sampling and examination of bituminous mixtures for roads and other paved areas
        Part 102  Analytical test methods
BS 812  Testing aggregates
        Part 109  Methods for determination of moisture content
BS 5835 Recommendations for testing aggregates
        Part 1  Compactibility test for graded aggregates
Draft Notes for Guidance ASPHALT ARISINGS AS SUB-BASE

The notes for guidance below are based on the Clauses of the Notes for Guidance on the Specification for Highway Works, Road Pavements - Unbound Materials, Series NG 800.
The following amendments/additions to these Notes for Guidance are proposed for use in the trials.

#NG 801 Unbound materials for sub-bases
i Add Type 4 granular sub-base as a permitted material.
ii Research has shown that compaction of Type 4 granular sub-base materials, by application of the current best practices can result in mean, in situ air voids contents of up to 10%. Such levels of air voids could lead to settlement of excessively thick layers of Type 4 granular sub-base material. By limiting the total thickness of unbound recycled asphalt material permitted in the combined capping (6F3) and the sub-base layer (Type 4 granular sub-base) to 600mm, it is considered that settlement will not be a problem.

Add:

#NG 805 Granular sub-base material Type 4

General
Trafficking trials of Type 4 granular sub-base material carried out by TRL have produced rut-depths well within the upper recommended limit of 30mm.

Any unusual behaviour of the laid material under construction plant should be investigated and, if considered necessary, the Engineer should carefully examine the Contractor’s laying and compaction methods.

Guidance on the protection of the subgrade and sub-base is already given in NG 704.
No limiting traffic design has been imposed for Type 4 granular sub-base material complying with Clause 805.

Transport and laying
1 When dry, Type 4 granular sub-base materials exhibit a considerable resistance to compaction due to the friction of the bitumen coating. The addition of water has a significant effect on the state of compaction by reducing the friction between the bitumen coated particles.

It has therefore been specified that Type 4 granular sub-base materials should be compacted at moisture contents close to the optimum as determined by the BS 5835 method. The temperature at which the optimum moisture content (OMC) was determined should also be reported.

The test procedure for the determination of the OMC in compliance with BS 5835 has been developed specifically for graded aggregates and gives more reproducible results than the vibrating hammer test of BS 1377: Part 4 for these materials.

Material properties
1 The particle size distribution of asphalt arisings is best described by the term ‘lump size distribution’ because of the binding effect of bitumen. The grading envelope obtained will be dependent on the duration of shaking, the temperature at which the determination is carried out as well as the grading of the mineral particles within the asphalt arisings.

Agglomeration of lumps can occur in stockpiled material especially in hot weather or when the material is stored for long periods. It is important that, at the time of placing, the asphalt arisings comply with the specified lump size distribution and care should be taken to ensure that material taken from a stockpile is to the required grading.

2 Particle durability in terms of the soundness test (BS 812: Part 121) has not been specified as the aggregates will have been tested prior to the introduction of bitumen.

3 Particle hardness in terms of the ten per cent fines test (BS 812: Part 111) has not been specified as the test is unsuitable for materials containing bitumen and because the aggregate components will have been tested prior to the introduction of bitumen.

4 The performance of unbound granular sub-bases is dependent on the bearing strength of the compacted material. The measurement of bearing capacity in terms of CBR has not been specified for Type 4 granular sub-base material. The measurement of CBR on the Type 4 granular sub-base materials containing bitumen is problematical because the results are dependent upon the temperature at the time of compaction, the
temperature at the time of testing and the duration of loading. However, as the grading envelope precludes material with less than 10% of the material retained on the 20mm sieve, it can be assumed without test that the material will have an adequate CBR value.

**Trafficking trial procedure**

**Location**

1. The trial area shall be located on suitable prepared sub-formation compacted in accordance with the Specification. The trial area may be located so that it can be incorporated within the Permanent Works if the resistance to wheeltrack rutting is demonstrated to the satisfaction of the Engineer.

2. The trial area shall be not be less than the 60 metres long and of sufficient width to allow the sub-base to be trafficked by a suitable vehicle.

**Materials**

1. If required within the Permanent Works, suitable capping layer material in sufficient quantity shall be provided to construct a platform approximately 55 metres long and 2 metres wider than the overlying sub-base layer. The capping layer shall be compacted to the thickness required in the contract.

2. The Type 4 granular sub-base material complying with Clause 805 shall be provided in sufficient quantity to construct a trial area approximately 50 metres long and having a sufficient base width that when trafficked the wheelpaths of the lorry shall be at least 1 metre from either edge of the platform of sub-base material. The sub-base material shall compacted to the thickness specified in the contract.

**Placement**

1. The materials shall be placed and compacted using the equipment proposed for use in the works.

2. The surface temperature and the temperature 100mm below the surface shall be recorded at the time of compaction.

3. If required in the contract, the capping layer material shall be placed and compacted in accordance with Table 6/4 Method 6 of the Specification for Highway Works.

4. The trial area shall be ramped at each end and rigid beams (wooden sleepers or similar) shall be incorporated into each end of the area for a distance of about 5 metres and shall have their upper faces level with the surface of the compacted Type 4 granular sub-base material. This will assist correct tracking by the test vehicle and minimise dynamic effects of the vehicle bouncing on its springs.

**Trafficking**

1. A convenient test vehicle is a 3-axle tipper lorry loaded to a gross mass of 24 tonnes (1 pass is equivalent to 3 standard axles). The selection of the test vehicle however should reflect actual site conditions and the equivalent standard axle load should be calculated for monitoring.

2. Longitudinal string lines shall be laid out on the surface of the trial area to help the driver maintain the same track on each pass and to achieve channelled rutting. Vertical deformation shall be measured in all wheeltracks using optical or laser levels at pre-determined monitoring points on five transverse lines spaced equally along the length of the trial bay. The transverse lines at the ends of the trial area shall be at least five metres from the rigid end beams. Monitoring vertical deformation should be carried out after 5,15,50,100,180 and 350 passes. The mean vertical deformation thus measured shall be plotted against the respective number of passes and the vertical deformation corresponding to 1000 standard axles shall be extrapolated.

3. The surface temperature and the temperature 100mm below the surface shall be monitored and recorded at regular intervals throughout the duration of the trafficking.

**Publications referred to in the Notes for Guidance**

- **BS 812** Testing aggregates
  - Part 111 Methods for determination of ten per cent fines value (TFV)
  - Part 121 Method for determination of soundness

- **BS 5835** Recommendations for testing aggregates
  - Part 1 Compactibility test for graded aggregates

- **BS 1377** Methods of test for soils for civil engineering purposes
  - Part 4 Compaction-related tests
Abstract

As a result of increasing demand for primary aggregates, coupled with an escalating environmental awareness, the UK Government is committed to promoting more efficient use of natural resources and secondary aggregates. Part of this strategy involves developing methodologies for enhancing the effective use of substitutes for primary aggregates, such as industrial waste, by-products and recycled materials. One such area is the use of recycled materials in sub-base layers. Previous TRL studies have led to the introduction of recycled asphalt arisings as Type 4 sub-base in the Specification for Highway Works. This report summarises TRL research carried out on these types of materials over the past twelve years. In total, seven sub-base trials are reported, encompassing three initial specification development trials, two specification validation trials and a small compaction trial. Data from the first full-scale site use of asphalt arisings as Type 4 sub-base in the UK are also reported. Other relevant data and advice on the use of asphalt arisings in sub-base, capping and structural fill applications are also presented and discussed. It was concluded that Type 4 asphalt arising sub-bases perform in a manner comparable to conventional Type 1 granular sub-bases. The current Specification requirements for both Type 1 and Type 4 materials has been shown to be appropriate and no major amendments are proposed, although a 200mm maximum compacted layer thickness is recommended for Type 4 sub-base applications.

Related publications

TRL523 Granular and bituminous planings mixtures for capping by MacNeil D J and Steele D P. 2002 (price £40, code JX)
TRL517 Compendium of selected research in ground engineering by Lewis J D. 2001 (price £25, code E)
TRL473 In-service performance of recycled asphalt roadbase by Megan M A and Potter J F. 2000 (price £25, code E)
RR64 Stability of gravel sub-bases by Earland, M and Pike D C. 1985 (price £20, code B)
LR901 The strength of clay subgrades: its measurement by a penetrometer by Black W P M. 1979 (price £20)
LR1132 The structural design of bituminous roads by Powell W D, Potter J P, Mayhew H C and Nunn M E. 1984 (price £20, code A)
Overseas Road Note 8 A users manual for a program to analyse dynamic cone penetrometer data by Transport and Road Research Laboratory. 1990 (price £10)

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