STRUCTURAL DESIGN AND PILOT-SCALE TRIALS OF MODULAR CELLULAR PAVING FOR GREENED SAFE HAVENS AND EMERGENCY ACCESS ROUTES

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STRUCTURAL DESIGN AND PILOT-SCALE TRIALS OF MODULAR CELLULAR PAVING FOR GREENED SAFE HAVENS AND EMERGENCY ACCESS ROUTES

FINAL REPORT

Project Reference – Task 377_2 Trafficking of hardened shoulders and safe havens

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Executive summary

In order to reduce congestion on the motorway network, the Highways Agency (HA) aims to create additional road space without road widening by turning some hard shoulders into extra traffic lanes. This objective raises safety concerns associated with the parking of broken down vehicles that would need to be left on strengthened verges or towed to purpose built safe havens.

In a parallel initiative, the HA aims to improve emergency access to, and egress from, the trunk road network through the provision of emergency turnaround areas (ETAs) in the road verge and other escape routes. For this purpose, grassed, cellular pavers are considered to be both cost effective and useful in providing a less conspicuous refuge to unauthorised users.

Following a previous study of methods and products that might be used to strengthening motorway grass verges, the HA commissioned TRL in 2004, under the Geotechnical Framework for Consultancy Services via the ARUP/TRL/BRE partnership, to research modular, grassed systems, which could be used to provide hardened verges, hard standings and low volume trafficked routes. This Final Report describes the main activities and outcomes of the study, which can be summarised as follows:

1. Review of modular, cellular pavers and the identification of six systems that were then assessed in laboratory tests.
2. Prediction of the design traffic and the determination of the maximum static load applied by propped trailers prior to the design of a pilot-scale trial in the TRL’ s Pavement Test Facility (PTF).
3. Choice of two modular systems for the PTF trial from a number of deserving systems.
4. Demonstration that, providing appropriate engineering standards are developed, grassed cellular pavers do provide a viable low cost surfacing option for safe havens and for emergency access and egress routes subject to heavy wheel loads.
5. Verification in the PTF trials of a theoretical analysis, which predicted that a load similar to that applied by the legs of propped trailers would not cause a shear failure of the foundation under the two selected pavers. In both cases, however, the pavers did fracture under the load.
6. Identification of the changes needed to the current specification for grassed cellular pavers contained in Clause 1109 of the Specification for Highway Works (November 2004). Amongst other requirements, this specification requires the bending strength of cellular pavers to be greater than 5 MPa (BS EN 1339). Experience gained in using the test has shown that the measurement
of cross-sectional area, used in the calculation of bending strength, is extremely difficult when using cellular pavers with voids and, sometimes, an indented surface profile.

7. Demonstration that the predominant modes of distress of the two cellular pavers used during the PTF trials were different. In the case of one of the systems, the pavers eventually began to crack after a significant amount of trafficking. Whereas the ‘rocking’ of the pavers under trafficking, of the other system, eventually resulted in increased ‘bedding-in’ of the pavers into the underlying sand layer, increasing the surface deformation.

8. Demonstration that the effective stiffness of modular, cellular pavers for pavement design purposes should be assumed to be similar to that of the underlying foundation.

9. Development of foundation designs for road trials of safe havens and emergency access or egress routes for a traffic loading up to 50,000 standard axles and heavy propped loads of approximately 10 tonnes.
1 Introduction

In order to reduce congestion on the motorway network, the Highways Agency (HA) is considering a number of alternative techniques to create additional road space. In a parallel initiative, they are also considering improving emergency access to, and egress from, the trunk road network.

To provide additional road space, the HA aims to turn some hard shoulders into extra traffic lanes without any overall widening of the roads. This raises safety concerns associated with the parking of broken down vehicles that would need to be left on strengthened verges or be towed to purpose built safe havens.

To provide improved emergency access to, and egress from, the trunk road network in England, the HA recently published Interim Advice Notes IAN 68/06 and IAN75/06. IAN 68/06 is concerned with the infrastructure changes required to implement this policy. IAN 75/06 provides a Code of Practice for emergency access to, and egress from, the trunk road network in England. In IAN 68/06, emergency turnaround areas (ETAs) in the road verge are proposed. This document states that construction of ETAs can include the use of approved cellular concrete pavers that permit grass growth, as these types of pavements are likely to be cost effective and will provide a less conspicuous refuge to unauthorised users.

Following a previous study of methods and products that might be used to strengthen motorway grass verges, the HA commissioned TRL in 2004, under the Geotechnical Framework for Consultancy Services and through the ARUP/TRL/BRE partnership, to research modular, cellular systems, which could be used to provide hardened verges, hard standings and low volume trafficked routes. The study included the following:

- Review to identify potential systems.
- Preliminary pavement design, including design traffic and maximum design loading, structural modelling and bearing capacity failure analysis under high loads.
- Laboratory testing and selection of systems to be used in pilot-scale trials in the TRL Pavement Test Facility (PTF).
- The design, construction, trafficking and monitoring of the PTF trial.
- Pavement design proposals for road trials.

The report summarises the study and makes recommendations for implementation through road trials.
2 Review of the available modular systems

2.1 Review

A review was carried out of commercially available systems that possibly could be used to strengthen grass verges. These systems were then compared to the following three criteria to allow the selection of suitable candidates for trafficking in a PTF trial. The criteria were:

1. The system, according to manufacturer’s literature, should be capable of carrying heavy traffic and to be constructed in a modular fashion to ease future maintenance.
2. The system should have an adequate surface area to prevent excess deformation or shear failure in any of the underlying foundation layers when typically loaded.
3. The system, according to the manufacturers’ literature, should have a suitable block thickness and degree of perforation to support grass growth.

Six candidate systems were selected for consideration. These were:

1. Grassguard 180 (Concrete)
2. Grasscel 125 (Concrete)
3. Tuff Turf (Concrete)
4. Sigma Grass Blocks (Concrete)
5. Armorturf Grid Paving System (Concrete)
6. Truckpave (Plastic)

Details of these products, abstracted from the manufacturer’s technical literature, are given below. Reporting these details should neither be taken as a TRL opinion, nor a TRL endorsement of the product.

2.1.1 Grassguard 180 (Marshalls)

The manufacturer’s literature states that Grassguard is able to cater for heavy-duty loading including commercial vehicle parking and emergency access (Marshalls, 2004a). The manufacturer’s installation guidelines state that a minimum depth of 80 to 150mm of Type 1 subbase should be provided for all applications (Marshalls, 2004b). The actual subbase depth should be dictated by prevalent ground conditions and anticipated traffic loadings. A subgrade improvement layer may be considered to improve bearing capacity of weak ground. Also, adequate drainage must be ensured through subbase/subgrade improvement layers to prevent an hydraulic head developing within the bedding/regulating layer. When overlaying impervious formations, drainage must be provided. The subbase needs to be overlaid by a bedding/regulating layer of sharp sand or pea shingle, 25mm thick, to ensure the units are adequately seated and evenly supported. Cementitious materials are not permitted for use in this layer. The proportion of the area of the bottom of a Grassguard 180 paver that is concrete for the transmission of traffic loads to the underlying materials is 82 per cent, whereas the proportion of its surface available for grass growth is 75 per cent due to its indented profile. The subbase is required to contain some humus to support root development and maintain grass growth during long dry periods. The guide states that the full strength of the installation will only be developed when grass cover is fully established. Photographs of a Grassguard paver and an installation are shown in Figure 1.
2.1.2 Grasscel 125 (Ruthin Precast Concrete Ltd)
Ruthin Precast Concrete Ltd (2004) state that Grasscel 125 can be used for heavy-duty access routes such as farm tracks, water and sewage treatment access, lorry parking areas and public highway verge strengthening. The blocks are manufactured to a Grade C35 concrete in accordance with BS 8500-1:2002 (BSI, 2002). The system needs to be constructed on a free draining granular subbase overlain by a 20 mm bedding layer of sharp sand. The proportion of the surface of Grasscell 125 available for grass growth is 70 per cent (Editorial comment – approximately 57% for root growth). A schematic representation is shown in Figure 2 with a photograph of an installation.

2.1.3 Tuff Turf (Supreme Concrete Ltd)
Supreme Concrete Ltd (2004) produces a concrete paver, called Tuff Turf. Their application is a grassed alternative to standard concrete and tarmac car parks and access areas. The blocks are required to be laid on a 20 mm thick bedding layer of sharp sand over a compacted subbase of quarry waste or similar approved unwashed material of maximum stone size of 75 mm. The subbase is to be at least 150 mm thick. Approximately 85 per cent of the block area is available for distribution of the load to the underlying area and about 79 per cent of the block area is available for grass due to its indented top surface. The pavers and a grassed installation are shown in Figure 3.
2.1.4 Armorturf Grid Paving System (Armortec Concrete Products Ltd)

Armortec Concrete Products Ltd (2004) produces the Armorturf Grid paving system, which is recommended for use on primary and overflow car parks, airfield taxiways and runways as well as emergency access roads. This system consists of flat lattice concrete pavers, where the concrete used has a minimum 28-day cube compressive strength of 50 N/mm². The manufacturer’s literature shows the pavers laid on 25 to 50 mm sand over 100 to 250 mm of subbase or hardcore for a typical car park. The use of geotextile filter fabric on the soil subgrade is also shown. Depending on the laying pattern, the open area for grass propagation is approximately 28 or 35 per cent of the paved area. Schematic diagrams of the pavers and a photograph of an installation are shown in Figure 4.

2.1.5 Sigma Block (Brett Landscaping Ltd)

Brett Landscaping Ltd (2004) produces Sigma Block that is said in their literature to provide a practical surface with a long life, suitable for car and caravan parks, industrial and agricultural environments in situations where prolonged heavy duty trafficking does not occur. Sigma blocks are laid on a 30 to 40 mm layer of screeded, uncompacted sharp sand over 200 mm of well-compacted granular subbase. Photographs of a Sigma block and an installation are show in Figure 5.
2.1.6 **Truckpave (Hoofmark UK Ltd)**

Truckpave, supplied by Hoofmark (UK) Ltd (2004), is manufactured from recycled plastic. The manufacturer’s literature states that these pavers can be used for lorry access roads and parking areas, service yards and other heavy duty applications. The underlying foundation must have sufficient strength to withstand the maximum bearing load likely to be applied, even in the wettest conditions. A typical construction is given as a 250 mm thick layer of well compacted, fines free graded stone beneath 30 mm thick layer of coarse sand. Type 1 subbase is not recommended. Truckpave is considered to have an advantage over concrete pavers in that it will flex instead of crack. Additionally, as plastic is an insulator, grass growing within the paver can survive much longer periods of hot weather. The proportion of its area suitable for grass growth is 53 per cent. A photograph of a Truckpave paver is shown in Figure 6.

![Figure 5 Sigma Block and installation](Reproduced with the permission of Brett Landscaping Ltd)

![Figure 6 Truckpave recycled plastic paver](Reproduced with the permission of Hoofmark (UK) Ltd)

2.2 **Product summary**

The physical dimensions of the products are summarised in Table 1. The data in this table was used in the shear failure analysis of the pavement layers loaded by point loads resulting from the jacking up of
heavy vehicles to change a flat tyre, or the propping of the trailer legs when the trailer is disconnected from its tractor unit.

Table 1 Dimensions of the products reviewed (order of increasing gross surface area)

<table>
<thead>
<tr>
<th>Product name</th>
<th>Material</th>
<th>Width (m)</th>
<th>Length (m)</th>
<th>Thickness (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Armorturf</td>
<td>Concrete</td>
<td>0.300</td>
<td>0.300</td>
<td>0.100</td>
</tr>
<tr>
<td>Grasscel 125</td>
<td>Concrete</td>
<td>0.400</td>
<td>0.440</td>
<td>0.125</td>
</tr>
<tr>
<td>Sigma Block</td>
<td>Concrete</td>
<td>0.400</td>
<td>0.600</td>
<td>0.080</td>
</tr>
<tr>
<td>Tuff Turf</td>
<td>Concrete</td>
<td>0.400</td>
<td>0.600</td>
<td>0.100</td>
</tr>
<tr>
<td>Grassguard 180</td>
<td>Concrete</td>
<td>0.400</td>
<td>0.600</td>
<td>0.120</td>
</tr>
<tr>
<td>Truckpave</td>
<td>Plastic</td>
<td>0.400</td>
<td>0.600</td>
<td>0.100</td>
</tr>
</tbody>
</table>
3 Testing of candidate systems

3.1 Laboratory test method

Currently, the strength requirements in the Specification for Highway Works (MCHW1, 2001): Series 1100, Part 1109: Grass/concrete paving in relation to the strength of the cellular blocks or reinforced, cellular in-situ concrete slabs are as follows:

- The grade of concrete used to manufacture the slab or block should have a minimum 28-day crushing strength of 35 N/mm².
- The characteristic bending strength at failure should be at least of 5MPa (BS EN 1339).

In terms of construction of the pavement, the specification requires that these panels (modular or in-situ concrete) shall be bedded on a layer of uncompacted sharp sand over a Type 1 subbase that complies with Clause 803 (MCHW1) and is compacted in accordance with Clause 801.

In order to assess the candidate modular systems, it was concluded that it was best to measure the bending strength and breaking load using Annex F of BS EN 1339. A schematic diagram of the test method is shown in Figure 7.

![Figure 7 Schematic representation of Annex F, BS EN 1339: 2003](Permission to reproduce extracts from the BS EN 1339: 2003 is granted by BSI. British Standards can be obtained from BSI Customer Services, 389 Chiswick High Road, London W4 4AL. Tel: +44 (0)20 8996 9001. email: cservices@bsi-global.com)

The calculation of Bending Strength (T) is given in Equation 1.

\[ T(\text{MPa}) = \frac{3 \times P \times L}{2 \times b \times t^2} \quad \text{...Equation 1} \]
Where:

\[ T \] is the Bending Strength (MPa).

\[ P \] is the Breaking Load (N).

\[ L \] is the distance apart of the supports (mm).

\[ b \] is the width of the paver at the failure plane (mm).

\[ t \] is the height of the paver at the failure plane (mm).

3.2 Test results

The test requires that the load is increased uniformly such that the breaking load is reached within 45 ± 15 seconds. Three samples from each system were tested. The first sample was used to gauge the time to failure and then the rate of loading changed to achieve the correct time during testing the second and third sample.

The results are shown in Table 2 and photographs of the systems prior to testing and after failure are shown in Figure 8.

<table>
<thead>
<tr>
<th>Product</th>
<th>Failure Load (P) (kN)</th>
<th>Failure Displacement (mm)</th>
<th>Width (b) (^1) (mm)</th>
<th>Height (t) (^1) (mm)</th>
<th>Bending Strength (T) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Armorturf</td>
<td>32.3</td>
<td>5.0</td>
<td>268</td>
<td>102</td>
<td>4.3</td>
</tr>
<tr>
<td>Armorturf</td>
<td>38.0</td>
<td>5.5</td>
<td>277</td>
<td>103</td>
<td>4.9</td>
</tr>
<tr>
<td>Armorturf</td>
<td>34.7</td>
<td>4.9</td>
<td>275</td>
<td>103</td>
<td>4.5</td>
</tr>
<tr>
<td>Grasscel 125</td>
<td>14.7</td>
<td>5.9</td>
<td>281</td>
<td>95</td>
<td>3.4</td>
</tr>
<tr>
<td>Grasscel 125</td>
<td>16.0</td>
<td>5.9</td>
<td>288</td>
<td>95</td>
<td>3.6</td>
</tr>
<tr>
<td>Grasscel 125</td>
<td>15.3</td>
<td>8.1</td>
<td>283</td>
<td>95</td>
<td>3.5</td>
</tr>
<tr>
<td>Sigma block</td>
<td>7.2</td>
<td>4.3</td>
<td>396</td>
<td>70</td>
<td>3.1</td>
</tr>
<tr>
<td>Sigma block</td>
<td>5.9</td>
<td>3.1</td>
<td>396</td>
<td>70</td>
<td>2.5</td>
</tr>
<tr>
<td>Sigma block</td>
<td>5.8</td>
<td>3.8</td>
<td>396</td>
<td>70</td>
<td>2.5</td>
</tr>
<tr>
<td>Tuff Turf</td>
<td>10.6</td>
<td>3.1</td>
<td>351</td>
<td>79</td>
<td>4.0</td>
</tr>
<tr>
<td>Tuff Turf</td>
<td>9.1</td>
<td>3.8</td>
<td>351</td>
<td>78</td>
<td>3.5</td>
</tr>
<tr>
<td>Tuff Turf</td>
<td>9.6</td>
<td>3.5</td>
<td>351</td>
<td>78</td>
<td>3.7</td>
</tr>
<tr>
<td>Grassguard 180</td>
<td>13.2</td>
<td>4.7</td>
<td>451</td>
<td>55</td>
<td>8.0</td>
</tr>
<tr>
<td>Grassguard 180</td>
<td>15.0</td>
<td>4.7</td>
<td>451</td>
<td>55</td>
<td>9.1</td>
</tr>
<tr>
<td>Grassguard 180</td>
<td>17.2</td>
<td>4.4</td>
<td>451</td>
<td>55</td>
<td>10.4</td>
</tr>
<tr>
<td>Truckpave</td>
<td>14.5</td>
<td>15.9</td>
<td>403</td>
<td>97</td>
<td>3.1</td>
</tr>
<tr>
<td>Truckpave</td>
<td>16.2</td>
<td>22.1</td>
<td>403</td>
<td>97</td>
<td>3.5</td>
</tr>
</tbody>
</table>

Note 1: The width and height are those recorded around the outside of the fracture plane of the grass paver.
Figure 8 Systems before and after testing
Figure 8 Systems before and after testing (Continued)
Figure 9 shows the load/displacement relationships for each of the candidate systems that were tested. The results included are those for the two samples that failed within $45 \pm 15$ seconds. Because of the complex nature of the fracture plane, the load at failure was considered a better measure of the relative resistance to structural cracking of the different systems. It can be seen that two of the systems failed at lower loads ($<10\text{kN}$) and at smaller displacements ($<4\text{mm}$) than the other four systems. These two systems, Sigma Block and Tuff Turf, were not considered for use in the PTF trial.

The results of the remaining four systems, Armorturf, Grasscel 125, Grassguard 180 and Truckpave, are summarised in Table 3.

<table>
<thead>
<tr>
<th>Product name</th>
<th>Failure Load (kN)</th>
<th>Failure Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Armorturf</td>
<td>36.4</td>
<td>5.2</td>
</tr>
<tr>
<td>Grasscel 125</td>
<td>15.0</td>
<td>7.0</td>
</tr>
<tr>
<td>Grassguard 180</td>
<td>16.1</td>
<td>4.6</td>
</tr>
<tr>
<td>Truckpave</td>
<td>15.4</td>
<td>19.0</td>
</tr>
</tbody>
</table>

The most obvious difference between the three concrete systems and the system manufactured from recycled plastic (Truckpave) is the magnitude of the displacement at failure. All the concrete systems failed at a displacement of less than 7mm, whereas Truckpave did not fail until a displacement of approximately 20mm. Truckpave would therefore be more tolerant than the other systems to uneven support that may result from poor construction practice. Truckpave, however, was not considered for...
use in the PTF trial as it was decided that two concrete systems of different sizes were required to assess the possible shear failure of the underlying subbase under simulative high propped leg loads.

A more detailed comparison between the Load/Displacement relationships for the three remaining concrete systems is shown in Figure 10, where it can be seen than the load to failure for Grassguard 180 and Grasscel 125 are similar, at approximately 15kN. The load to failure of the Armorturf system was considerably higher at approximately 35kN.

![Figure 10 Load/Displacement relationships for Armorturf, Grasscel 125 and Grassguard 180](image)

### 3.3 Pavement test facility (PTF) trial

It was not an objective of the PTF trial to evaluate the performance of individual grass pavers but rather to evaluate the proposed structural design of the ‘safe havens’, emergency turnaround areas (ETAs) and other low volume, trafficked routes built with grass pavers, both in terms of possible failure through structural rutting by traffic or shear failure of the Type 1 subbase or lower layers by high, static concentrated loads.

To assess shear failure, it was necessary to include a system with a relatively small surface area as well as one of the systems with a large surface area. Reference to Table 1 shows that the most appropriate systems, in this respect, are Grassguard 180 and Armorturf or Grasscel 125. These three systems had the highest load at failure and also have the highest and lowest surface area respectively. Grassguard 180 and Grasscel 125 were eventually selected for the trial. Grasscel 125 was chosen in preference to Armorturf, because it had a greater void area available for grass growth.
4 Porous pavements and material requirements

In general, pavements can be grouped into two categories, that is porous and “non-porous” (or better defined as “less permeable”) pavements. Conventional asphalt or concrete surfaced pavements fit into the “non-porous” category and the intention in designing these pavements is to keep water away from the subgrade. Porous pavements, on the other hand, consist of wearing surfaces that have greater than normal percentage of voids that permit water to pass through the structural layers. Pavements built with cellular modules also fit into this category of pavements. As water is allowed to enter the foundation layers, these layers therefore need to serve two functions, namely provide strength and allow water to pass through at a prescribed rate. Water may be temporarily stored in such pavements before entering the underlying ground or passing into a piped drainage system. Porous pavements that perform this function are also termed “reservoir pavements”.

4.1 Subgrade strength requirements

A subgrade’s structural behaviour is closely linked to its drainage conditions. In conventional pavement construction, the subgrade is normally protected from the ingress of water. In a porous pavement, as surface water is allowed to enter the foundation and ultimately into the subgrade, the structural properties of subgrade for pavement design should relate to a wetted condition unless it is protected from infiltrated water.

In this study, the California Bearing Ratio (CBR) test has been adopted to characterise the structural properties of the subgrade as it can be measured both in the laboratory and in-situ and can also be related to subgrade stiffness.

In practice, construction operations ensure that the subgrade is never completely undisturbed and many are substantially remoulded, at least at depths close to formation that determine performance. Therefore, as a conservative measure, CBR values of remoulded cohesive subgrades are preferred. CBR strengths of soils can be obtained using various methods, including: 1) In-situ methods which involve carrying out either in-situ CBR tests or dynamic cone penetrometer (DCP) tests and assessing CBR values from DCP – CBR correlations; 2) the Suction-Index method of Black and Lister (1979), with its application to typical road foundation conditions developed by Powell et al (1984).

The in-situ methods estimate the current CBR of the subgrade at formation or sub-formation, if a capping is used, at its existing moisture regime.

The Suction-Index method has often been used in design to predict soil strengths/stiffnesses in both the construction and long-term condition. The information required for this analysis is the soil type (i.e. plasticity index), the water table depth and thickness of foundation only and foundation plus pavement respectively.

Assessment of short and long-term CBR strengths is described in more detail in HA44/91 and IAN73/06. In adoption of these procedures, economic designs with an acceptable risk will most likely be achieved when the engineer bases his choice of soil CBR strength on his knowledge of local materials and their likely response to particular prevailing drainage conditions.

4.2 Capping and subbase

Capping is a layer immediately above the subgrade, which is made up of relatively cheap materials that provide an improved subgrade and a formation suitable for adequate compaction of the subbase and, in a conventional pavement, construction of the pavement layers. This layer also provides sufficient support in the long term under equilibrium moisture conditions, enabling the pavement to meet the required performance. Its structural aim is to increase the stiffness modulus and the strength of the formation, on which the subbase will be placed.
Conventional subbase materials have a dual role in that they not only provide structural support to the pavement, but they also provide protection to the subgrade against weather action. The permitted materials (MCHW1) include a continuously graded granular material known as Type 1.

For porous pavements, a more permeable subbase material than Type 1 subbase may be required to drain infiltrated water sufficiently fast to prevent a build-up of water in the upper foundation and pavement layers. In reservoir pavements, the subbase plays the additional role of allowing temporary storage of drained water during heavy rainfall or excess runoff from adjacent impervious surfaces. To satisfy these roles, the grading requirements of the subbase may need to be different than those of conventional subbase materials such as Type 1 subbase to ensure that there is a sufficient air voids content in the compacted materials. However, this required porosity must not be achieved at the cost of an undesirable reduction in the structural performance of the subbase. Chaddock et al (2000) researched more open graded and coarser subbase. They showed that both the strength and stiffness of Type 1 and coarse subbases decreased with saturation, strength more markedly than stiffness. Upon draining, some of the lost stiffness was recovered. Upon drying, most of the subbase strength was recovered in the coarse subbase. In comparison to Type 1 subbase, the coarser material had better structural performance under adverse moisture regimes when directly trafficked, although there was an indication that the coarse subbase might be more susceptible to deformation under high shear forces caused by construction equipment. Therefore, a balance needs to be reached in relation to the grading requirements such that a subbase material is able to drain and, if required, store water as well as provide an adequate load bearing capacity. Chaddock et al’s research showed that coarse subbase materials investigated should be laid to the same thicknesses as Type 1 subbase material.

In choosing materials for the upper layers of the foundation, the potential effect of frost heave should be considered. Dependent on the mean annual frost index, a depth of 350 or 450 mm of non-frost susceptible material is required for conventional pavement design. It would therefore be prudent for these permeable modular pavements to ensure that materials used beneath the paving modules are not frost susceptible to this depth measured from the top of the pavement. This requirement may limit the use of some unbound granular capping materials and require the subbase to be thickened.

### 4.3 Surfacing

Over the design life, the structural behaviour of the surfacing layer is required to withstand, without excessive deterioration, the design traffic and the applied forces caused by either jacking the vehicles to change tyres, or propping the trailer to permit the tractor unit to be removed for repair.

### 4.4 Material used in the PTF trials

Although a surfacing comprising cellular pavers may be characterised as a porous pavement, the foundations for the pavements in the PTF trial were not specifically designed as a purposely-built reservoir pavement because the pavements were protected from rainfall in the PTF. Also, storage of run-off water is not considered a primary requirement for the applications of safe havens, ETAs and low volume trafficked routes. Conventional unbound granular subbase materials, which often have a lower permeability to water than materials used in reservoir pavements, were therefore used in the trial. However, if these materials are found to be inadequate under typical in situ moisture regimes, then modifications to the permitted materials will need to be made.
5 Design loading

This section of the report describes how the design traffic for safe havens and emergency turnaround areas was estimated. A description is also given of the determination of the maximum load that could be applied to a safe haven pavement when the legs of a trailer are propped following separation from the tractor unit.

5.1 Usage

5.1.1 Safe Havens

The average stoppage rate for heavy vehicles on motorways, together with the total distance travelled annually by these vehicles on the UK motorways, was used to calculate the yearly number of stoppages per kilometre of a motorway. Using the work of Summersgill et al (2004), the number of overall heavy vehicle stoppages during a 24 hour period was estimated as 281 stops per million vehicle kilometres, where the stoppages included those due to breakdowns, vehicle checks, attendance by a recovery vehicle and other stops. The Road Traffic Statistics (DfT, 2003) showed that, during 2003, the total annual distance travelled by heavy vehicles on UK motorways amounted to 11.5 billion vehicle kilometres. The same statistics showed that total motorway length in the UK is 3476 kilometres. Therefore, the total number of stoppages per kilometre of motorway was calculated as given in Equation 2.

\[
\frac{281\text{ stops}}{10^6\text{ heavy veh.km}} \times \frac{1.5 \times 10^9\text{ heavy veh.km}}{\text{year}} \times \frac{1}{3476\text{ km}} = \frac{930\text{ stops}}{\text{km.year}} \quad \text{...Equation 2}
\]

Assuming that safe havens on motorways will be located once every kilometre and their design life is 10 years, then the overall number of stops of heavy goods vehicles per safe haven will be 9,300 over the 10-year period.

5.1.2 Emergency turnaround areas

When a carriageway on a motorway or other dual carriageway road is blocked by a road accident or other incident, it may be necessary to isolate the affected section of road and extricate trapped road users by turning around vehicles and routing them onto the adjacent carriageway. This procedure would involve the use of emergency turnaround areas (ETAs) built into the road verges. The number of heavy vehicles \( (N_{\text{hgv}}) \) to be carried by an ETA in its design life of 10 years was estimated using Equation 3.

\[
N_{\text{hgv}} = N_i \times S_{\text{ETA}} \times d_{\text{hgv}} \times n_{\text{CW}} \quad \text{...Equation 3}
\]

Where \( N_i \) is the number of emergency incidents in the design life of 10 years,

\( S_{\text{ETA}} \) is the spacing of ETAs,

\( d_{\text{hgv}} \) is the total number of HGVs per lane kilometre,

and \( n_{\text{CW}} \) is the number of lanes per carriageway.

IAN 75/06 estimates the number of incidents in which the carriageway is blocked at least three hours for the trunk road network across the whole of England. For the motorway trunk road network, the number of incidents over any twelve month period is suggested as 357. That is, on average, one incident for each 8.54 km of the motorway trunk road network. Analysis of sixteen motorways in England showed that, on average, junctions are spaced about 6.3 km apart. Consequently, it has been assumed that, on average, there is one incident between two junctions every 1.35 years, or 7.5 incidents in 10 years \( (N_i) \).
For emergency turnaround areas (ETAs) placed between junctions, the longest section of road supplying vehicles to any ETA ($S_{\text{ETA}}$) is the length of the road between two ETAs. Although this distance is not specifically stated in IAN 75/06, it is taken to be the distance between major changes on the road to ensure emergency access and egress. This distance is given as 5 km for motorways carrying over 15,000 vehicles per day in a single direction.

According to IAN 75/06, the percentage of vehicles that are HGVs on motorways is 18.5 per cent. Based on this value, the number of HGVs per lane km ($d_{\text{hgv}}$) can be interpolated from the information given in IAN 68/06 as 24.5.

IAN 68/06 only suggests dimensions for ETAs for roads with between 2 and 3 lanes per carriageway and so the number of lanes ($n_{\text{cw}}$) is restricted to either 2 or 3.

The total number of HGVs to be carried by each ETA on motorways is therefore estimated from equation 3 to be about 2750 for three lane carriageways and about 1825 for two lane carriageways. A similar procedure could be used to calculate the total HGV numbers in a design life period to be carried by an ETA for the all purpose road network.

5.2 Maximum design load for failure analysis of the foundation

The design load for the failure analysis was based on the maximum load that occurs under the propped legs of a fully loaded trailer when it is disconnected from the tractor unit. The following three types of supporting legs are commonly used:

(a) A twin-wheeled support comprising 210 mm diameter wheels, where each wheel is 80 mm wide and the overall width of the twin-wheels is 320 mm.

(b) A curved plate of 240 mm width that is similar to the profile and width of a tyre.

(c) A flat bed plate, which is 300 mm $\times$ 240 mm.

Legs of Type (a) and (b) are shown in Figure 11.

![Figure 11 Layout of different legs](image-url)

In order to establish the maximum loads exerted by typical lorries, the axle loads of 38 tonne (five axles) and a 44 tonne (six axles) fully laden lorries were weighed with weigh pads. The resulting loads on each axle are given in Table 4, where it can be seen for a correctly loaded trailer that none of the axle loads exceeded the maximum permitted of 10.5 tonnes. The loads carried by the propped legs and the tyres on the trailer, when the tractor unit was detached, were then weighed using weigh
pads. These measurements are also given in Table 4 and showed that the highest loading was 9.5 tonnes under each propped leg of the trailer. This loading has been used for the theoretical failure analysis and the pilot-scale trial static load measurements.

**Table 4 Axle weights on 38 and 44 tonne fully laden lorries (including a full tank of petrol)**

<table>
<thead>
<tr>
<th>Axles no.</th>
<th>Axle load (kg)</th>
<th>44 tonne lorry</th>
<th>38 tonne lorry</th>
<th>Three axle trailer unit on propped legs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axle 1</td>
<td>5,000</td>
<td>6,300</td>
<td></td>
<td>2 propped leg weighing 19,000</td>
</tr>
<tr>
<td>Axle 2</td>
<td>5,870</td>
<td>8,100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Axle 3</td>
<td>9,480</td>
<td>8,020</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Axle 4</td>
<td>8,150</td>
<td>7,810</td>
<td></td>
<td>6,540</td>
</tr>
<tr>
<td>Axle 5</td>
<td>8,080</td>
<td>7,820</td>
<td></td>
<td>5,790</td>
</tr>
<tr>
<td>Axle 6</td>
<td>8,020</td>
<td>-</td>
<td></td>
<td>5,660</td>
</tr>
<tr>
<td>Total</td>
<td>44,600</td>
<td>38,050</td>
<td></td>
<td>36,990</td>
</tr>
</tbody>
</table>

5.3 **Traffic loading for design**

5.3.1 **Safe Havens**

The design traffic loading in standard axles (sa) for safe havens was calculated as the summation of the heavy vehicles trafficking the safe haven over the design period plus the increased loading as a result of the occasional propped loading of trailers.

5.3.1.1 **Traffic**

The generic form of the detailed method for traffic calculation HD24/06 (DMRB 7.2.1, 2006) was applied to “other goods vehicles” classes 1 and 2 (OGV1 and OGV2) as given in Equation 4. The applicable annual flow of each vehicle class was calculated from the estimated total lorry usage recorded in Section 5.1.1 using the proportion by class factors given in Table 5. The growth and wear factors for each vehicle class are also recorded in the table.

\[ T = Y \cdot F_Y \cdot G \cdot W \]  

**…Equation 4**

Where  
\( T \) = Design traffic (standard axles),  
\( Y \) = Design period (years),  
\( F_Y \) = Flow of commercial vehicles in one year,  
\( G \) = Growth factor,  
\( W \) = Wear factor (standard axles).
Table 5 Traffic parameters for various classes of commercial vehicle

<table>
<thead>
<tr>
<th>Goods vehicle class</th>
<th>Proportion by class</th>
<th>Growth factor</th>
<th>Wear factor (sa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OGV1</td>
<td>0.325</td>
<td>1.0</td>
<td>0.6</td>
</tr>
<tr>
<td>OGV2</td>
<td>0.675</td>
<td>1.3</td>
<td>3.0</td>
</tr>
</tbody>
</table>

5.3.1.2 Propped trailers of articulated lorries.

The increased traffic loading resulting from occasional propped loading of “other goods vehicles” class 2 (OGV2) has been calculated using Equation 5.

\[ T_p = Y \cdot G_2 \cdot (F_2 \cdot P_V) \cdot (W_2 \cdot P_F) \cdot (P_P) \]

**…Equation 5**

Where
- \( T_p \) = Loading as a result of propped loading (standard axles),
- \( Y \) = Design period (years),
- \( G_2 \) = Growth factor for OGV2 vehicles,
- \( F_2 \) = Flow of OGV2 vehicles in one year,
- \( P_V \) = Proportion of OGV2 vehicles that prop trailer loads,
- \( W_2 \) = Wear factor for OGV2 vehicles (standard axles),
- \( P_P \) = Probability of a propped load placed on a particular surface module, and
- \( P_F \) = Adjustment factor to relate relative damage of propped leg load to wheel load.

In Equation 5, \( P_V \) is the proportion of the articulated commercial vehicles that prop the legs to support a trailer so that the tractor unit can be disconnected. There are no national statistics on this figure for UK motorways and an estimated value of 0.10 was initially assumed in the calculations, meaning that only 1 in 10 trailers of all articulated vehicles will be propped.

Also, in Equation 5, \( P_F \) is the adjustment factor, based on subgrade strain, used to take account of the increased damaging effect of a propped leg load in comparison to that caused by wheel loads of a standard lorry. The maximum subgrade strain was calculated at 0.75m depth and then the adjustment factor was determined from Equation 6 (Powell et al, 1984).

\[ P_F = \left( \frac{\text{maximum subgrade strain beneath the propped trailer}}{\text{maximum subgrade strain beneath the fully laden lorry}} \right)^{3.95} \]

**…Equation 6**

The method adopted to derive the subgrade strain uses a multi-layer elastic model of the foundation with surface loads distributed over a standard circular contact area of radius 0.15m. The loadings listed in Table 4 were used at the axle/propped leg locations and were determined from measurements made on a six axle articulated lorry or its propped trailer. The analysis used the same elastic layer properties as those used in the thickness design method described later in the report. The adjustment factor was found to be nearly 14.9. The effect on this factor of depth in the subgrade was not investigated.

It was assumed that not all propped legs will be concentrated on the same location along a given safe haven. The probability of a propped load being placed on an individual module is determined from the length of each block compared with that length of the safe haven that may be loaded by propped legs. It has also been assumed that lateral position of the trailer is fixed and the longitudinal position of the prop load follows a uniform distribution. If it is assumed that a 60m typical length of the safe haven is used for propped legs, then the probability (\( P_P \)) that a propped leg will be positioned on a particular 0.6m long, cellular paver is 0.01 (i.e. one in 100).
5.3.1.3 Design Traffic Loading

The traffic calculated separately for vehicle classes OGV1 and OVG2 using Equation 4 and the traffic determined as equivalent to propping the trailer unit of articulated vehicles of class OGV2 by using Equation 5 were summed to give the design traffic loading of 26,663 standard axles (sa). If the proportion of articulated vehicles that are propped ($P_V$) is increased to 0.2, then the traffic increases to 27,026 sa; that is an increase of less than 1.4 per cent. If, on the other hand, the length of the cellular pavers is halved (i.e. to the dimensions of the smallest block sizes considered), then the design traffic level, in fact, also reduces by about 0.7 per cent to 26,481 sa. These changes are not considered significant to impact the overall design traffic and further refinement of the traffic calculation method is not justified. Therefore, by considering that 20 per cent of the articulated vehicles propped their legs, and applying an overall factor of safety of 1.11 to the traffic levels gives a traffic design traffic loading of 30,000 sa. This value of traffic loading was used to design the PTF trials.

The actual traffic using a safe haven could vary significantly from this estimate. Information on the traffic mix, the frequency of stoppages and the particular layout of safe havens for a specific road, when possible, should be used to improve the estimate of traffic loading.

5.3.2 Emergency turnaround areas

The design traffic loading in standard axles predicted to use an ETA over its design life was calculated using Equation 3 for two and three lane carriageways. In this equation, the traffic flows were those given in section 5.1.2. The growth factor in this case, however, relates to the anticipated change in the relative proportions of HGVs to other vehicles and is taken as unity in the absence of any specific information. Otherwise, the parameters given in Table 5 were adopted. The design traffic for 2 and 3 lane motorway carriageways was found to be approximately 4000 sa and 6000 sa respectively.

The actual traffic using an ETA could vary significantly from these average estimates. Information on the traffic mix, the frequency of incidents and the particular layout of ETAs for a specific road, when possible, should be used to improve the estimate of traffic loading.
6 Structural design

6.1 Pavement design

The structural design of modular, cellular pavements for vehicles is considered in this section. The design requires information on the likely strength requirements of the subgrade, which was discussed in Section 4.1, as well as knowledge of the likely design traffic that is estimated in Chapter 5.

Initially, the modular systems used for the surfacing of porous pavements were assumed, for conservatism, to offer no contribution to the overall structural performance of the pavements. The pavement design process therefore reduced to just designing the foundation. Thereafter, the modular systems were assumed to have a structural contribution equivalent to that of an equal thickness of good quality crushed rock subbase.

6.1.1 Method

The thickness design of the modular pavement is based on a similar method to that adopted by Powell et al. (1984). Designs were produced for a subbase only foundation as this foundation was to be built in the PTF trial. Traditional unbound granular materials have been considered and their thicknesses obtained as a function of subgrade CBR strength and for a range of traffic in standard axles.

The design method is based on a theoretical model of the foundation that is represented by series of linear elastic layers on a semi-infinite subgrade layer and loaded at its surface to simulate traffic. The model predicts the vertical compressive elastic strain induced at the top of the subgrade by the applied load. In the design process for modular pavements, the layer thicknesses were calculated to ensure that the traffic induced vertical subgrade strains were less than permissible values. These permissible values of strain were derived from an analysis of foundations on the road network and pilot-scale trials. The permissible compressive elastic subgrade strain, $\varepsilon_z$, induced by a standard 40kN wheel load is related to the cumulative traffic, which is defined as the total number of standard axles (N) in the design life of the pavement, by the following deformation criteria.

$$\log N = -7.21 -3.95 \log \varepsilon_z$$  \hspace{1cm} \text{...Equation 7}

Subbase only designs were produced for two levels of risk. In the most conservative approach, it was known that the cellular pavements would be subjected to infiltrated water throughout their life and, unlike foundations of paved roads during their construction, traffic could not be kept off the foundation when it was subjected to adverse weather conditions. To make some allowance for this situation, an adjustment factor of 0.75 was applied to the subgrade strain criterion instead of directly using Equation 7 as was normal for foundations for paved roads during their construction. The resulting increased thickness of subbase also provided some allowance for the effective thinning of the subbase due to contamination with fine-grained cohesive soil of the subgrade. It was also assumed that the modules did not make a structural contribution to the pavement.

In the least conservative design, no adjustment factor was applied to the subgrade strain criterion and the modules were considered to make a structural contribution equivalent to that of an equal thickness of good quality, unbound granular subbase. This latter assumption was assessed by the analysis of FWD tests on the pilot-scale foundations as described in Section 8.3.

The analysis was carried out with the computer programme called BISAR (Shell International Oil Products BV, 1998). The structures analysed are shown schematically in Figure 12 for the two levels of risk.
Note 1: Subbase layer stiffness is 150 MPa when laid on soil of subgrade CBR greater than 5 per cent. But, when subbase is laid on weaker subgrade soil and its thickness is over 225 mm, the subbase is sub-divided into two layers to permit the assignment of different moduli to the lower and upper subbase layers. The sub-division is such that the upper layer is taken to be at least 80 mm thick and the lower layer as thick as possible, but no thicker than 225 mm. The lower subbase layer is assigned a layer stiffness of 3 times the stiffness of the subgrade, whereas the upper subbase layer is taken to have a layer stiffness of 150 MPa.

Note 2: Full friction has been assumed at the interface between the subbase and the subgrade.

Figure 12 Subbase only foundation structures analysed
Using the above process, the thicknesses for the unbound granular subbase only foundations were calculated that satisfied the relevant subgrade strain criteria. The results of the subbase only design calculations are given in Figure 13 and Figure 14 as a series of curves of layer thickness against subgrade soil CBR for different design life traffic levels (i.e. total number of standard axles over the 10-year assumed design life).

For the lowest risk level designs, Figure 13 gives the thickness of subbase for designs with an adjustment factor of x0.75 applied to the subgrade strain criteria. The required subbase thickness decreases with increase in soil strength and with lower amounts of traffic. As an example, an increase in the design traffic by x10 (1x10³ to 10x10³ sa) requires between 130 mm and 170 mm of additional subbase depending on the soil strength. To cater for a soil CBR strength reduction from 5 to 2 per cent, requires an additional thickness of subbase of between 100 mm and 130 mm depending on the envisaged design traffic.

![Figure 13 Variation of subbase thickness with soil strength – Most conservative designs](image)

For the highest risk level designs, Figure 14 gives the total thickness of subbase and cellular paver for designs with no adjustment factor applied to the subgrade strain criteria. As an example, for a module of thickness 120 mm and a pavement with a design traffic amount of 10,000sa, 360 mm of subbase is required for soil of CBR strength 2 per cent, which is 240 mm thinner than the designs given for the lowest risk level design.
6.1.2 Pavement test facility (PTF) trial

The pavement designs were tested in the PTF for a single soil CBR strength, whose value was estimated as 4 per cent in the manner described in Section 7.1.1. The specified subbase thicknesses for a traffic level of 30,000 sa are given in Table 6 for the two levels of risk examined.

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>Module plus subbase</th>
<th>Subbase only</th>
<th>Structural contribution from cellular paver</th>
<th>Adjustment factor of x0.75 applied to soil strain criteria</th>
<th>Soil CBR strength (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>580</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>4</td>
</tr>
<tr>
<td>470</td>
<td>-</td>
<td>Yes</td>
<td>No</td>
<td></td>
<td>4</td>
</tr>
</tbody>
</table>

Subbase on capping designs could be produced that utilise local available material. These designs were not considered in the PTF trial. The use of capping materials may be limited if a material of a relatively open grading specification to achieve a prescribed permeability is specified or if the capping material is frost susceptible.

6.2 Shear failure analysis

When the trailer unit of a fully laden heavy lorry is detached from its tractor unit during a mechanical breakdown, two legs are propped to the front of the trailer. These legs are expected to carry about half the weight of the trailer. When such loads are exerted directly onto a modular, cellular paving structure, there is a danger of shear failure in the underlying unbound granular materials and soils.
6.2.1 Method

The loads applied by the prop legs are highly concentrated, with the contact area being smaller than any of the grass pavers in the review. Providing the modules do not crush, then the load can be considered to be transmitted to the underlying foundation via a footing the size of the module. According to Ramiah and Chickanagappa (1984), there are various forms of shear failure that could occur in a foundation, namely general shear, local shear and punching shear. In order to investigate the likely stability of these modular pavements when heavy lorry legs are propped, a theoretical analysis was carried out of the bearing capacity of footings. Two structures were considered, namely subbase only and subbase overlying a clay subgrade.

The subbase only structure was examined to assess the potential of shear failure within itself. The ultimate bearing capacity of this theoretical structure at the subbase surface was determined by Terzaghi’s solution for a circular footing in a shallow foundation (Scott, 1974) where, in this case, there was no embedment.

If the subbase material does not fail by shear due to the applied load, then, for a subbase on subgrade structure, it is considered safe to assume that the fully compacted subbase material would transfer the applied load at the surface of the subbase layer directly onto the subgrade surface or formation. It is assumed that the total load at formation is not reduced by the subbase, although the pressure is reduced as a consequence of this load being distributed over a greater area by the subbase layer. The approximate method of load spreading by Spangler and Hardy (1973) was adopted in which the diameter of the subsurface loaded area or impact area is defined by lines descending from the edges of the footing at an angle of 30° with the vertical.

Terzaghi’s solution was applied at the top of subgrade using a footing embedded to the depth of subbase and with the above increased width. This solution neglects the frictional properties of the subbase and therefore underestimates the ultimate bearing capacity of the foundation.

Meyerhof (1974), however, took fuller account of the mechanical properties of the subbase. He proposed a punching, shear mechanism for a layered system comprised of a cohesionless material (such as sand) overlying a frictionless material (clay). In adopting Meyerhof’s mechanism for this application, the mass of unbound granular subbase is assumed to push into the clay subgrade as a truncated pyramid, such that the frictional properties of the subbase and the undrained cohesion of the clay subgrade are mobilised in the failure zone. The inclination angle to the vertical of the truncated pyramid is taken as 30°. MacNeil and Snowdon (1996) reviewed the mechanical characteristics of construction materials from various UK sources and found that the maximum drained friction angle ranged from 47.5° for a gravel resembling a capping material to 61.7° for a crushed rock, Type 1 subbase. From this review for a Type 1 subbase, the minimum value of 55° was adopted for subbase in the bearing capacity calculations. The ultimate bearing capacity for a circular footing for this mode of failure was calculated.

The calculations of ultimate bearing capacity described above were carried out for circular footings of specific diameters that were selected so that the footing areas equalled the gross areas of the rectangular or square modules detailed in Table 1.

6.2.2 Factor of safety

The ultimate bearing capacity stresses (q_u) determined from the previous analysis were then compared with the stress resulting from the applied load by the propped legs of a lorry trailer when it is unhitched from its tractor unit. The maximum load on each propped leg of the trailer was measured as 9.5 tonnes as reported in Section 5.2. The stress due to the applied load by the legs (q_p) is calculated by dividing the propped leg load by the impact area, which is dependent on the reference level chosen for the stress calculation. For example, to calculate the applied stresses on top of the subbase layer, the impact area for the applied load becomes the gross area of the footing placed on the subbase; that is, the cellular paver. As the reference level moves lower down within the foundation, the impact area increases.
The resulting stress was calculated as follows, where the cylindrical weight of the material above the impact area has also been included in stress calculation.

\[
q_p = \frac{9.5 \text{ tonnes} + \text{Cylindrical weight of the soil above the impact area}}{\text{Impact area at the specified depth}} \quad \ldots \text{Equation 8}
\]

The safety factor (SF) against shear failure was calculated from the ratio of ultimate bearing capacity to the stress due to surface load as follows:

\[
SF = \frac{q_u}{q_p} \quad \ldots \text{Equation 9}
\]

The larger the value of the safety factor above one, then the greater the margin of safety against failure of the foundation subjected to these propped leg loads.

### 6.2.3 Safety factors for PTF trial

The safety factors were calculated and are given for various theoretical approaches in Table 7 for the two most vulnerable structures that were to be built in the PTF trial. These structures comprised the thinnest subbase layer of 350mm on a clay subgrade of CBR strength 4 per cent, or undrained shear strength of 92 kPa. These foundations were surfaced by two sizes of pavers; namely, 0.40m x 0.60m and 0.40m x 0.44m for Grassguard 180 and Grasscel 125 respectively. Subbase only structures were modelled in a similar manner and the deduced safety factors are also included in Table 7.

#### Table 7 Safety factors against shear failure at different layer boundaries using different solutions

<table>
<thead>
<tr>
<th>Paver</th>
<th>Paver dimensions</th>
<th>Safety factors for the structures:</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>B (m)</td>
<td>L (m)</td>
<td>Subbase only</td>
<td>Subbase on clay subgrade</td>
<td>Terzaghi’s solution</td>
</tr>
<tr>
<td>Grasscel 125</td>
<td>0.40</td>
<td>0.44</td>
<td>6.1</td>
<td>4.2</td>
<td>5.8</td>
</tr>
<tr>
<td>Grassguard 180</td>
<td>0.40</td>
<td>0.60</td>
<td>9.7</td>
<td>4.9</td>
<td>6.5</td>
</tr>
</tbody>
</table>

All factors of safety were markedly greater than unity. Hence, the subbase itself should be resistant to internal shear failure. The thinner subbase on clay structure and, by inference, all similar structures built with thicker subbases, should also be resistant to shear failure. The analysis therefore indicated that the foundation designs in the PTF trial were more than adequate to support the loads applied by propped legs of fully loaded trailer units.
7 Design and construction of the Pavement Test Facility (PTF) trial

7.1 Design of the trials

7.1.1 Clay subgrade strength
The strength of the existing clay subgrade in the PTF was initially assessed for pavement design using the MEXE Cone Penetrometer. The Cone Index value (CI), as measured by the penetrometer with a 20mm diameter cone, was converted into in-situ CBR values using the following relationship:

\[ \text{CBR} \, (\%) = 0.046 \times \text{CI} \quad \ldots \text{Equation 10} \]

The results showed that the strength of the clay subgrade was reasonably uniform over the trial area to a depth of 1.5 metres. The subgrade CBR, using the relationship above, was estimated to be between 4 and 5 per cent. A CBR value of 4 per cent was therefore adopted for design of the pavements.

7.1.2 Experimental design
The trial was designed with a combination of two foundation designs and the two selected concrete modules. The design approach is summarised below.

- Three trafficked lanes were to be constructed using foundations of Type 1 subbase material (MCHW 1) only. The paving modules were to be bedded on a 25mm thick layer of sharp sand previously placed on top of the Type 1 material.
- Two thicknesses of subbase given in Table 6 were specified based on the designs of Figure 13 and Figure 14 and the strength of the subgrade as determined in Section 7.1.1. Both Grassguard 180 and Grasscel 125 pavers were to be laid on the thinnest foundation but only Grassguard 180 was to be constructed on the thickest foundation.
- Each lane was to be dynamically trafficked by at least 30,000 standard axles using a dual wheel load having a maximum axle load of 11.5 tonnes.
- A static load of 10.0 tonnes to simulate a propped leg load of a trailer was to be applied to the thinner structures after the dynamic loading by the rolling wheel assembly.

7.2 Construction of the trials

7.2.1 Clay subgrade.
After trimming the existing clay subgrade, it was compacted using 12 passes of a Rammax RW2900. The strength of the subgrade was then measured with the Mexe cone penetrometer at three offsets, at one metre spacings, in each of Sections 1, 2 and 3. These tests were repeated in Section 3 after the trafficking the pavement. The measurements were analysed as described in 7.1 and are summarised in Table 8. The results show that the mean CBR strength of the clay subgrade was 4.1 per cent prior to trafficking and, according to tests on Section 3, remained constant throughout the trafficking period.
Table 8 CBR strength of clay subgrade as measured by the Mexe penetrometer

<table>
<thead>
<tr>
<th>No. of tests in each section</th>
<th>Subgrade CBR (%) within the upper 150 mm:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Before trafficking</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
</tr>
<tr>
<td>Section 1</td>
<td>4.4</td>
</tr>
<tr>
<td>Section 2</td>
<td>4.1</td>
</tr>
<tr>
<td>Section 3</td>
<td>3.9</td>
</tr>
</tbody>
</table>

Notes  
1. 18 tests were carried out in Section 3 after trafficking.  
2. Standard deviation

In addition to the Mexe cone penetrometer tests, samples of the clay subgrade were taken using a core cutter and the CBR of the intact recovered material tested under standard laboratory procedures (BS 1377-4: 1990). The CBR strength results of these tests were 4.3, 4.0 and 4.4 per cent with a mean value of 4.2 per cent and justified the decision to use a CBR value of 4 per cent for the design of the trials.

7.2.2 Subbase

The Type 1 subbase material was obtained from Mountsorrel Quarry in Leicestershire. A sample particle grading shown in Figure 15 confirmed that the subbase complied with the specification (MCHW1, 2004). The subbase was compacted in 150 mm layers using 8 passes of a Bomag 120ADH. Dynamic cone penetrometer (DCP) tests carried out before and after trafficking showed the in-situ CBR strength of the confined Type 1 subbase to be consistently greater than 100 per cent. Table 9 shows the constructed subbase thicknesses, which were determined from optical levels, compared to the design thicknesses.

Table 9 Design and constructed thicknesses of the Type 1 subbase

<table>
<thead>
<tr>
<th>Paver</th>
<th>Section</th>
<th>Thickness of Type 1 subbase (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Design</td>
</tr>
<tr>
<td>Grassguard 180</td>
<td>1</td>
<td>580</td>
</tr>
<tr>
<td>Grassguard 180</td>
<td>2</td>
<td>350</td>
</tr>
<tr>
<td>Grasscell 125</td>
<td>3</td>
<td>350</td>
</tr>
</tbody>
</table>

The constructed thicknesses of the Type 1 subbase were in good agreement with the target values being within about ± 15 mm of the design thicknesses.
7.2.3 Cellular paver surfacings

Grassguard 180 was laid in Sections 1 and 2 and Grasscell 125 laid in Section 3. Both of these pavers require a 25mm layer of sharp sand to be placed on top of the Type 1 subbase. The grading of the sand used is given in Figure 16. This material was placed by hand, approximately levelled and then compacted with a hand operated vibratory plate compactor. After compaction, the sand was accurately levelled prior to laying the grass pavers. The pavers were then filled with a 50/50 mixture of sharp sand and top soil, which was vibrated into the voids using a plate compactor. The construction sequence is shown in Figure 17.
Figure 17 Construction of modular surfacing
8 Trafficking and performance monitoring of the PTF Trial

8.1 Outline
The trials were subject to both dynamic loading to replicate traffic and to static loading to simulate the loading of a supporting leg of a fully laden trailer when disconnected from its tractor unit. The performance of the pavement was determined from its deformation and the observation of the deterioration of the cellular pavers. The pavements were tested by the Falling Weight Deflectometer (FWD) to assess the contribution of the systems to the structural behaviour of the pavements. Finally, after trafficking, the pavements were lifted paver by paver to establish the cause of the observed performance.

8.2 Trafficking

8.2.1 Test programme
The dynamic trafficking was carried out between 13th January and 2nd February 2006. The majority of the trafficking was done with a dual wheel, travelling at 10 kph and loaded to 5.75 tonnes that is equivalent to an axle load of 11.5 tonnes. The loading was not completely ‘channellised’ as it was considered, given the size of the pavers in relation to the width of the dual-wheel assembly, that this procedure would not only be unrealistic but might also move the pavers before they had been ‘bedded-in’ by the early stages of trafficking. The dual wheel assembly was therefore indexed by ± 300mm during trafficking; a distance of approximately half the width of a dual wheel.

Figure 18 Trafficking of the PTF Trial
The trafficking with a steadily increasing load was carried out to the timetable shown in Table 10.

The number of wheel passes is converted to equivalent number of standard axles using a fourth power law in the absence of a damage law specifically developed for these structures. The cumulative traffic loadings carried by the whole pavement and the central sections of each pavement are given in this table. As the performance of all three pavements was better than expected, the trafficking of the two thinner pavements in their central sections was increased from the planned 30,000 sa to approximately 50,000 sa.
Table 10 Periods of traffic loading in the PTF trial

<table>
<thead>
<tr>
<th>Date</th>
<th>Wheel Load (KN)</th>
<th>No of applied passes</th>
<th>No of standard axles</th>
<th>Cumulative standard axles</th>
<th>Cumulative standard axles</th>
</tr>
</thead>
<tbody>
<tr>
<td>13/01/2006</td>
<td>40</td>
<td>60</td>
<td>60</td>
<td>60</td>
<td>40</td>
</tr>
<tr>
<td>14/01/2006</td>
<td>40</td>
<td>60</td>
<td>60</td>
<td>120</td>
<td>79</td>
</tr>
<tr>
<td>14/01/2006</td>
<td>40</td>
<td>240</td>
<td>239</td>
<td>359</td>
<td>237</td>
</tr>
<tr>
<td>14-15/01/2006</td>
<td>40</td>
<td>600</td>
<td>599</td>
<td>958</td>
<td>632</td>
</tr>
<tr>
<td>16/01/2006</td>
<td>40</td>
<td>960</td>
<td>958</td>
<td>1915</td>
<td>1264</td>
</tr>
<tr>
<td>17/01/2006</td>
<td>49</td>
<td>960</td>
<td>2156</td>
<td>4072</td>
<td>2687</td>
</tr>
<tr>
<td>18/01/2006</td>
<td>57</td>
<td>960</td>
<td>3949</td>
<td>8020</td>
<td>5293</td>
</tr>
<tr>
<td>19/01/2006</td>
<td>57</td>
<td>1440</td>
<td>5923</td>
<td>13943</td>
<td>9203</td>
</tr>
<tr>
<td>23-25/01/2006</td>
<td>57</td>
<td>3900</td>
<td>16042</td>
<td>29985</td>
<td>19790</td>
</tr>
<tr>
<td>27-29/01/2006</td>
<td>57</td>
<td>4800</td>
<td>19743</td>
<td>49729</td>
<td>32821</td>
</tr>
<tr>
<td>31/01-02/02/06</td>
<td>57</td>
<td>7200</td>
<td>29615</td>
<td>79344</td>
<td>52367</td>
</tr>
</tbody>
</table>

Note 1. Cumulative traffic loading at centre of the loaded lane

8.2.2 Optical levelling and straight edge measurements

Prior to trafficking, the surfaces of the grass pavers were permanently marked as can be seen in Figure 18. Optical levels were then taken at identical points, both across and along each of the trafficked sections, after each period of trafficking. The deformation of each pavement was then calculated as a function of the traffic carried. An example of the information gained can be seen in Figure 19, which shows the deformation transversely across the wheelpath at different positions along Section 3 at the end of trafficking.

![Figure 19 Transverse profiles recorded in Section 3 at the end of trafficking](image-url)
The mean deformation in each of the three pavements, as measured by optical levelling, is shown in Figure 20 as a function of traffic in the central section of each pavement.

![Figure 20 Average deformation for Sections 1 – 3](image)

These deformation results are consistent with the ruts measured with a 2 metre straightedge and wedge at the end of the trafficking period and recorded in Table 11. These results are discussed in more detail in Section 9.

**Table 11 Measurements by optical levels and by a 2m straight-edge**

<table>
<thead>
<tr>
<th></th>
<th>Section 1</th>
<th>Section 2</th>
<th>Section 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Levels</td>
<td>4.0</td>
<td>Levels</td>
<td>Levels</td>
</tr>
<tr>
<td>S/E</td>
<td>4.4</td>
<td>S/E</td>
<td>S/E</td>
</tr>
<tr>
<td></td>
<td>7.3</td>
<td>7.6</td>
<td>12.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>14.7</td>
</tr>
</tbody>
</table>

Note 1. S/E denotes 2 metre straight-edge and calibrated wedge

### 8.2.3 Performance of the cellular pavers.

#### 8.2.3.1 Grassguard 180

Apart from the flexing of the complete pavement structures under load, the Grassguard 180 pavers did not visibly move under dynamic loading. The pavers did, however, fracture by cracking and surface chipping. The proportion of pavers that fractured increased with traffic as recorded in Table 12. The types of fracture; that is, cracking or chipping, are illustrated in Figure 21 a to c.
Table 12 Occurrence of cracking in Grassguard 180

<table>
<thead>
<tr>
<th>Traffic loading (sa):</th>
<th>19,790</th>
<th>32,821</th>
<th>52,367</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 1</td>
<td>29</td>
<td>48</td>
<td>-</td>
</tr>
<tr>
<td>Section 2</td>
<td>24</td>
<td>40</td>
<td>60</td>
</tr>
</tbody>
</table>

8.2.3.2 Grasscel 125

Because of the chamfer on the units, in the direction of trafficking, the Grasscel 125 pavers ‘rocked’ under the moving wheel loads. This behaviour was evident throughout the trafficking, but did not constrain trafficking in the PTF. Module rocking, however, probably contributed to the increased deformation of the Grasscell 125 structure compared to the Grassguard 180 structure of similar thickness.

None of the Grasscel pavers cracked, although at the end of the trafficking period there were a few (less than 5 per cent) pavers that had been ‘chipped’ at their corners. This degradation, shown in Figure 21d, occurred where they touched each other on the non-chamfered side of the paver.
8.3 FWD testing

FWD tests were carried out during the construction and trafficking of the PTF trials. Tests were carried out;

- on the top of the compacted Type 1 subbase;
- on the top of the cellular pavers prior to observed cracking of the pavers; and
- on top of the cellular pavers after dynamic trafficking was completed.

On all three occasions, the FWD tests were carried out using three different applied loads to establish whether the stiffness of the structures were stress dependent. The stiffness of the foundation or pavement was expressed as its surface modulus. Further details of this method of analysis are given elsewhere (FEHRL, 1996).

The surface modulus at the top of the foundation or pavement (equivalent depth = 0mm) is calculated as:

\[ E_0 = 2(1-\nu^2) \frac{\sigma_o a}{\delta_o} \]  

Equation 11

Where

- \( E_0 \) = the surface modulus at the centre of the loading plate (MPa)
- \( \nu \) = Poisson's ratio (value of 0.35 used)
- \( \sigma_o \) = contact pressure under the loading plate (kPa)
- \( a \) = radius of the loading plate (mm)
- \( \delta_o \) = deflection at the centre of the loading plate (microns)

The averages stiffnesses of the foundations in each of the sections measured prior to construction of the pavers are given in Table 13. The results show that the stiffness of each foundation was not significantly stress dependent and also the mean stiffness recorded in each section increases with increase in the thickness of Type 1 subbase (See Table 9).

<table>
<thead>
<tr>
<th>Section No</th>
<th>Stress (kPa)</th>
<th>Stiffness (MPa)</th>
<th>Stress (kPa)</th>
<th>Stiffness (MPa)</th>
<th>Stress (kPa)</th>
<th>Stiffness (MPa)</th>
<th>Mean Stiffness (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>102</td>
<td>241</td>
<td>161</td>
<td>246</td>
<td>245</td>
<td>249</td>
<td>245</td>
</tr>
<tr>
<td>2</td>
<td>103</td>
<td>141</td>
<td>163</td>
<td>137</td>
<td>248</td>
<td>134</td>
<td>137</td>
</tr>
<tr>
<td>3</td>
<td>102</td>
<td>117</td>
<td>162</td>
<td>114</td>
<td>245</td>
<td>112</td>
<td>114</td>
</tr>
</tbody>
</table>

The mean foundation stiffness values recorded in Table 13 were then used to calculate the effective stiffness of the modular grass paver layers before and after substantial trafficking. This calculation was done by using the linear elastic programme BISAR (Shell International, 1998) to model the complete layered pavement as a surfacing layer and a foundation with the stiffnesses given in Table 13 and manually adjusting the stiffness of the grass paver layers until the calculated FWD central deflection matched the measured value. The results are shown in Table 14 at a stress level of approximately 700kPa. This is the stress level that is normally used to test flexible road pavements (Load 50kN on a 300 mm diameter plate).
Table 14  Stiffness of the grass pavers (FWD)

<table>
<thead>
<tr>
<th>Section</th>
<th>Foundation stiffness (MPa)</th>
<th>Stress (kPa)</th>
<th>Surface layer stiffness (MPa) After 5,293 sa¹</th>
<th>After 52,367 sa¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>245</td>
<td>694</td>
<td>300</td>
<td>300</td>
</tr>
<tr>
<td>2</td>
<td>137</td>
<td>669</td>
<td>150</td>
<td>95</td>
</tr>
<tr>
<td>3</td>
<td>114</td>
<td>661</td>
<td>100</td>
<td>70</td>
</tr>
</tbody>
</table>

Note 1. Cumulative traffic loading at the centre of the loaded lane

The results in Table 14 indicate that the measured stiffness of the cellular paver layers mirrors that of the underlying foundation. This observation indicates that the pavers act as individual units, without any significant interlock between adjacent modules. The initial assumptions in Section 6.1 that the modular surfacing can be considered to act as a good quality, unbound granular subbase is also justified as reasonable.

There was a small reduction in stiffness of the surfacing layer with trafficking. This behaviour is believed to be caused by either cracking of the Grassguard 180 pavers or voids created by the ‘rocking’ of the Grasscell 125 units by trafficking.

8.4 Static Loading

Static loading tests were carried out to simulate what would happen when a fully loaded lorry was parked on the cellular pavers and its tractor unit removed. The thinnest pavements in Sections 2 and 3 were tested to a maximum load of 10.0 tonnes that was applied through a purpose built loading platen which simulated a twin-wheeled support leg (See Figure 11).

The static load was applied to uncracked pavers by jacking against the gantry of the PTF as shown in Figure 22. The deflection under the incremental load was measured at two locations adjacent to the centre of the load and at off-sets to the applied load of 350 mm and 450 mm for Grasscell 125 and 400 mm and 600 mm for Grassguard 180.

![Figure 22 Static loading tests](image)

The resultant load/deflection graphs for Grasscell 125 and Grassguard 180 are shown in Figure 23 and Figure 24 respectively. The results show that the maximum deflection under the Grasscel 125 paver was approximately 3.6 mm, in comparison to 2.8 mm under a similar load on the Grassguard 180. This increase in deflection can be explained, in the main, by the smaller contact area of the Grasscel 125 compared to Grassguard 180 and indicated that the pavements were acting in an elastic manner.
There was no evidence of any shear failure in the Type 1 in either Sections 2 or 3. However, not surprisingly, the loaded Grasscel 125 and Grassguard 180 pavers both fractured during the test.

Figure 23  Deflection versus load test – Grasscell 125

Figure 24  Deflection versus load test – Grassguard 180
8.5 Forensic investigation after trafficking

After all the dynamic and static loading tests were completed, the grass pavers were removed from Sections 2 and 3 to determine the cause of the pavement deformation that had occurred during dynamic trafficking. The depth of bedding sand between the top of the Type 1 subbase and the bottom of cellular paver was then measured. These measurements showed that the majority of the deformation, measured on the top on the pavers (see Table 11), was not ‘structural’ deformation through an accumulation of strains in the Type 1 subbase and clay subgrade, but rather was caused by the pavers bedding into the sand layer. After this effect is taken into account, the ‘structural component’ of the recorded pavement deformation is more likely to be that shown in Table 15.

Table 15 ‘Structural component’ of the observed deformation (mm)

<table>
<thead>
<tr>
<th>Section 1</th>
<th>Section 2</th>
<th>Section 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>7</td>
</tr>
</tbody>
</table>
9 Summary and discussion

9.1 Project outcome

The primary activities and findings of the project were as follows:

1. Commercially available modular, cellular grass pavers were reviewed and a selection tested in the laboratory for strength. From amongst a number of suitable candidates, two pavers; namely, Grassguard 180 and Grasscel 125, were selected and built into a trial in TRL’s pavement test facility (PTF). The trial foundations were designed to carry 30,000 standard axles and to resist high loads from lorry trailers when disconnected from their trailer units.

2. The PTF trials have shown that, providing appropriate engineering standards are developed, modular, cellular pavers do provide a viable low cost surfacing option for safe havens and for emergency access to, and egress routes from, the trunk road network that can support heavy goods vehicles.

3. Theoretical analysis of the results from the PTF trials has shown that the effective stiffness of a cellular paver surfacing for pavement design purposes can be assumed to be similar to that of the underlying foundation.

4. The PTF trials have enabled foundation designs, surfaced with cellular pavers, to be developed for safe havens, emergency turnaround areas and other low volume, trafficked routes for traffic levels up to 50,000 sa. These designs will be further developed during the proposed trials described in Section 10.

5. The current specification for cellular grass pavers for paved areas is contained in Clause 1109 of Specification for Highway Works (MCHW1 November 2004). Amongst other requirements, this specification requires the bending strength of pavers to be measured according to BS EN 1339 and to be greater than 5MPa. Experience gained in using the test during the project has shown that the ‘breaking load’ would be a better parameter to specify because of the variety in design of grass pavers.

6. The ultimate modes of distress of the two paver systems used during the PTF trials were different. In the case of Grassguard 180 the pavers eventually began to crack after a significant amount of trafficking. In the case of Grasscel 125, the ‘rocking’ of the grass pavers under trafficking eventually resulted in increased ‘bedding’ into the sand and thereby increased surface deformation. Despite these behaviours both systems were still serviceable at the end of the trial. These different forms of distress should be taken into account during the development of appropriate engineering standards for grass pavers.

7. Excavation of the PTF trials has shown that a substantial proportion of the recorded surface deformation was caused by the grass pavers ‘bedding’ into the underlying sand layer. Revised engineering standards are required for the surface tolerances on the subbase material to control sand layer thickness, as a marked variation in thickness of these sand layers could cause differential ‘bedding’ into the sand layer and, therefore, an unacceptable surface profile after trafficking.

8. Static loading tests on the PTF trials have verified the theoretical analysis that predicted that a load of 9.5 tonnes, chosen to simulate the propped leg load of trailers, would not cause shear failure of the foundations to the selected pavers tested. In both cases, the individually loaded cellular pavers did fracture under the load.

9.2 Cellular pavements with enhanced drainage

The adoption of more permeable subbase than unbound granular Type 1 material within modular, cellular pavements has a variety of functions including:
a) Temporary storage of rainfall and run-off from paved areas.
b) Enhanced drainage to remove water from the structure by sideways drainage to pavement edge drains.
c) Enhanced drainage to improve vertical drainage to permeable subgrades, which could remove the need for pavement edge drains.

To maintain the drainage capacity of the subbase, permeable geotextiles can be placed on the subgrade to prevent the granular material being contaminated by the upward migration of soil subgrade fines. Also, a geotextile membrane could be placed on the subbase to prevent sand used for bedding the cellular pavers modules migrating downwards into the subbase.

The structural behaviour of such pavements under rolling wheel loads has not been investigated in this project for the reasons recorded in Section 4.4, but, if required, would require trials under a variety of moisture regimes to be implemented appropriately. Within these road trials, the adverse effect on the propagation of surface vegetation by the more rapid removal of water could also be monitored.
10 The next steps

10.1 Pavement designs for road trials

The trial in the PTF trial at TRL was carried out to provide a first stage validation of proposed structural designs for safe havens, emergency turnaround areas (ETAs) and other low volume, trafficked routes surfaced with grass pavers for design traffic up to 50,000 sa. Over the course of the trials it was shown that both of the selected cellular pavers used provided a viable surfacing. Revised specifications for the paving modules, however, would be needed to ensure that universal performance was satisfactory.

It was not possible during the PTF trial to assess the performance of cellular paver surfacings under the actual environmental conditions and harsher trafficking, such as ‘turning’ movements and impact loads that would be associated with use on the network. Therefore, the excellent performance of both the most and least conservative foundation designs in the PTF trials does not justify marked reductions in thickness design until proven in further studies. It is recommended that an intermediate design to those tested be assessed under conditions similar to actual road use. The recommended designs are shown in Figure 25 and given in Appendix A. A conservative adjustment factor of x0.75 to the subgrade soil strain criterion is maintained and, following the structural assessment in Section 8.3, the cellular modular pavers are assumed to have a structural contribution to the pavement equivalent to that of an equal thickness of good quality, unbound granular Type 1 subbase.

![Figure 25 Variation of subbase thickness with soil strength – Recommended for road trials](image)

Note The above designs are for a module thickness of 120 mm. For thinner modules, increase subbase thickness by the amount that maintains unchanged the combined thickness of subbase and module; and vice-versa.

Although designs are given for long-term subgrade soil CBR strength values as low as 1 per cent, it is advised that the procedures of IAN 73/06 are followed for soil strength of less than 2.5 per cent. Remedial work should aim for the long-term, not temporary, improvement in the strength of the upper subgrade soil above this threshold strength. For pavement design, the CBR strength is normally taken as a CBR value of 2.5 per cent, whatever the strength achieved.

In choosing materials for the upper layers of the foundation, the potential effect of frost heave should be considered. Dependent on the mean annual frost index, a depth of 350 or 450 mm of non-frost susceptible material is required for conventional pavement design. It would therefore be prudent for
these cellular pavements to ensure that materials used beneath the paving modules are not frost susceptible to this depth measured from the top of the pavement. This requirement may limit the use of some unbound granular capping materials and require subbase to be thickened. In Appendix A, the designs that may be affected by frost penetration to a depth of 450mm and 350mm from the pavement surface are identified.

The subbase on subgrade designs were theoretically examined as described in Section 6.2 against shear failure by loads applied to cellular pavers of various sizes by the prop legs of trailers disconnected from their tractor units. The sizes of the pavers were varied from 0.3m x 0.3m to 0.6m x 0.4 m. The safety factor against failure was about 2 or higher, for foundations designed for subgrades of CBR at least 2.5 per cent and for traffic of 2,000 sa and higher. This analysis suggests that these foundations will support loads applied by propped legs of lorry trailers.

10.2 Recommended research

Prior to the widespread use of modular cellular pavers on the trunk road network, it is proposed that there should be a ‘live’ trial adjacent to the network, such as, for example, an access road to a maintenance depot. It is recommended that the trial be used for the following investigations:

- Assessment of the performance of the grass pavers under the stresses caused by manoeuvring heavy vehicles.
- Further development of the pavement designs.
- Assessment of the performance of free draining subbase material (Type 3), as opposed to a less permeable Type 1 subbase material.
- Monitoring of the performance of individual modular grass pavers against their manufactured properties to enable a universal specification to be developed.
- Assessment of the relative performance of alternative vegetation covers.
- Measurement of the effect of vegetation growth on the skid resistance of the pavement’s surfacing.

11 Acknowledgements

The work described in this report was carried out by the Pavement Structural Assessment Team in the in Infrastructure and Environment Division of TRL Limited. The authors wish to recognize the contribution by P Scott and M Zohrabi (2000), who formulated the concept of the project and carried out the earlier work.
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## Appendix A: Subbase only designs recommended for road trials

Table A1 records the designs recommended for road trials. Thicknesses are given for subbase plus modules and separately for subbase only when used with modular, cellular pavers having a thickness of 120mm. The designs assume a structural contribution from the modules equal to that of an equivalent thickness of subbase and an adjustment factor of x0.75 applied to the subgrade strain.

### Table A1: Design thicknesses of subbase for various anticipated levels of traffic

<table>
<thead>
<tr>
<th>Design traffic (sa)</th>
<th>Soil CBR (%)</th>
<th>Thickness of subbase and modules (^{1,2}) (m)</th>
<th>Thickness of subbase only (^{1,2}) (m)</th>
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<tr>
<td></td>
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<td>Thickness of subbase and modules (^{1,2}) (m)</td>
<td>Thickness of subbase only (^{1,2}) (m)</td>
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Note 1. Corrected up to nearest 5 mm
2. When used in conjunction with module of thickness 120 mm

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Frost depth 450 mm
Frost depth 350 mm
Refer to IAN 73/06 for soils with low CBR strengths