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THE PERFORMANCE OF PAVEMENT FOUNDATIONS DURING CONSTRUCTION

By

Matthew W. Frost,
BEng (Hons)

A Doctoral Thesis

Submitted in partial fulfilment of the requirements for the award of Doctor of Philosophy of Loughborough University

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ABSTRACT

There is an impetus in the UK to move away from empirical pavement foundation design and the current method specification, towards an analytical design approach. For an analytical design approach to be adopted, the required performance properties of stiffness and resistance to permanent deformation of the foundation materials (subgrade and capping) need to be measured, both in the laboratory for design and in the field in order to ensure compliance.

This thesis studies the influence of the subgrade on the constructability and performance of a series of full-scale pavement foundations. This has been achieved by measuring the performance parameters of several subgrade materials in the laboratory, using repeated load triaxial testing. These data have been compared to comparable data collected in situ using dynamic stiffness measuring devices during the construction of trial pavement foundations. The performance of the trial foundations has been measured during the placement and compaction of the different foundation materials, and again after their subsequent trafficking.

The testing demonstrates the stress dependency of the foundation materials. The laboratory testing shows that the subgrade permanent deformation under cyclic loading (used to simulate construction operations) becomes unstable at a deviator stress of half the deviator stress at failure ($0.5q_{\text{max}}$). The stiffness at this applied stress and above is shown to be approaching a consistent value. This indicates that large changes in the stiffness of inversely stress dependent fine grained soils occur below the deviator stress at which the permanent deformation becomes unstable. Significant variability of data has been found in the performance parameters measured (both in the laboratory and in the field) for samples of subgrade collected from small areas of the same site. However comparable patterns of stress dependency have been observed between measured laboratory and field performance using the different apparatus.

The resistance to permanent deformation is shown to be a more critical design load case for construction than the need for adequate stiffness of support required to compact the foundation layers. The performance of a composite road foundation is shown to be material and site specific, and this will have important implications for design and site compliance testing.

Keywords

Pavement Foundations, Performance Assessment, Stiffness, Permanent Deformation, Repeated Load Triaxial
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1.0 INTRODUCTION

1.1 Background

Statius, poet laureat to the Roman Emperor Diocletian (AD 81-96), described the construction of a Roman road:

*The first stage of the work was to dig ditches and to run a trench in the soil between them. This empty ditch was filled with other materials (flat stones or a sand and mortar course) and a foundation was prepared to carry the pavement. For the surface should not vibrate otherwise the base is unreliable or the bed in which the paving stones are to be rammed is too loose.* (Hill 1996).

So the need for adequate drainage and the provision of a good foundation to provide an adequate platform upon which to construct the road pavement has been appreciated since Roman times.

To achieve this today UK road pavements are traditionally designed in two stages, using a series of empirically designed road foundation layers (normally constructed of granular or stabilized materials) which provide a foundation upon which to construct a series of structural pavement layers.

The road foundation layers consist of capping (where necessary) and sub-base that overlie the natural soil subgrade. The capping layer is a subgrade improvement layer, and is regarded as a construction expedient to facilitate good compaction of the overlying layers. The capping is traditionally a low cost granular material that is ideally won locally or made by stabilising the subgrade with either lime or cement or both. Frequently it is an imported material. Sub-base is normally an imported, well-graded, good quality, granular material, and is regarded as a structural layer. It acts as a regulating course upon which to compact the bound layers, (Figure 1.1).
The current UK specification for road foundations is based on a recipe approach, whereby selected materials are laid and compacted with specified plant in a specified manner to achieve an assumed minimum level of performance. The pavement foundation designs are based primarily on the use of the California Bearing Ratio (CBR) to characterise the subgrade, capping and sub-base materials. Although the use of CBR as a performance parameter is widely acknowledged as being not wholly satisfactory, CBR has been correlated with pavement performance in many countries over many years and provides a trusted empirical indicator of material behaviour. Such an approach based on CBR is unlikely, however, to result in the efficient use of materials and plant, does not easily allow for the use of recycled, new or marginal materials and does not permit the rigorous use of analytical design procedures.

The pavement foundation performs two structural functions both during construction and when in-service. In particular it acts as load-spreading layers to reduce to acceptable levels the stresses transmitted to the subgrade, often as temporary haul roads during construction (which is often the critical load case), and as a construction base on which the overlying pavement layers can be adequately compacted. In addition it acts as a frost protection layer.

To enable the pavement foundation to fulfill the structural requirements the foundation materials must possess two specific properties. They must have adequate resistance to permanent deformation (i.e. individual and composite strength) to prevent the accumulation of permanent strain within the foundation itself, and they must reduce the applied stress transmitted to the subgrade to such a level that the subgrade does not deform, generating a rut both during construction and in service.

To enable the subsequent layers to be compacted the materials must possess sufficient resistance to resilient deformation (stiffness). In the long term the foundation must also possess sufficient stiffness to prevent excessive resilient deformation leading to premature flexural fatigue cracking of the structural layers.
Recent technical advances in the in-situ testing of pavement foundation materials now allows the performance parameters of stiffness and, more indirectly, strength and resistance to permanent deformation to be assessed. Laboratory techniques have also been developed independently to assess the performance properties of fine grained subgrades.

To allow a performance based design and specification approach to be introduced for road foundations the material performance parameters need to be measured prior to construction to enable design. These parameters can then be used in a performance specification to derive a design thickness and set target values for the required performance parameters that can be assessed on site to ensure the appropriateness of the design. If a performance based design and specification can be implemented the use of previously untried materials, such as recycled materials and waste products can be made, hence reducing the use of good quality quarried stone. In addition by directly measuring the performance parameters of the materials as they are constructed into pavement foundations greater efficiency of site operations is also anticipated.

1.2 The Need for the Research

As described above road foundations are currently constructed to a recipe specification based on long-established, empirically-based designs. The performance of the foundation is seldom tested during or after construction.

Considerable research has been undertaken over the past few years to develop portable dynamic stiffness measuring devices that can quickly measure the stiffness of pavement materials (both the subgrade and the pavement layers) during the construction. Separate research has also been undertaken to develop repeated load triaxial testing equipment that can directly measure routinely, in the laboratory, the performance of samples of fine-grained subgrade materials under similar loading conditions experienced in a pavement. Similar reliable laboratory assessment tests on the performance of coarse granular materials suitable for routine use are still to be developed.
Little work has been performed with the developed test apparatus in practical applications, the majority of the research having concentrated on developing the test equipment itself. Therefore research is required to study the data collected from the laboratory tests on fine-grained soils typically found as UK subgrades, at the environmental conditions experienced during the life of a road pavement. Additionally the on-site performance of road foundations and the change in the performance of the foundation during construction need to be established. If a link between the two sets of (laboratory and field) tests can be established then use of the laboratory-defined subgrade performance parameters can be made to assess the influence of the subgrade on the overall foundation performance and hence directly in road foundation design. Thus the current empirical design and recipe specification approach can be revised to incorporate functional material performance characteristics leading to fully analytical pavement design.

1.3 Aims and Objectives of the Research

The aim of this thesis is to assess the influence of the subgrade on road foundation performance during construction.

The hypothesis behind road foundation design using virgin aggregates is that by removing the adverse influence of (fine-grained) subgrades, where they are not inherently up to the required standard to support the structural pavement layers, by adding granular materials of appropriate thickness and properties enables these layers to be adequately constructed and used for construction operations without damage to the subgrade below.

The thesis objectives are as follows:

- Produce a critical review of research undertaken to establish the factors which affect the performance of road foundations and the materials used to construct them, and to
identify available methods to measure that performance in the field and in the laboratory.

- Assess the performance of fine grained-soils, typical of those found in UK subgrades, at the states they may experience during construction of a pavement foundation.

- Measure the change in the road foundation performance parameters under controlled conditions, starting at the surface of the subgrade immediately prior to addition of granular material, with increasing foundation layer thickness and compactive effort for different foundation materials.

- Assess the relationship between the methods used to assess the materials in laboratory and field to determine any correlation between the tests, to facilitate analysis of the data and thereby establish the influence of the subgrade on the composite field measured foundation performance.

1.4 The Research

The research work described herein formed part of a research project undertaken by Loughborough University in association with Scott Wilson Pavement Engineering Ltd. and Nottingham University. The research was commissioned and funded by the UK Highways Agency to produce a performance-based specification for subgrade and capping, to replace the current empirical CBR-based road foundation design and specification.

The research work discussed in this thesis can be split into two main elements.

- laboratory testing of subgrade samples and
- controlled field trials,
Samples of the subgrade, granular capping and sub-base materials used in the field trials were collected and standard classification tests performed upon them. Samples of subgrade collected from the field trials, as well as additional samples of typical UK subgrade materials, were tested using various sample preparation techniques and test methodologies in repeated load triaxial test apparatus at Nottingham University. As well as providing information on various laboratory tests regimes and test repeatability, the tests provided subgrade performance data (both stiffness and permanent deformation) to compare to the subgrade performance measurements made on site.

Four controlled field trials were built in which short lengths of pavement foundation were constructed using typical capping materials. The material type, construction thicknesses and compactive effort applied were varied from trial to trial. The performance parameters of the subgrade and capping were measured at various stages of construction. In three of the trials sub-base was laid upon the constructed foundation and the performance parameters of the sub-base measured during its installation. The trials were subsequently trafficked with a laden lorry to assess their resistance to permanent deformation.

An assessment of the variability of the various test results has been made under both controlled and practical conditions. Error assessment tests were performed in the repeated load triaxial test apparatus utilizing rubber dummy triaxial test specimens. Samples of a similar rubber to that used in the triaxial tests were assessed with the field performance tests.

1.5 Thesis Structure

The thesis structure is based upon the parallel testing undertaken in the laboratory and in the field. It is initially structured to place the work in context of the research currently undertaken in this field. It subsequently reports the results from the laboratory and field testing, and draws together the information obtained from the comparable laboratory and field performance measurements to assess the link between the two sources of data,
which is required for analytical pavement foundation design. Finally it assesses the influence of the subgrade performance on foundation performance observed in the field.

The thesis is divided into seven chapters. A review of literature is included in Chapter 2. This chapter introduces the principles behind road foundation design and details the current design philosophy used in the UK and its limitations. It details road foundation loading and the performance parameters that need to be quantified if the design of road foundations is to move to a more analytical approach. It then reviews the methods by which these parameters can be measured in the laboratory and in the field. It discusses relevant research using the test methods previously reviewed.

Chapter 3 explains the philosophy behind the research methodology. It also details why the chosen methods have been adopted in preference to others available.

Chapter 4 reports the detailed research methodology. The repeated load triaxial test apparatus, the test regimes and the sample preparation methods are described. It explains the process of construction of the field trials, and the performance measurements made. Error assessment experimentation is then described.

Chapter 5 presents the data recorded during the various programmes of experimentation. Chapter 6 discusses these data and the associated errors, comparing the various field and laboratory tests, and assesses the data to establish the influence of the subgrade on the performance of the pavement foundations constructed.

The conclusions from the research and suggested future work arising from the research findings are discussed in Chapter 7.
Surfacing

Base

Sub-Base

Capping

Subgrade

Skid Resistant Surface

Main Structural Element
Bound material. High stiffness, and crack and deformation resistance

Foundation
Selected granular material over soil. Adequate platform to place layers above.

Figure 1.1. Structure and Function of a Typical Flexible Pavement.
2.0 LITERATURE REVIEW

2.1 Introduction

This chapter aims to review the current understanding of the requirements of road foundation performance and the methods available to measure that performance.

It explains the loading experienced by a pavement both during construction and in service, and it then goes on to describe the functional requirements of pavement foundations needed to resist that loading. The current philosophy of UK pavement design is explained and the position of foundation design within that philosophy described. Details of the materials currently used to construct pavement foundations are then given, followed by an explanation of both the empirical and analytical approaches used to design the foundations. The behaviour of foundation materials under load is then explained and the factors that affect that performance are described. Performance is described both for the individual material layers and their composite performance in a pavement.

Methods developed to measure the performance parameters are described for both laboratory and field assessment. The limited performance data collected from the trials are then discussed, along with the comparisons made between laboratory and field derived performance data. The available methods of data analysis are also described. Finally, conclusions from the literature are then made.

2.2 Pavement Loading

The loading applied to a pavement can be summarised as follows.

- A relatively small number of large stress applications, applied directly to the foundation surface by construction vehicles very soon after it has been constructed (i.e. used as a haul route).
- A relatively small number of large stress applications, caused by compaction of the overlying layers during construction (to produce strong/stiff layers above).

- A very large number of small stress applications applied indirectly via the bound layers over several years after construction, when the road is in service.

This loading is very complex, it can vary in magnitude and duration of applied stress (vehicle load and speed), direction of applied stress (direction of travel), period between stress applications and due to the relative position and loading of adjacent vehicle wheels.

The effects of loading on a small element of a pavement by the passage of a loaded single wheel in terms of the transmitted stress pulse is shown in Figure 2.1 (after Brown, 1996). This shows a pulse of vertical and horizontal stress with an approximately sinusoidal (double) pulse of shear stress (with a sign reversal occurring on both planes). This stress pattern causes the element of the pavement to be subject to a rotation of its principal stresses (Figure 2.2), as opposed to a reversal in the shear stress caused by the application of a pulsed vertical load (Figure 2.2) (Brown, 1996).

2.2.1 Vehicular Loading

Barksdale (1971) researched vehicle load pulse duration at various depths in a (full) pavement for various vehicle speeds (Figure 2.3). This figure shows that the deeper within the pavement, the longer the stress pulse lasts. For instance, a vehicle travelling at 30 mph generates a pulse of 40 milliseconds at the pavement's surface, 60 milliseconds at 250 mm depth and 110 milliseconds at 600 mm depth below the surface.

Similarly the lower in the pavement layer, the lower is the applied stress, due to the load spreading capabilities of the pavement (Figure 2.4). Figure 2.5 (after Brown, 1996) shows the passage of a series of wheels over a 165 mm asphalt pavement (supported on 150mm
of granular material) indicating a maximum subgrade stress of 15kPa, and a load pulse duration of approximately 0.6 seconds. The magnitude and duration of a wheel load from a laden lorry over a 350mm granular haul road, measured at the subgrade, is shown in Figure 2.6 (after Little, 1993). This shows that the stress measured at the subgrade is approximately 100kPa, with a load pulse duration of less than 1 second. Thus it can be seen that the most serious traffic loading condition occurs during construction.

The loading that the pavement layers sustain can be broken down into two elements: the stresses applied and the number of passes of that applied stress. These two elements have been simplified for design to the number of vehicle passes of a standard axle load.

Powell et al. (1984) defined typical loading for both the construction and in-service load cases. For the construction case they state that typical construction traffic using the foundation as a haul route to be equivalent to the passage of between 100 and 1000 standard axles. For the in-service case the loading becomes the predicted number of standard axles over the design life of the pavement (i.e. many millions of standard axles).

A standard axle (LR883, 1978, Kennedy and Lister) is defined as 80kN over two wheels, giving a load of 40kN per wheel over an assumed contact radius of 0.151m, giving a contact stress of 560kPa for a single wheel. For a dual wheel arrangement each tyre will carry 20kN, with a typical tyre pressure of 480kPa for a dual wheel, giving a contact area of 0.035 m². Other more complex interactions between loads occur when dual wheels or series of axles pass in succession. The standard axle and applied stress described above are normally adopted as the standard loading arrangement for pavement design, with heavier and lighter axle loads being resolved to a standard axle using the fourth power law for design purposes.

\[
EF = (W/40)^4
\]

Where EF = Equivalence factor to standard axle, W = Wheel load (kN) (half an axle for single wheel arrangement).
A maximum axle load of 110kN is permitted in the UK, and this is perhaps a more likely axle load for construction material delivery lorries. This equates to 3.6 standard axles.

2.2.2 Loading During Compaction

The stress applied by compaction plant varies according to the type of plant used to compact (roller or plate compaction), and the frequency of the vibrations of the plant. Stresses measured at shallow depth in a granular material during compaction with twin drum rollers indicate a 1 tonne roller to generate a contact stress of 300-400kPa at 75mm depth (Figure 2.7, after Parsons, 1992). A 7 tonne roller generates a stress of 400 to 600kPa at 130mm depth (Figure 2.8, after Parsons, 1992). To comply with the current UK specification (Volume 1, Manual of Contract Documents for Highways Works (MCHW), 1994) between 3 and 8 passes of the roller are required, depending on the specific plant used, the material to be compacted, and the layer thickness. Therefore the stresses experienced during compaction are higher than those experienced from wheel loads during trafficking. Thus greater support from the pavement is required to resist these stresses, the magnitude of which vary within the pavement due to the pavement materials’ properties (Section 2.5)

2.3 The Functional Requirements of Pavement Foundations

To resist the applied loading defined above, a pavement foundation must fulfil the following mechanical functions:

i) It must support construction vehicles during the construction of overlying layers. The foundation must not deform significantly (resilient deformation) under trafficking as to reduce the effectiveness of the compacted structure. It must dissipate the applied stresses from the wheel loads to a sufficiently low level to ensure that the subgrade does not sustain significant permanent deformation
(rutting). If excessive rutting of subgrade occurs, ponding of water may occur within the subsurface rut, which may in turn lead to further softening and weakening of the subgrade in the long term.

ii) It must provide an adequate base for the placing and compaction of the overlying layers, (i.e. not deform excessively so reducing the effectiveness of the compacted structure).

iii) It must provide adequate support to the overlying bound layers when the road pavement is in-service and distribute the stresses transmitted through the bound layers to reduce the applied stress to the subgrade to a sufficiently low level. If not, flexural fatigue cracking of the upper layers can propagate and the progressive accumulation of permanent strain (rutting) after a large number of small stress applications may lead to deterioration of the complete pavement.

In addition the materials used must possess chemical and physical stability in the long-term and the overlying materials must provide frost resistance to the subgrade. These durability parameters, although important, are not investigated in this thesis.

To perform these mechanical functions the materials used in pavement foundations must possess the two primary performance parameters of adequate stiffness and resistance to permanent deformation as defined in Section 1.1.
2.4 Pavement Foundation Design

2.4.1 Current UK Road Foundation Design.

Current UK pavement foundation design is based on LR1132, The Structural Design of Bituminous Roads (Powell et al., 1984). This document describes the philosophy for design of all standard flexible UK pavements and is the basis of the design information given in the current UK pavement design manual (Volume 7, Design Manual for Roads and Bridges (DMRB), 1994). LR1132 suggests the two stage approach to pavement design described in Section 1.1.

There are two long-term design criteria in LR1132. Firstly, the pavement should restrict the subgrade deformation to an acceptable level (to limit vertical strain applied to the subgrade). Secondly, the pavement should not crack under trafficking at the base of the road-base, or at the base of a cement bound lower layer if present (to limit the horizontal tensile strain in this layer). This is illustrated in Figure 2.9. However, it should also be noted that cracking from the top of the structural layers downwards can occur, with cracks propagating from the shoulders of ruts if excessive permanent deformation occurs (Croney, 1997).

In order to predict these strains at the design stage, knowledge of both the stiffness and strength behaviour of the materials included in the pavement foundation is required. However, adequate foundation performance is currently assumed, and foundation thickness design is based on the California Bearing Ratio (CBR) test on the subgrade. The design utilises long-established empirical relationships that relate capping thickness and CBR to observed road performance. This information is used to create the pavement foundation thickness design curves included in the current UK pavement design manual (HD25/94, Volume 7, DMRB, 1994).
The design CBR is normally derived from the equilibrium (soaked) CBR test (BS 1377, Part 4, 1990). However, correlations have also been proposed that empirically link the CBR of subgrades to measurable parameters such as Consistency Index (CI), related to soil suction and hence strength, by regarding the CBR test as a bearing capacity type of test (Black, 1962). This allows the use of soil plasticity data to derive a design CBR value (without performing a CBR test), and this approach is also included in the design guide.

Once a foundation design thickness has been derived, the foundation is then constructed in accordance with the current recipe specification (Volume 1, MCHW, 1994), whereby specified materials are compacted with specific plant for a certain number of passes.

2.4.2 Road Foundation Materials

2.4.2.1 Capping

If the value of subgrade CBR chosen for design is low (<5%), then an additional subgrade improvement layer known as a capping layer is provided. Capping is used to provide an adequate working platform upon which to construct the sub-base. It is regarded as an earthworks material, and is used as a short-term expedient for construction. It is not considered to contribute significantly to structural performance, but it plays a role in the long-term performance of the pavement structure since it forms part of the total road construction.

In current UK pavement foundation design it is assumed that capping is not directly trafficked. However, in practice it is frequently used as a haul route, both during the construction of the capping itself and of the subsequent layers. The sub-base is the layer that is assumed to be trafficked in current design and a limit of 40 mm maximum surface rut depth is set (Powell et al, 1984). This ensures that the subgrade does not sustain excessive rutting during construction (assuming the passage of a maximum of 1000 standard axles).
Capping materials are described in the MCHW (1994) (Volume 1, Clause 613). Granular materials are currently specified in terms of material grading and durability requirements. They are covered under two classes of material, 6F1 fine capping (graded from 75 mm particle size down) and 6F2, coarse capping (graded from 125 mm particle size down).

The use of stabilised and recycled materials as capping is also permitted in the current UK specification. Stabilisation of suitable subgrade with either lime or cement or both to form capping is covered in MCHW (1994) under clauses 614 and 615. More recently in a revision to the specification the use of recycled materials as capping has also been incorporated into the specification under Class 6F3. Materials such as crushed concrete and recycled bituminous planings (Fleming, 1998) are permitted, but they must comply with the specification clauses for standard granular cappings. Detailed consideration of these two types of materials is outside the scope of this thesis.

2.4.2.2 The Sub-base

The sub-base is regarded as a structurally important layer that forms a working platform upon which to transport and compact the structural layers. It also acts as a level regulating course to set carriageway levels and tolerances. Unlike capping, sub-base is regarded as acting as part of the pavement structure (Powell et al, 1984).

To achieve the functions described above, the sub-base needs to be a stiff and strong layer, therefore the material used as sub-base is a tightly-graded good quality granular material (maximum specified particle size of 37.5 mm) and is normally an imported material. Sub-base is specified in MCHW (1994) in Clause 801 (Volume 1). Similar to capping it is constructed using a method specification.
2.4.3 Empirical Design

The CBR test measures the resistance of a sample of subgrade soil to a 51mm diameter plunger forced into the sample at a rate of 1mm/minute. The CBR is evaluated by comparing the load required to cause two specified penetrations (2.5 and 5.0 mm), relative to the comparable performance of a standard crushed rock. For the laboratory based test samples are remoulded and re-compacted, often at worst case water contents (equilibrium). The sample is prepared and tested in a standard mould of 152mm diameter by 127mm deep (Figure 2.10, see BS1377, Part 4, 1990). The CBR test can also be performed in situ with the apparatus being mounted on vehicle to provide dead weight and the plunger forced into the soil under test. Consequently the in-situ test has different confining conditions to the laboratory based test and is seldom used for design purposes (Croney, 1997).

The CBR is therefore a (relative) measure of the resistance to penetration of the material. This is a function of the strength and stiffness of the material under test (i.e. for a strong material such as a crushed rock it is more ‘stiffness related’, whereas for a wet clay it is more ‘strength related’) (Croney, 1997). However, where its measurement is more ‘stiffness related’ it infers stiffness at much larger strains and at a lower strain rate than that caused by a wheel load and does not model the application of repeated load pulses that occur under trafficking (Brown et al, 1990). The test does not guarantee failure of the material under the plunger, and cannot be regarded as a true measure of strength either (Hight and Stevens, 1982). Nonetheless, the CBR test is quick and inexpensive and with the vast experience of its use it has traditionally been used (Volume 7, DMRB, 1994).

For analytical pavement foundation design (which is allowed in the current UK design specifications), the CBR has been empirically related to stiffness via the equation:

$$E = 17.6 \times (CBR)^{0.64}$$
This equation is a lower bound solution and is valid for CBR values between 2 and 12% (Powell et al., 1984). Other CBR/stiffness correlations have been proposed for different materials and over different ranges of subgrade CBR (for example Heukelom and Klomp, 1962). The stiffness value calculated is used to derive design thicknesses via static linear elastic analysis using assumed capping and sub-base material parameters that limit the critical design strains defined in Section 2.4. (e.g. Valkering et al., 1978, Shell Pavement Design).

Brown (1996) states that these correlations are unsatisfactory. Soil stiffness varies with soil type, applied load, stress history and with rate of load application, due to the influence of deviator stress, soil suctions and soil type, therefore there can be no unique relationship between CBR and resilient modulus (Brown et al., 1990). Sweere (1990), could find no correlation between CBR and stiffness for a range of granular materials.

2.4.4 Analytical Design

The methods by which the pavement structural layers are designed have moved to a more analytical approach, encouraged by changes in the way highway contracts are procured by the Government. The drawbacks of the CBR test have restricted the full use of analytical design. The CBR test, being an index type test, does not directly measure any of the fundamental material properties required for analytical design at appropriate loading.

These problems indicate that if a pavement performance type approach is to be adopted, then tests are required which can relate measured functional subgrade and capping parameters to specified performance targets. These tests would enable appropriate material behaviour to be used in analytical pavement design, allowing the actual pavement foundation performance to be incorporated in an overall pavement design, rather than foundation performance being assumed and the structural layers being designed separately.
Therefore if methods can be found that can measure the required parameters of stiffness and resistance to permanent deformation under similar loading conditions to those experienced in a pavement, both in the laboratory and in the field, then analytical design can be implemented. To define such tests the nature of the loading and behaviour of the materials under that load need to be considered.

2.5 Material Behaviour

2.5.1 Introduction

There are two aspects of material behaviour relevant to the performance of pavement foundations. Firstly, there is the individual material behaviour (how each material behaves separately under load). Secondly, the composite behaviour of two or more layers of materials acting together, whereby the behaviour of one material affects the performance of the other. Given the stress dependency of the materials used in a road foundation (Boyce, 1978), an understanding of the composite performance is critical.

In the previous section the performance parameters of the materials and the nature of the loading imposed upon them at the critical design conditions have been explained. This section aims to define the behaviour of typical pavement foundation materials and the factors that affect that behaviour. Since the two performance parameters are largely independent of one another, and granular and fine grained (cohesive) materials behave differently, these aspects are dealt with separately below.

2.5.2 General Behaviour of Materials Under Cyclic Loading

Figure 2.11 shows the (exaggerated) behaviour of a material subject to a single cycle of deviator stress in a triaxial cell. This shows two elements of the deformation behaviour of the material. Upon loading the sample sustains a deformation (P+R). Upon unloading some of the initial deformation is recovered (resilient deformation, R), and a small element of permanent deformation (P) is sustained. The materials that are used in road
foundations are not truly elastic but typically are considered so for small changes in stress. Their elastic properties can thus be defined by two constants, elastic modulus ($E$, the ratio of stress to strain) and Poisson's ratio ($\nu$, the ratio of horizontal strain to vertical strain).

From the two deformations described, the performance parameters of resilient elastic modulus (stiffness) and permanent strain can be calculated. The resilient elastic stiffness is calculated from the resilient strain and the change in stress measured on unloading (Seed et al, 1962). The permanent strain is found from the permanent change in length of the sample caused by the cumulative loading.

2.5.3 Resilient Elastic Stiffness of Coarse Granular Materials

Figure 2.12 (after Thom, 1988) shows the behaviour of a well-graded sample of granular material subject to a series of axial load pulses in a repeated load triaxial test. This shows a number of crucial elements of material behaviour. The first is the stress-dependency (or strain-dependency) of the initial loading curve, showing that an increasing shear stress causes a progressive increase in shear strain. Secondly, the material appears to be approaching its failure stress on the first loading cycle (due to the flattening of the stress-strain curves). If the shear stress applied had been reduced then the shear strain would have been lower and the gradients of both the first unloading curve and the subsequent loading-unloading hysteresis loop would have been greater (i.e. the value of stiffness would have been considerably higher). Thus any test that induces a significant permanent strain on loading or takes the material close to its failure condition will result in artificially low values of stiffness being measured (Thom, 1988).

This figure indicates that any resilient deformation measurement that relies on small stress applications can result in relatively large values of stiffness being measured, and that the value of stiffness calculated depends upon the magnitude of the stress applied and the previous loading sustained. The continued application of loading cycles thereafter
causes a progressive stiffening of the material, as evidenced by the steepening of the hysteresis loops.

The constantly changing gradient during each load cycle shows the change in the resilient behaviour of the material with changing stress. The increase in stiffness with increasing number of stress applications has been attributed to an increase in the effective stress under load that "prestresses" the material, inhibiting the formation of tensile stresses and hence mobilising higher stiffness (Brown, 1996). Consequently, the principal factor in the measured resilient properties of granular materials is the stress level (Hicks and Monismith, 1971, Sweere, 1990). These authors showed that the stiffness (measured in a triaxial cell) increased significantly with increasing confining pressure and slightly with repeated deviator stress, as long as shear failure was not approached. The following simple relationship to predict material behaviour (known as the K-θ model) was proposed.

\[ M_s = K_1 \theta^{K_2} \]

Where \( K_1 \) and \( K_2 \) are material parameters, and \( \theta \) is the sum of the principal stresses \((\sigma_1 + 2\sigma_3)\). This formula although describing the general behaviour of granular materials cannot be used to predict precise behaviour during loading. It does not incorporate a number of the other factors that affect behaviour such as Poisson’s ratio, which is itself stress dependent and changes significantly when dilation of materials is occurring (Brown and Hyde, 1975). Therefore other more complex models have been proposed which include various other variables such as Poisson’ ratio, or stress ratio (Lekarp et al 2000), or alternative approaches (Boyce, 1980) that used a volumetric strain approach. However, the K-θ model is regarded as an adequate simplification for design purposes where other factors, such as adjacent material and layer parameters, affect response (Brown, 1996).

Several other physical material properties that affect the resilient response of granular materials have been identified, such as the material type, its particle size, water content,
surface characteristics (angularity and roughness) and the compacted density (Hicks and Monismith, 1971). However, the effect of density has been shown to reduce with higher fines content and to only be significant at low deviator stress (Barksdale and Itani, 1989). Chen et al (1994) carried out tests on a variety of materials using various test methods. They showed that the resilient response of materials varied according to the source and type of aggregate, and showed the influence of the test regime stress sequence on the material response seen.

Rada and Witzac (1981) performed tests on six types of granular materials and found that the resilient response of the materials was significantly affected by the state of compaction (i.e. compacted density). They also showed a change in material response due to variations in the water content of the material (degree of saturation), but the magnitude of this influence varied from material to material. Grading of the materials was also shown to play a role in the resilient response. Materials with higher fines content tend to show a lower stiffness due to the loss of interlock between larger particles, although a small amount of fines can improve stiffness as it can fill the voids within the matrix (Jorenby and Hicks, 1986, as reviewed by Lekarp et al, 2000). Consequently the absolute particle size will also play a role as the effect of fines will vary with maximum particle size, but again this will also be related to material type, shape and grading.

Barksdale and Itani (1989) showed a change in material resilient response with a change in the particle characteristics of roughness, shape and angularity. This indicated that better particle interlock improved the resilient response. This will also be related to material grading and water content (confirmed by Thom, 1988).

Brown and Hyde (1975) showed that the load history did not affect the resilient response of the materials, but did affect the permanent strain. This was also observed to be true for materials where the magnitude of the loading applied was near to that which would cause failure (Selig, 1987). Tests undertaken varying the frequency of load application (from
0.2Hz to 20Hz) showed no significant change to the material’s resilient response (Boyce et al., 1976).

The tests performed to assess the changes due to material parameters, described above, were all performed using repeated load triaxial testing. These do not model the rotation of principal stresses that are experienced in road pavements. Significantly, Chan (1990) showed that the values of stiffness measured incorporating rotation of principal stresses (using the Hollow Cylinder Apparatus, HCA, Section 2.7.1.1) varied little from those found from repeated load triaxial tests.

Powell et al. (1984) suggest capping stiffness to be between 50 and 100MPa and approximately 150MPa for sub-base beneath a full pavement where applied stresses are lower than during construction. Brown and Dawson (1992) suggest stiffness values of between 200 to 400MPa for granular foundation materials during construction. Various values of stiffness for granular materials have been found in triaxial tests, for instance Hughes (1997) found values of stiffness of between 200 and 800MPa for different granular materials. However the stiffness of material measured in the triaxial cell will be significantly affected by the test methodologies used (Hughes, 1997). From such triaxial tests values of $K_1$ have been reported as being typically between 2000 and 6000 and $K_2$ between 0.4 and 0.7 (Hicks and Monismith, 1971; Brown, 1974; Hughes, 1997) however these values will be significantly affected by the shapes of the curve plotted from the collected data.

2.5.4 Permanent Deformation Behaviour of Coarse Granular Materials

Figure 2.12 also shows some important features of the permanent deformation behaviour of granular materials. It can be seen that the unloading curve of the first cycle is steeper than the loading cycle. This indicates that a large permanent strain has occurred, and thereafter the continued application of load cycles causes a progressive build up of permanent strain shown by the curve moving to the right.
This shows that the permanent deformation induced in the sample increases with increasing number of load cycles, and this has been found to be proportional to the logarithm of the applied number of cycles (Barksdale, 1972). Thorn (1988) showed that deformation sustained was also a function of the shape of the load pulse, the applied maximum strain and the loading period. When the load period was increased by a factor of 10 then the deformation sustained doubled. Thorn also showed that if the sample was loaded closer to failure, greater permanent deformation was generated. Boyce et al (1976) showed that insignificant permanent strains developed if the peak stress ratio \( p/q_{\text{max}} \), remained below 70\% of the load required to cause monotonic failure (where \( p \) = normal stress, and \( q_{\text{max}} \) = deviator stress at failure). Beyond this value the strain rate increased rapidly. The limiting stress at which this rapid change in the rate of accumulation of permanent strain occurred was subsequently defined as the threshold stress, or shakedown limit (Lekarp and Dawson, 1998).

Similar to the resilient response of granular materials, the particle shape, angularity and roughness have been shown to affect the permanent deformation behaviour (Barksdale and Itani, 1989). The material grading also affects this performance, with an optimum grading for resistance to permanent deformation being found (Brown and Chan, 1996, Nataatmadja, 1992). Samples that were better compacted (as defined by having higher density) sustained lower permanent strains (Thorn, 1988), probably due to better particle interlock.

Stewart (1986) showed that the previous load history has an effect on the material behaviour, such that if a cyclic load below that previous sustained was applied then little further deformation was observed.

The behaviour discussed above has all been for materials tested using axial cyclic loading, however Wong and Arthur (1986) showed that shear strains could be generated by rotating the principal stresses. Chan (1990) compared deformation behaviour of
samples of granular materials with and without rotation of principal stresses. His mini­
pavement tests showed that the rate of permanent deformation increased threefold when a
wheel travelled across a sample relative to a similar load being applied statically. This
was further confirmed in a triaxial test with and without rotation of principal stresses on
the same sample (Figure 2.13), where a rapid increase in permanent strain is observed
when shear stresses are reversed.

2.5.5 Resilient Behaviour of Fine Grained Materials

Various authors have shown that the resilient stiffness of cohesive soils decreases non­
linearly with increasing applied stress, when all other factors are kept constant (Seed et al,
following simple expression for resilient behaviour, where \( q_r \) is the repeated deviator
stress, is commonly used and is similar to that defined for granular materials, and is
recommended for use in the American design procedures (Mohammad et al, 1995).

\[
M_r = C_1 (q_r) C_2
\]

The subgrade stiffness has been shown to depend upon the structure of the soil and, in the
case of clay soils, upon the over-consolidation ratio and initial applied stress (Brown et al,
1975). They showed that the stiffness of cohesive soils is inversely proportional to the
ratio of deviator stress to normal effective stress, and proposed a refinement to the above
equation.

\[
M_r = K (p_0' / q_r)^n
\]

Where \( p_0' \) is the initial mean normal effective stress and \( q_r \) is the repeated deviator stress.
Therefore, for a saturated material if there is an increase in the pore water pressure (i.e.
reduction in effective stress) the stiffness should reduce (O’Reilly et al, 1991, Mohammad
et al, 1995).
The compacted state of the materials (i.e. density and water content) has been shown to affect the resilient response, with wetter and less dense samples (i.e. higher degree of saturation) having lower resilient moduli (Thompson and Robnett, 1979). In addition, the method of compaction has also been shown to affect the response. Samples compacted statically show higher values of resilient modulus than materials compacted by a kneading process (Seed et al, 1962; Elliott and Thornton, 1988). Re-compacted samples have also been shown to produce poor performance relative to undisturbed samples of the same materials (Hicher, 1996). Lee et al (1997) showed similar values of stiffness from tests on field and laboratory compacted samples.

For partially saturated soils a linear relationship between suction and resilient modulus has been shown (Fredlund, 1977, Cheung, 1994). Materials exhibiting high suctions (negative pore water pressures) have been shown to have higher resilient modulus, indicating stiffness to be a function of three stress variables, the confining stress, the axial stress and the matric suction of the materials. However, suction itself also affects the previous two factors.

Seed et al (1962) showed that soil thixotropy (stiffening due to time) affected the resilient response of saturated materials tested in the laboratory. They demonstrated that soils stored for a period of time (up to 50 days) before testing were much stiffer than those tested soon after preparation and more so for materials that have experienced shear strains caused by compaction. They attributed this thixotropy to a progressive change in particulate arrangements within the soil structure and pore water pressures. Thompson (1984) suggested that fine grained soils with a low plasticity index, high silt content, low clay content and low specific gravity tended to exhibit low stiffness. Brown et al (1975) and Konrad and Wagg (1993) tested soils at various repeated load frequencies (0.1 to 10Hz and 0.05 to 0.5Hz respectively). They observed no significant changes in material performance with changing load frequency (similar to granular materials).
Various alternative numerical models have thus been proposed to model the resilient response of fine grained soils. These incorporate some of the main variables already described, particularly suction. However all the models proposed tend to be slightly different, depending on the type of material tested and the methodology used. Therefore the models are constructed to fit the data collected and hence they are all different. Significantly this implies that material stiffness is highly material specific (Dawson and Gomes-Correia, 1998).

Similar to granular materials the values of stiffness observed during laboratory testing of fine grained soils varies according to the test methodology and the nature of the materials assessed. Therefore quoting typical stiffness values is difficult, however values of the constants $C_2$ from Equation 4 has been shown to be in the region $-0.2$ to $-0.5$ (Mohammad et al, 1995).

2.5.6 Permanent Deformation Behaviour of Fine Grained Soils

Monismith et al (1975) demonstrated the reduction in generation of permanent strain with re-loading following previous loading (Figure 2.14). This again shows the importance of the stress history of materials in controlling the subsequent behaviour. Permanent deformation has been shown to increase with the logarithm of the number of cycles (Cheung, 1994), with the rate of accumulation of permanent strain increasing as the stress increases. This then eventually leads to a level where the rate of accumulation of deformation increases exponentially (i.e. a threshold stress exists similar to granular materials).

Figure 2.15 (after Monismith et al, 1975) shows the behaviour of a cohesive soil subject to three series of repeated loads. This figure shows that where a sample is loaded with a second phase of loading at a lower level than the first, no further permanent deformation is sustained until the maximum stress of the initial phase is exceeded. This again shows the influence of previous stress history. Cheung (1994) showed that the response of clay
soils to repeated load depended primarily upon their stress history and water content, and thus shear strength. Therefore it has been suggested (Brown, 1996) that the dominant factor in determining permanent deformation is the relationship between shear stress applied \( (q) \) to the shear strength of the soil (i.e. stress ratio).

A limiting value of \( q/q_{\text{max}} \) has been suggested \( (q_{\text{threshold}}) \), above which plastic deformation increases relatively rapidly. According to this relationship, the build-up of permanent strain should be approximately linear with the logarithm of the number of load applications at \( q/q_{\text{max}} \) ratios that lie below the ratio for threshold stress \( (q_{\text{threshold}}) \). However, if \( q \) becomes greater than \( q_{\text{threshold}} \), then the permanent strain increases at a dramatically increased rate, as described. Brown and Dawson (1992) suggest that for design purposes this threshold should be taken at a deviator stress equivalent to 50% of the soils’ measured suction. Cheung (1994) proposed an alternative approach suggesting that the threshold stress occurs at a deviator stress required to generate 1% permanent strain in a sample. This implies that the level of stress at which the onset of permanent deformation becomes unstable is a function of the initial stress state of a soil, which is difficult to predict for a compacted soil (Brown, 1996).

Behzadi and Yandell (1996) showed that permanent deformation was unaffected by confining pressure or the frequency of loading, but increased with an increase in water content and reduced with increasing soil density. This again indicates the influence of the soil’s suction for partially saturated soil.

O’Reilly et al (1991) showed that a soil could withstand a cyclic load greater than a load that would cause static failure. Brown et al (1975) showed that the behaviour of a soil that had sustained cyclic loading was different to one that had sustained static loading to a similar level, although the soil type and stress history (among other factors) were shown to have affected the behaviour. However frequency of cyclic loading has been reported to have no effect (Cheung, 1994). The effects of rotation of principal stresses have been
shown to have a similar effect on fine grained soils as granular materials (Hight et al, 1983).

2.5.7 Behaviour of Materials in a Pavement Foundation

Much research has concentrated on defining the factors that affect the individual material behaviour. Yet the interaction between the capping/sub-base and the subgrade (composite behaviour) is also of crucial importance. The interaction between two layers of different shear strength and elastic stiffness under transient loading produces a major influence on how each layer and type of material can act (Fleming and Rogers, 1996). If an unbound granular material overlying a softer, weaker subgrade is subject to loading, the deflections will be partially controlled by the upper layer as a result of its load spreading ability (stiffness). This controls the level of stress transmitted to the subgrade. However, the subgrade will influence the amount of load spreading that can take place by the way in which it reacts to the stress that is transmitted. This will be experienced both during compaction and under trafficking, and hence will have determined the elastic stiffness achieved in the upper layer during construction (Thom, 1988). Thus the layer interaction affects the stress distribution, which in turn affects the elastic and plastic strains that are developed (Fleming and Rogers, 1996).

In the previous sections it has been shown that the stiffness of materials varies with the degree of confinement and the applied stress. Therefore the properties within each layer of material within a pavement will vary with magnitude, arrangement and position relative to the applied load, the thickness of the layers and the pavement structure as a whole, and the degree of confinement provided by the overburden from the pavement above. All of these factors are affected by the performance of the materials to the applied stresses and their load spreading ability.

During compaction the value of compacted density achieved has been shown to vary with depth in any one layer (Figure 2.16), with reductions in dry density occurring near the
surface where there is a lack of confinement and at depth where the compaction energy has dissipated (Thom, 1988). Figure 2.17 shows that due to load spreading lower densities are achieved by compaction onto softer substrates (Thom, 1988; Parsons, 1992). Consequently, during construction a particular level of stiffness should be required to allow satisfactory compaction of the overlying layers in order to ensure adequate performance (Fleming and Rogers, 1996). Powell et al (1984) suggest that a maximum increase in stiffness of three times that of the subgrade is achievable on top of a pavement foundation.

For any given aggregate grading, the degree of compaction achieved will vary according to the type of plant used and the effort applied. This will also vary according to the moisture content of the compacted materials, and to a lesser extent, the type of aggregate and speed of compaction pass (Parsons, 1992). Semmelink and Visser (1994) showed that the shape and texture of the material did not affect compaction (as evaluated by density), probably due to density being assessed as a relative property. However, they showed that these factors did affect in-situ CBR. They attributed this mainly to the material grading and the influence of the particle interlock (hence shape and texture). Therefore for any given combination of the above there will be a maximum degree of compaction relative to a maximum density that can be achieved on site.

The compacted state of subgrade has a significant effect on the performance of the pavement as a whole, with better compacted density showing better performance in terms of both stiffness and strength (Lotfi et al, 1988). Subgrades with high gravel contents have shown much improved performance due to the influence of the gravel and its interlock within a soil matrix (Houston et al, 1994).

Poor compaction has been shown to increase the rate of accumulation of permanent deformation of sub-base (Marek, 1977), due to further densification under trafficking and poor confinement of materials at depth leading to rutting. The effect of the compacted density on the resilient response of materials appears to vary according to the nature and
grading of the material tested, with coarse materials showing improved resilient modulus with improved compacted state. However, this appears to be materials specific as other studies have shown no improvement (Roston et al., 1976).

Permanent deformation is caused by shear within the material itself, either due to insufficient inherent strength or due to a weak underlying layer (Fleming and Rogers, 1995). The nature of permanent deformation sustained by a pavement foundation under wheel loading (rutting) will vary according to the relative material properties and the loading sustained. Dawson (1997) suggests that there are three basic types of rut formed. These can be summarised as follows.

- Compaction under trafficking, whereby the vehicle causes compaction of the aggregate or possibly particle fracture. This mode tends to be self-stabilising, and can cause the materials to stiffen and hence spread load better. This type of rutting results in a narrow depression forming in the wheel paths (Mode 0).

- Rutting due to weak granular materials, whereby local shear occurs adjacent to the wheels causing dilative heave adjacent to the wheel paths (Figure 2.18). This is largely due to inadequate aggregate shear strength or layer thickness. The consequent thinning of the aggregate layer may result in accelerated damage to the subgrade (Mode 1).

- Whole foundation rutting, whereby the subgrade deforms with the aggregate layers as a composite. The surface pattern is a broad rut with slight heave remote from the wheel (the displacement of the underlying soil causes the rut to form as shown in Figure 2.19). The influence of the subgrade on the formation of this type of rut can be seen by the formation of a ‘bow wave’ before the load (Figure 2.20). This is the most significant mechanism leading to a buried subgrade rut, and is further discussed below (Mode 2).
The onset of one type of rut can lead to the development of a different mechanism which ultimately dominate, with both thinning of the aggregate and subgrade depression being observed (Little, 1993).

If a large transient deformation is sustained under loading due to a poor subgrade, then the material in the upper layers will undergo significant bending. Due to the differences in the modular ratios of the different materials, there will be a difference in the reaction and movement of the materials above and below the boundary between layers. This differential response will in effect cause a change in the confining conditions at the interface in relation to the main body of the upper layer, and relative particle movement can occur. This reaction to loading provides the facility for significant plastic deformation to occur progressively with significant elastic deformation. Thus the differential tendency of permanent deformation to occur between the two materials will influence the tendency to rutting of the foundation as a whole, the shear stresses under the wheel load at the interface being both horizontal and vertical (Fleming and Rogers, 1995). This was supported by the findings of Little (1993) who showed that when pavement foundations were reinforced with a fabric geotextile at subgrade level (limiting vertical strain by controlling horizontal strain), a lower amount of rutting occurred.

At higher stress, beyond the threshold of the granular materials the plastic strain within the pavement materials increases due to shearing. This in turn increases the applied stress to the subgrade and can lead to rutting. If a strong granular material is laid on a weak supporting layer the resultant composite deformation upon loading results in a particle reorientation due to low confinement. This leads to a rut forming in the granular material, hence increasing the applied stress to the subgrade (Fleming and Rogers, 1995). Therefore, the key to avoiding permanent deformation within the pavement materials is adequate shear strength, although if the applied stresses are still too great they can cause weakening and a progressive increase in the permanent deformation.
There is no complete model that can define the formation of permanent deformation of a pavement. However, a division between the stable and unstable build-up of material permanent deformation has been observed. Thus if the applied stresses in a pavement are maintained below the threshold values by providing adequate strength and thickness of the placed materials, the formation of rutting should remain within the stable zone (Fleming and Rogers, 1996).

2.6 Environmental Factors that Affect Material Performance

The performance of the pavement will be affected by changes in material performance caused by changes in the environmental conditions experienced in both the short and long-term. Road foundation design is primarily based on construction loading, but consideration of the long-term behaviour and potential changes during construction is essential. It is not uncommon for pavement foundations to be partially constructed and left for some months before they are completed, and during that time the material characteristics can change. In addition, the material characteristics at construction stage affect the nature of the material’s changes, and hence characteristics in the long-term.

Equilibrium water content is reached after the equilibration of (usually dissipation of negative) pore water pressures in the subgrade. This equilibrium value, once attained, remains relatively stable under impermeable pavements. Equilibrium water content can therefore be used as a long-term design subgrade water content (Croney, 1997, Black and Lister, 1979).

Factors such as a lowering of the height of the water table (due to the early installation and effectiveness of the pavement sub-surface drainage), changes to the stress history of materials (due to the removal of overburden in cutting or addition of overburden due to pavement construction), changes to the material structure (due to the construction operations), material type, temperature, humidity and rainfall may all result in changes to the material’s suction (which controls the equilibrium water content), and hence the
material’s mechanical performance (Black and Lister, 1979). The factors described above primarily affect the subgrade, but capping and sub-base can also be affected, especially if poor weather affects construction.

Much work has been done to establish the relationship between soil suction (water content) and soil index properties (Black, 1962 and Black and Lister, 1979). It has been shown that soil attains a different equilibrium water content depending upon whether it is drying back or wetting up, i.e. this process exhibits hysteresis, (Black and Lister, 1979). This can result in a 1 to 3 % change in the water content of the soil depending on the way in which equilibrium has been attained. This implies that a subgrade that becomes wet during construction will attain a greater equilibrium water content than the same subgrade which is wetting-up from a dry condition having been protected from the weather during construction. If at the earthworks stage, the subgrade is drier than its equilibrium suction value would predict and remains drier throughout the construction it will ultimately wet up and come into equilibrium with the water table (the suctions dissipate). Conversely if rain infiltrates a completed foundation and causes wetting of the subgrade which subsequently dries, the equilibrium water content that it will attain in the long-term will always be wetter (due to the hysteresis effects mentioned previously), leading to a poorer subgrade and hence composite pavement performance.

The effects of changes in water content (suctions) on pavement performance are difficult to predict. Many authors have monitored the change in pavement performance with season, mainly in tropical and cold climates where the environmental variations are greater than in the temperate UK (de Bruijn, 1966, Lary and Mahoney, 1984, and Basma et al, 1991, Andrew et al, 1998). These changes have been shown to have a considerable effect on pavement response.

Seasonal changes to the level of the water table have been shown to affect the performance of typical UK subgrades (Sha’at et al, 1992 and Kamal, 1993). However, this is normally insignificant for the performance of a full pavement construction as long
as the granular layers are adequately drained and worst case subgrade design data are used. However, it could be important at the construction stage where applied stresses are higher, hence lower subgrade stiffness is experienced.

To allow assessment of material response due to the change in water content, the changes in environmental condition must be predicted. There are three main methods of predicting the equilibrium water content (Fleming et al, 1998):

- Prediction from soil suction data, based on the position of the water table and soil plasticity (Black and Lister, 1979).

- Thornthwaite Moisture Index, which uses climatic data to define soil water content based on soil moisture deficit (Russam and Coleman, 1961). It is particularly useful for conditions where the water table is below the presumed zone of influence of soil suctions (approximately 6m for clays and 1m for sands), although it is not particularly applicable for the UK climate.

- Prediction from field investigations of the water contents under exiting pavements, for similar types of subsoil. Due to the site specific nature of the information, this method is most applicable for widening or reconstruction of pavements.

Due to weather effects and problems of predicting subsurface water regimes and material response, it is difficult to anticipate the long-term changes to the performance of unprotected or partially completed pavement foundations. In addition, during construction, measured subgrade performance will be influenced by temporary changes caused by drying and re-wetting, and re-grading and re-compaction. These changes to the materials' structure and water content make it very difficult to suggest a suitable target site stiffness to measure on site in order to confirm the laboratory values of stiffness used for the subgrade in design. Furthermore, capping materials (especially granular capping with high fines content) may also be subject to temporary influences on strength and
stiffness caused by suctions induced by compaction and the infiltration of rain (Thom and Brown, 1988).

2.7 Measurement of Performance Parameters

In the preceding sections it has been shown that there are many factors that affect the behaviour of pavement foundation materials. It has been shown that the material’s response is different according to the physical and environmental characteristics of the material itself and of the adjacent layers, the material’s position in the layer and in the pavement, and the magnitude and nature of the loading applied. Different considerations must be given to subgrades and capping as their nature and the loading they sustain are different. Therefore in order to be able to measure these parameters accurately (for inclusion in analytical pavement assessment) these factors must be simulated in tests used to establish material performance either in the laboratory or in the field.

Current methods developed to measure the required performance parameters of both stiffness and permanent deformation are described in the following sections for both laboratory and field assessment.

2.7.1 Laboratory Testing

2.7.1.1 Granular materials

Various laboratory tests have been developed to assess the behaviour of granular materials. In particular devices have been developed to assess the permanent deformation response, notably the Directional Shear Cell (Wong and Arthur, 1986) and the cyclic simple shear apparatus (Ansell and Brown, 1978). However, the most realistic development in terms of mirroring the stress application is the Hollow Cylinder Apparatus (HCA, see Figure 2.21). The HCA was developed at Nottingham University (O'Reilly, 1985) and was refined by Thom (1988) and then Chan (1990). This is the only suitable laboratory test capable of applying a confining stress and incorporating the
rotation of principal stresses. However, all these devices are limited to testing of granular materials, with severe limitations on the maximum size of particles that can be tested due to the nature of the samples required and the method of sample manufacture. The HCA can test particles of 12.5mm maximum size, while the other devices can test up to a maximum size of 40mm.

The inability to simulate the effect of rotation of principal stress is a potentially severe limitation for the study of permanent deformation behaviour since the permanent strains measured with rotation of principal stresses can be at least three times as great as those under monotonic loading (Chan, 1990).

The problem of particle size and distribution can partly be overcome in the large repeated load triaxial test. It matches the loading and confining stresses that are experienced in the field but it cannot apply rotation of principal stresses. The repeated load triaxial test has formed the basis of much research into the performance of both soils and granular materials, but again the size of the samples to be tested is controlled by the maximum particle size. To remove particle size effects, a minimum diameter of seven times maximum particle size should be used (Cheung, 1994), although Sweere (1990) suggests 10 times. This means that a triaxial sample of 525mm diameter is required for a typical fine graded capping (e.g. 6F1 capping which has a 75 mm maximum particle size).

Although the stiffness of materials has been shown to be unaffected by the rotation of principal stresses, enabling the use of the repeated load triaxial test for assessing the resilient response of granular materials, samples of large diameter are still required. Although large-scale repeated load triaxial tests are undertaken, their use tends to be restricted to finer granular materials, with much work concentrating on sub-base or model gradings, utilising small particles mimicking the gradings of coarser materials. However their use for collecting data for direct inclusion in design is questionable (Hughes, 1997) and was shown to be unrepresentative by Sweere (1990). A test to derive the performance of capping with large particles remains an area requiring research. Research is currently
being undertaken in South Africa to develop the K-Mould (Semmelink et al, 1997), which can test coarse materials with particle size up to 125mm. The device features a segmented large diameter steel cylinder with movable top and bottom load plates. Each segment can be fitted with different spring to provide variable confinement to the materials compacted within the mould, and each segment and plate can be instrumented. The mould is then loaded via the top plate. It is believed there is only one such device in the world.

To provide adequate resistance to permanent deformation the need for adequate shear strength has already been identified. Measuring the shear strength of granular materials and relating this to their observed susceptibility to permanent deformation under trafficking was undertaken by Earland and Pike (1985). They used large shear box tests to assess the peak shear stress ratio (PSSR) of sub-base, related to the observed performance under trafficking on site to determine a suitable pass/fail criterion. It was found that the PSSR was only effective as a guide for either very good or very poor materials, and that a significant proportion of materials still required a site trial to determine suitability and assess the composite performance with subgrade. Again particle size effects make this test unsuitable for coarse material since a 300 by 300mm shear box is only suitable for assessing particle sizes up to 40mm.

2.7.1.2 Fine Grained Soils

The problems of material particle size do not affect the testing of fine grained soils. Repeated load triaxial test equipment has been developed and is used in many countries to assess the performance of such soils (Seed et al, 1962, Brown et al, 1975). Devices that incorporate the rotation of principal stress for clays have been developed, but these can only test reconstituted samples (Hight et al, 1983, Pezo et al, 1991, and Abrantes and Penumadu, 1998). Similar to granular materials the problems of sample manufacture make the modelling of permanent deformation behaviour of undisturbed fine grained soils under realistic loading conditions an area where future research is required.
The repeated load triaxial test has been used to provide stiffness data for inclusion in an empirically based pavement design nonogram. In the United States the empirical approach uses a relationship between resilient triaxial modulus and soil support and allows for other factors such as seasonal variation, which is related to a structural number and hence a variety of acceptable layer thickness designs (Elliott and Thornton, 1988). These tests were developed in the 1970s (Barksdale et al, 1975), and different methodologies are employed for cohesive and granular materials. The tests employ conditioning of the sample prior to testing at different deviator stresses. The stiffness is then calculated for a range of deviator stresses. The modulus tends to be quoted at a high deviator stress, at which the stiffness is tending towards a stiffness asymptote. This approach does not take into account the highly stress-dependent nature of fine grained soils at lower deviator stress. A similar test is used in Australia (Austroads Guide, 1992) where the conservative data are used directly in analytical pavement design.

The original test apparatus used a single strain measuring device to define the deformation behaviour of the whole sample. The measurement of axial strain therefore incorporated end effects from restraint from the load platens and the effects of platen/sample contact (Pezo et al, 1998), which can lead to an underestimate of resilient stiffness by 50% (Mohannmad et al, 1994). The use of on-sample instrumentation and sample end plate grouting has been used to remove these end effects (Kim and Drabkin, 1994), allowing the measurement of vertical and radial strains across the central third of the sample. However, the weight and anchorage systems of such strain-measuring devices have been shown to lead to variations (with a greater effect on clay soils than on sands), possibly due to anchorage of the transducers (Mohammad et al, 1994).

A repeated load triaxial test apparatus, utilising lightweight on-sample strain measurement, was developed by Cheung (1994) to measure fine grained soil performance parameters for direct use in analytical design. This apparatus is described further in Section 3.4. Brown (1996) stated that this apparatus requires further development, but has the potential for use in engineering design practice to measure the resilient modulus,
permanent deformation and shear strength of fine grained subgrade materials. The work reported to date with this apparatus has concentrated on using laboratory-prepared samples of low strength. Two separate samples have been used to derive the resilient and permanent deformation behaviour to failure for a soil, utilising an element of conditioning of the samples prior to obtaining resilient properties (Cheung, 1994). The results were validated by comparing the data collected to patterns of behaviour found in other testing performed with similar apparatus and other models derived for the performance of similar materials, rather than absolute values of stiffness collected.

The most realistic means of assessing the resistance to permanent deformation of subgrade is via wheel loading since this simulates most accurately the applied loading regime in the field, including the rotation of principal stresses. It will also, importantly, measure interlayer effects (Chan, 1990). A number of wheel trafficking tests of various sizes have been developed to assess permanent deformation, such as the pavement test facility at Nottingham University (Brown and Brodrick, 1981) and the Heavy Vehicle Simulator at the Transport Research Laboratory (TRL).

2.7.2 Modelling Field Conditions and Sample States in the Laboratory

In a road foundation the subgrade can experience many conditions depending on its location (cutting or embankment) or water content. For accurate laboratory testing the subgrade’s condition must be modelled. There are four material states that should be considered (Fleming et al, 1998).

- Undisturbed soil, as found in the base of cuttings at the time of construction.

- Remoulded, re-compacted soil at the in-situ water content, as found in embankments at the time of construction or after reworking.
• Samples in the two conditions above but at their long-term equilibrium water content after dissipation of suctions or wet weather.

The undisturbed samples can be prepared directly from the subgrade. Remoulded samples can be prepared by re-compacting soil from a sample of the subgrade using appropriate compaction. Thus the modelling of these two states is quite straightforward. However, to obtain samples that accurately represent the equilibrium condition, the prepared sample (either undisturbed or remoulded) should be allowed to change water content at the equivalent conditions of confining stress and suction that would be experienced in the field.

Various authors have proposed methods of doing this by forcing water into pre-compacted (i.e. remoulded) samples (Chu et al, 1977; Drumm et al, 1997). They state that the soil requires a considerable time to equilibrate, even for relatively high permeability soils. It was found that when water was forced radially into a clay under a back pressure in an attempt to reduce the preparation time, considerable softening of the outside of a specimen of remoulded soil occurred with very little water penetration, i.e. an even water distribution could not be achieved, (Drumm et al, 1997). The difficulty of bringing undisturbed samples to equilibrium would be greater.

2.8 Field Measuring Devices

2.8.1 Introduction

This section describes test equipment which measures (or infers) one of the performance parameters. The nature of field tests are such that they measure the composite performance of materials compacted into the pavement structure, although where only subgrade is tested it can be regarded as an element test. Much research has been undertaken to develop tests that measure stiffness or modulus in the field. Little attention has been paid to permanent deformation characteristics, i.e. the measurement of the
resistance to rutting of the compacted materials. This is partly due to the inherent complexity of the distribution of stresses and strains beneath a rolling wheel and the composite behaviour of a layered system. For the resilient behaviour (stiffness) it is the response to a single pass of a wheel that is important. For permanent deformation behaviour it is the response to a number of wheel passes that is important. The use of a rolling wheel to measure such parameters is impractical as the measurement of the resilient strains induced by the passage of the wheel is practically impossible to quantify, therefore alternative tests that model these parameters are required. The means of assessment are split into two categories, those devices that measure stiffness and methods developed to assess field rutting performance.

2.8.2 Equipment for Measuring Elastic Stiffness

There are five basic types of in-situ stiffness measuring device: static plate loading tests, vehicle loading tests, vibration methods, impulse loading methods and impact methods. The nature of these methods and devices based upon them are discussed below. This section summarises the detailed reviews of stiffness measuring devices produced by Fleming and Rogers (1995) and Fleming (2000).

2.8.2.1 Static Plate Loading

Plate loading tests (Day, 1976) involve the application and removal of a load to a circular plate by jacking from the underside of a sufficiently heavy vehicle. It is normal to load the plate in four increments, the resilient deflection being measured during the unloading phase of the test. The operation is often repeated to remove bedding error(s). The test is relatively slow, 2-3 tests per hour being possible once established on site.

From the deflection data an equivalent Surface Modulus or a Modulus of Subgrade Reaction (K), standardised for a 760mm diameter plate, can be calculated. The modulus on loading (Ev) is often compared for load cycles 1 and 2 to determine a ratio (Ev₂/Ev₁).
with a ratio of 2 or more generally deemed to signify inadequate compaction. The modulus on unloading \( (E_0) \) is often correlated with other test measurements. Other methods of measuring stiffness at various stresses using the plate bearing test to define the stress-dependency of pavement foundations have been proposed (Dastich and Dawson, 1995).

### 2.8.2.2 Vehicular Loading Devices

These devices measure the deflection and rebound of the road pavement during the passage of a laden vehicle (McFarlane et al, 1975). They were devised to measure deflection of a full pavement in service. There are two main vehicle load devices the Benkelman Beam and the La Croix Deflectograph, which is an automated Benkelman Beam. A beam between the dual rear wheels of a loaded vehicle supports a probe that bears onto the ground and is positioned initially between the wheels. The truck is driven forward slowly and the rebound of the surface is recorded. Problems have been observed in use on unbound pavement layers as a result of heave due to surface instability, which may cause a ridge to form between the wheels (Cobbe, 1986).

### 2.8.2.3 Vibratory Loading

Two similar types of vibratory loading test developed in the USA are the Dynaflect (Scrivener et al, 1964), which uses counter-rotating masses, and the Road-Rater (d'Amoto and Wiczac, 1980), which incorporates electro-hydraulic systems to induce a steady-state harmonic load in the pavement. The peak-to-peak deflection is measured using geophones at different radial distances from the applied force to define a deflection bowl. By back-analysis of the deflection bowl it is possible to estimate the layer moduli. These devices have only been used on full pavement constructions. Neither device is currently used in the UK.
2.8.2.4 Natural Vibrations Method

The Natural Vibrations Method, or NVM (Ehrler, 1989), has been developed into the Humboldt Stiffness Gauge (HSG). The devices consist of a bearing plate that rests on the ground and supports the body of the machine. The machine vibrates (excited by an internal mechanism) to produce small changes in the force applied, which in turn produce small deflections. Geophones are used to measure both the changes in force and deflection for 25 frequencies between 100 and 200 Hz. The HSG is proposed by the manufacturers as a method of measuring density rather than stiffness (Skiekmere, 1999).

It is claimed from the manufacturer’s literature that the HSG has "a high accuracy and a small variation in the measured stiffness, thus permitting computer regression of the average values at different pressure levels to produce parameters for a non-linear function of stress-dependent elastic modulus". The original NVM has been shown to correlate well with the plate loading test (Dastich and Dawson, 1995). The HSG is currently only available in the United States.

2.8.2.5 Impulse Loading

These devices generate a load pulse by dropping a mass on to a plate bearing on the ground via a spring. The devices are instrumented to measure combinations of the applied load acceleration and/or velocity. Each device loads over slightly different stress ranges and load pulse duration. The deflection is measured upon loading and is interpreted from either single or double integration of the velocity or acceleration transducer data. The total deformation (as both resilient and permanent deformation are measured) is used to calculate stiffness, thus for weak materials at high stress low stiffness may be observed due to large permanent deformation rather than the effect of the resilient response alone. In addition static theory is used to calculate the stiffness and this can also lead to errors (Section 2.11).
The Falling Weight Deflectometer (FWD) is a trailer-mounted device which consists of a weight that is raised mechanically and dropped onto a set of rubber cushions bearing onto a steel plate. The force is measured directly and surface movement is monitored by up to seven radially-spaced velocity transducers, which define the surface 'deflection bowl'. Back calculation, utilising an iterative process to match a predicted bowl to the measured bowl, allows for estimation of individual layer moduli. Drop height, plate size and mass are variable, thus the stress application and pulse duration can be controlled.

Testing has primarily been carried out on complete pavement structures, with extensive use for the assessment of in-service pavements for maintenance work. Recently it has been used in the UK for un-surfaced pavement assessment, particularly by the TRL (Chaddock and Brown, 1995) and Loughborough University (Fleming and Rogers, 1995).

The FWD, although large and very sophisticated, has a growing world-wide acceptance for assessment of pavement construction. It requires a vehicle to tow it and this can cause problems on very soft subgrades. The device is shown schematically in Figure 2.22.

It has a realistically high mass and loading time (due to its sprung plate system), but it stresses to a considerable depth and concern has been expressed regarding theoretical curve fitting to the full measured deflection bowl on granular materials. However, measuring only from the central (plate) geophone removes this problem and leads to a simplified analysis using the Boussinesq equation for plate loading on a linear elastic half space, assuming a rigid plate.

\[ E = \frac{\sigma \pi r (1-\nu^2)}{2d} \]

(where, \( E \) = stiffness, \( r \) = plate radius, \( \nu \) = Poisson’s ratio and \( d \) = deflection)
The Portable Falling Weight Deflectometer, or Loadman, is a Finnish device (Whaley, 1994). It comprises an enclosed tube (1.2 m long) with a 10 kg falling mass contained within it, the total mass being 18 kg. The tube is inverted to 'raise' the mass, which is held magnetically, and when upright and appropriately positioned it is released to fall 800 mm. The impact is transmitted to the ground via a rubber spring and steel bearing plate (which can be varied between 110 mm, 150 mm and 200 mm diameters), which is rigidly attached to the bottom of the tube. An accelerometer is mounted at the top of the tube in the readout unit, the signal from which is used to interpret deflection, modulus, length of loading impulse (typically 10-30 ms), percentage of rebound deflection compared to the maximum deflection, and the ratio of the second measurement compared to first. The interpretation of modulus is based on the assumption of constant applied stress (approximately 1200 kPa for 110 mm plate diameter), which is not directly measured. It is shown schematically in Figure 2.23.

The German Dynamic Plate Bearing Test

The German Dynamic Plate Bearing Test (GDP), or Light Weight Drop Tester, is described in a German technical specification (TPBF-StB, 1991) and comprises a falling mass (10 kg) impacting a 300 mm diameter contact plate (total mass 15 kg) via a rubber buffer giving a load pulse of 18 ms ± 2 ms. The device incorporates a transducer which captures the impact signal and interprets a deflection upon loading. The drop height of the falling mass is set such that the peak applied force is 7.07 kN (yielding a stress of 100 kPa), on a standard foundation, and consequently no recording of applied force is made.

A modulus is calculated, which is displayed on a hand held read/control unit. It is claimed that it can measure in the range 10-125 MN/m². It is thought the transducer is an accelerometer (according to the UK distributor). The device is intended for stiff cohesive
soils, mixed soils and for coarse grained soils up to 63 mm in size. In Germany it is used to indicate to standard foundation performance, and is used as a comparative test relative to a static plate loading test. It is shown schematically in Figure 2.24. Some published information of its use in the UK exists (Shahid et al, 1997, Biczysko, 1995).

**TRL Foundation Tester**

The TRL Foundation Tester (TFT) was developed at Loughborough University under contract to the TRL. It is similar in principle to the Loadman and the GDP. The TFT comprises a falling mass inside a guide tube (10 kg) with variable drop height of up to one metre (Rogers et al, 1995). The mass impacts a plate on the ground via a rubber buffer. The plate is 300 mm in diameter, and the stresses applied are in the range 20 to 200kPa with a load-time pulse of 15-25 ms.

It measures both the force applied to the plate via a load cell and the velocity of the ground, via a velocity transducer which bears onto a foot which bears onto the ground through a hole in the centre of the loading plate. It was found that for very stiff foundations, locating the foot on the ground produced results that correlated more closely to those of the FWD (Rogers et al, 1995). It is shown schematically in Figure 2.25. Stiffness is calculated using similar Boussinesq half space analysis to the FWD.

**La Dynaplaque**

La Dynaplaque (LCPC, 1997) is a vehicle-mounted device developed in France. It was designed to simulate the wheel load of a 13 tonne axle moving at 60 km/h. The impulse load is applied by a falling weight loading a 600mm diameter bearing plate via a system of coiled steel springs. The dynamic surface modulus is obtained by correlation with the coefficient of restitution of the mass, which is measured as the maximum rebound height.
2.8.2.6 Impact Loading

Clegg Hammer

The Clegg Hammer (Clegg, 1976), developed in Australia, consists of a guide tube, a 4.5 kg 50mm diameter cylindrical weight with a handle and a digital readout unit. The maximum deceleration of the hammer, falling through 450mm, on impact with the surface is measured by an accelerometer fitted to the hammer head (Figure 2.26). The Clegg Hammer was designed for the assessment of granular base-course materials to aid in compaction control and the location of weak spots.

2.8.2.7 Requirements of a Test for Assessing Pavement Foundations

Fleming and Rogers (1995) considered the requirement of a suitable test to measure the in-situ stiffness of pavement foundations. They stated that tests should apply a similar frequency and magnitude of load, over a similar area of loading to that applied by a vehicle and be unaffected by particle size, and thus create a similar stress/strain regime to that operating in a pavement foundation. For efficient use on site the test should be simple, quick and flexible to use. This ideally means a device that is portable and can take a large number of readings reasonably quickly.

2.8.2.8 Summary

Many of these existing test devices have been developed specifically for testing full pavement constructions, whilst others have been modified from airfield or earthworks applications. Most of these tests, i.e. Deflectograph, Dynaflect, Road Rater, have limitations when applied to unbound (and weaker) materials. The Dynaplaque and HSG are unavailable in the UK.
Of the devices specifically developed or under development for the testing of unbound materials for elastic stiffness, only the static plate bearing test cannot be classed as rapid, since it requires large kentledge, is time consuming to perform, and does not load at an appropriate frequency to traffic loading. The Clegg Hammer is widely used for indicating uniformity of compaction or for indirectly indicating CBR. However, because of its small diameter and high mass, particle size effects become significant for coarse materials and it is known to cause fracture of particles under test. The other devices (FWD, TFT, GDP and Loadman) are available and are worthy of further investigation as they meet most of the requirements for a test defined above.

2.8.3 Equipment for Measuring Resistance to Permanent Deformation

The direct development of a test that can measure a parameter related to permanent deformation in the field also remains an area requiring research. This is due to the complex nature of the mechanism of permanent deformation and the number of external factors that affect material behaviour. Currently the mechanism of development of permanent deformation is not well enough understood to model with any real confidence, and as a result the prediction of the amount of rutting likely to occur in a pavement foundation remain a significant challenge. Recent studies have attempted to use a strength measurement as an indicator of resistance to rutting. This section reviews devices capable of measuring 'strength' in-situ. Any test adopted must minimise disturbance to the constructed foundation.

2.8.3.1 Dynamic Cone Penetrometer (DCP)

The DCP (Kleyn et al, 1982) comprises a series of slender rods at the base of which is attached a 20 mm diameter 60° cone. A sliding mass arrangement, in the top rod, falls 575 mm to strike an anvil which then drives the cone tip into the material under test. It typically measures to a limit of 800mm in depth (Figure 2.27).
The relative strength of each layer is quantified by penetration rate (mm/blow) of the cone (measured manually), which is often correlated to CBR. It is a low cost device and is simple to use. Problems have been highlighted regarding its robustness when used in very strong soils or materials with large particles. The DCP has also been used to correlate to stiffness via strength/CBR (Du Pleiss and Beer, 1996 and Siekmeier et al, 1999). However given the problems of such CBR to stiffness correlations (discussed in Section 2.4), correlating stiffness with the DCP data is unlikely to be accurate.

2.8.3.2 Other Strength Related Tests

Several in-situ tests that are applicable for determining strength are commonly used in geotechnical ground investigations. These include:

• Hand vane, which is used for assessing the undrained shear strength of cohesive soils (close to the surface).

• Plate load bearing test, in which general failure of the soil beneath a loaded plate is assumed and the undrained shear strength for relatively stiff soils can be determined.

• Penetration tests. A range of penetration tests exist which can be correlated to in-situ undrained shear strength and range from small-scale equipment (the Mexi-cone) to relatively large-scale automated devices (such as the Penine Probe)

2.8.3.3 Summary

Because of the complex nature of formation of rutting the only acceptable method to generate it is by appropriate wheel loading. However, measuring strength and relating this to rutting appears to be the most appropriate method of assessing permanent deformation (Fleming and Rogers, 1996). To measure strength the soil must be failed and therefore all
the tests are intrusive and the magnitude of disturbance generated by the tests must be considered before a final test is selected.

2.8.4 Other Performance Related Tests

Although density is not a specific requirement as a performance parameter, its measurement is critical in assessing the adequate compaction of materials. A material once laid within a pavement foundation may possess sufficient stiffness to allow the compaction of the subsequent layers, yet may not sustain trafficking because of poor strength due to inadequate compaction. Therefore to assess the compacted state of materials various authors have suggested that density should be measured on site and compared to the maximum density that the same material can be compacted to either in the laboratory or in a trial (Marek, 1977 and Siekmeier et al, 1999).

There are two basic methods of measuring density, replacement methods and nuclear transmission methods (Figure 2.28). Both are covered in British Standards (BS1377, 1990 Part 9). The replacement tests involve the excavation of a small element of the compacted material and their replacement with a uniform sand or water (filling a pre-positioned liner). The volume of the hole can thus be measured together with the mass of the compacted material removed, and hence the density determined. The tests are time consuming and operator sensitive (Roston et al, 1976).

Nuclear density testing involves a radiating source being applied to the compacted material. The amount of radiation detected decreases in proportion to the density of the material between the source and receiver (Roston et al, 1976). Another source measures the water content by assessing the amount of radiation intercepted by hydrogen ions, hence facilitating the calculation of bulk and dry density. Two modes of testing with the device are available. The back-scatter method assesses the top 100 to 150mm of material and is heavily influenced by the material near the surface. The transmission mode, where the source penetrates the materials under test, provides a more representative density. The
test is relatively quick and readily repeatable, although laboratory calibration is required for each material tested. Care must also be taken when assessing the results from carboniferous materials due to their ability to absorb water.

2.9 Field Trials

Field trials have been undertaken by various researchers to assess both the performance of completed pavement foundations under trafficking and the devices used to measure that performance. From a number of these trials target values have been suggested for the top of foundation stiffness from observations of materials that have shown acceptable performance. Relevant trials and the foundation target values proposed are reviewed below.

2.9.1 Comparison Between Devices to Measure Stiffness

Several comparative trials on granular materials comparing in-situ dynamic stiffness measuring devices have been undertaken both in the UK and overseas. However, any comparisons between devices must consider effects such as the applied stress, the load pulse duration and plate size. Therefore the correlations have to be applied strictly in the context of which the results were obtained (i.e. stress history and materials type).

In a recent study by Chaddock and Brown (1995), the FWD (using measurements from the central geophone only), the TRL Foundation Tester (TFT) and Loadman were compared on a variety of standard foundations. The stress-dependent nature of the materials under test is clearly evident, with the subgrade stiffness inversely proportional, and the granular layer stiffness proportional, to applied stress (Figure 2.29). The TFT measured a similar stiffness to the FWD (when comparing the results for the same plate size of 300 mm), whilst the Loadman (smaller plate size) gave similar results on a weaker foundation (at a much higher applied stress) but underestimated the stiffness on stiffer foundations. Some of this discrepancy may be attributable to the rate of loading applied.
by the devices. A faster loading rate has been shown to give different results for the same dropped mass and plate size (Fleming, 1999). The Loadman loads at a considerably higher frequency than the other devices (Rogers et al, 1995), therefore the discrepancies observed with it were attributed to the effects of the higher frequency load and the effects of a higher (assumed) contact stress.

Shahid et al (1997) carried out comparative trials on four UK road construction sites testing subgrade and capping materials. The testing was undertaken with the FWD (measuring the central geophone only), the TFT and the GDP, but applying lower contact stresses than Chaddock (100kPa) to facilitate a direct comparison with the GDP. This work showed the importance of good substrate support on the pavement stiffness as a whole. However, considerable scatter was found in the measurements made with each device. The GDP was shown to measure approximately 50% of the stiffness value of the FWD and TFT, which showed a reasonable correlation between readings (Figure 2.30). The GDP showed less sensitivity to changes in stiffness, and its accuracy was questioned as it does not measure the applied load.

Siekmeier et al (1999) undertook comparative trials at five sites with the Loadman, the Humbolt stiffness gauge (HSG) and the DCP (correlating from DCP penetration index via CBR to stiffness). The data were compared to previously obtained FWD back-calculated moduli for the materials. The correlation between DCP measurements and other values of stiffness was shown to be poor. The Loadman and HSG showed similar values across all materials. The Loadman showed similar values between the large and small plates used for soils (presumably due to the high applied stress and lack of soil stress-dependency at high stress), but showed significant variability between plates due to stress-dependency on granular materials.

Nunn et al (1997) tested trial road foundations of various constructions. Testing was undertaken with the FWD, GDP and static plate test. The GDP again produced
considerably lower values that the FWD and static plate test, whereas the FWD showed a reasonable correlation with the static plate tests.

2.9.2 Field Trials and Target Values

Much work has been undertaken to assess the field performance of granular capping and sub-base materials, often comparing marginal or new materials to conventional crushed rock. Much of this recent work (Chaddock, 1994) has utilised the DCP for strength assessment and attempted to correlate this with (controlled) trafficking data. Attempts have also been made to correlate 'foundation stiffness' (i.e. measurements on top of sub-base), measured by the FWD, with the permanent deformations caused by trafficking. There was large scatter, and material-specific relationships were typically found where different materials used as sub-base with similar stiffnesses deformed very differently. The construction traffic that can be carried by a foundation was generally greater where the stiffness of the foundations was higher and the penetration rate of the DCP was lower (i.e. higher inferred strength). This work suggests that a DCP test cannot predict the absolute deformation by a given amount of trafficking of any foundation. However it is able to discriminate between foundations that vary in their resistance to rutting (Chaddock, 1994). The DCP has proven to be very useful in indicating layer thickness and where a contrast in strength exists. All authors show that where adequate pavement thickness is provided then the rate of permanent deformation increases with the logarithm of number of vehicle passes.

Work carried out at Loughborough University (Fleming and Rogers, 1994) determined the traffickability of four different 300 mm thick unbound granular layers compacted to different states (classed as poor, medium or good). These materials were compacted over both a soft subgrade and a synthetic substrate, comprising a thick 20mm rubber layer over a concrete slab. Several dynamic stiffness measuring devices were tested, including the FWD, and the foundations were also assessed with the DCP. In general, notwithstanding the rounded sand and gravel used, all the materials performed exceptionally well over the
fully resilient foundation (having received 'good' compaction) resulting in only a few millimetres of surface deformation after 1000 passes of a vehicle. This contrasted starkly with the poorer performance on the natural subgrade. The DCP ranked 15 out of 16 bays in the correct order when compared to traffickability. In general, the unconfined upper zone of the granular materials provided less resistance than the central zone, and the lower zone again exhibited lower resistance due to the influence of the soft subgrade. Excavations were made through sections of the trial to measure subgrade deformations, which typically contributed more than 50% to the measured surface rut. It was noted that the single front wheel did more damage than the dual back wheels of a 2 axle vehicle on materials that deformed most easily.

Little (1993) observed that approximately 80% of the surface rut was transferred to the subgrade in four un-reinforced pavement foundations consisting of two bays of standard sub-base (400mm and 550mm thick) and two bays of a sand and gravel of similar thickness. The transfer of rut to the subgrade was believed to be influenced by the low stiffness of the site used (although it was observed to have high subgrade strength, at least at the surface), which again suggests the influence of subgrade stiffness on rut formation. This was considerably more transfer of rut to subgrade than that observed by Fleming and Rogers (1994) and more than the 50% transfer suggested by Powell et al (1984) when they set their top of sub-base maximum rut to 40mm.

The field trial reported above (Chaddock, 1994) compared the results of trafficking with stiffness values measured on the top of the foundation with the FWD using a stress of 200kPa (corrected to a plate diameter of 450mm). Foundations having a measured stiffness of 80 -110MPa performed adequately, whereas those having a stiffness of less than 80MPa developed excessive rutting. He suggested a limiting criterion of 80MPa. He argues that a higher value of stiffness would tend to be measured with a smaller diameter plate due to stress-dependency of the materials (as observed by Siekmeier et al, 1999). Smaller plates stress to a shallower depth in the pavement and hence the stresses are less
well distributed, therefore higher stresses give higher values of stiffness for the overlying granular materials due to their depth of influence (Fleming, 2000).

Fleming and Rogers (1995) suggested that values of stiffness of no less than 30MPa at the top of subgrade and 100MPa at the top of sub-base could be adopted (based on linear elastic analysis of the results of the Loughborough trafficking trial). Since the stiffness should increase upwards through the pavement structure, the value at the top of the capping layer (where present), should be no less than 30MPa (the subgrade requirement) or perhaps 50-60MPa (an improvement on the minimum subgrade requirement). A minimum CBR of 15% is suggested by Powell et al (1984) for a suitable target for top of capping. This can be equated to a calculated stiffness of 99MPa (Fleming and Rogers, 1995, using Equation 1, Section 2.4). However, it should be noted that this equation is for cohesive soils and has a quoted accuracy range of 2-12% CBR (Section 2.4).

These levels would not appear to be inconsistent with those specified in other countries. In Germany, for example, a minimum $E_{v2}$ from a static plate test at the top of subgrade or formation of 45MPa is required, as determined by the second loading cycle of the static plate load bearing test with a stress of 100kPa applied to a plate of 300mm diameter. The equivalent value for the top of the frost protection layer is 100-120MPa, depending on the road classification, and for the top of the sub-base layer is 150-180MPa. Similarly, in Finland the value of $M_r$ at the top of sub-base is 150-180MPa. In Austria the required value of stiffness at the top of the formation, or subgrade, is 35MPa. In France a minimum stiffness at the top of capping is specified as 50MPa, as measured by the Dynaplaque using a 600 mm diameter plate.

Thus the French value of 50MPa on a 600mm plate would be expected to rise if corrected to a smaller diameter plate, and the German value of 120MPa on a 300mm plate would reduce. The suggested value of 80MPa for a 450mm plate would thus appear to be reasonable. However in Germany a thick granular frost protection layer is provided and it
is likely that the stiffness value measured is limited to that of the granular material due to the depth that the plates stress the foundation.

The trials by Nunn et al (1997) involved trafficking of various thicknesses of granular and cement bound materials. They noted that stiffness could not replace thickness design, since a weak subgrade overlain by a very stiff but thin layer may provide adequate stiffness but will not sustain trafficking, due to the high shear stresses applied by construction traffic loading resulting in a punching shear bearing capacity type failure. They suggested that not only should a minimum level of stiffness be set but a minimum value of compacted density of granular materials should also be applied, quoting different values of target for formation (top of capping) and top of foundation (top of sub-base). These values are summarised in Table 2.1. It should be noted, however, that these target values are not lower bound values but those at which acceptable performance was observed, and that the value of a target will vary according to the method by which it is measured.

2.10 Comparisons of Laboratory to Field Derived Data

The majority of the work undertaken to assess the comparative performance of materials in laboratory and field tests has been undertaken in the United States and has concentrated on resilient behaviour of subgrade materials (probably due to the problems of laboratory assessment of granular materials with large particle size). This work has typically utilised back-calculated FWD layer stiffnesses relative to the laboratory repeated load triaxial tests.

These tests have been performed for three main reasons, to validate analytical pavement design models, to assess and model long-term subgrade performance when subject to large environmental changes, and to validate design models using simplified tests or testing regimes that do not simulate the true loading environment.
Significant variability in the correlations between laboratory and field data have been observed, partly due to problems with back-analysis and accuracy of the models used (Stolle and Jung, 1992). The method of laboratory sample preparation and the magnitude of applied stress used to calculate the stiffness have also been shown to lead to poor correlations (Daleiden et al, 1994) due to the differences in stress distributions applied between field testing and laboratory repeated load triaxial testing. Houston et al (1992) state that good agreement between laboratory and field data should not be expected, since the effects of sampling disturbance will not be assessed and the volumes under test vary, hence the variations in material properties with depth are not assessed in the laboratory. In addition, the method by which the stiffness is calculated will also play a role (Section 2.11). Van Deusen et al (1994) compared FWD data on a cohesive subgrade to comparable repeated load triaxial testing. They showed a reasonable correlation between laboratory and field data, but observed a considerable amount of variability between the field derived stiffnesses, which was in part attributed to the surface condition at the test location. Ping et al (1996), compared static plate bearing test data to repeated load triaxial data on reconstituted samples of subgrade compacted at similar water content and density. They found that a reasonable correlation existed between the data but the tests were performed at high applied stresses where the soil should have reached its stiffness asymptote. Therefore the loading used is not directly comparable to that experienced under trafficking of a part-constructed pavement.

Little information comparing laboratory data to permanent deformation data derived on site has been undertaken, due to the lack of a model to predict rutting. As previously described, the threshold stress has been suggested as a limiting level for stress application, and some work has been undertaken to link threshold stress to shear strength and suction in the laboratory (Cheung, 1994 and Brown and Dawson, 1992; see Section 2.5.6). To achieve this a predictive method of assessing the threshold stress is required. For granular materials attempts have been made to identify this stress mathematically using modified 'Sherby-Dorn' plots (Lekarp and Dawson, 1998). These plot the change in strain per cycle with the cumulative strain. They found that for granular materials there
was a distinct change in material response (Figure 2.31). For fine grained materials the threshold remains as yet mathematically unproven, although evaluation using the stress value at 1% permanent strain in a repeated load triaxial test has been used (Cheung, 1994).

2.11 Methods of Numerical Analysis of Pavements

In the previous sections, the factors that affect a material’s response have been described and consideration has been given to the differences between laboratory and field assessment. However the method by which the collected data is analysed and used to facilitate an analytical approach to pavement design is critical. In this section a brief outline of the methods of data and pavement analysis are given, describing the assumptions made and the appropriateness of the various approaches.

To enable analysis of the pavement structure a number of simplifications need to be made to both the structure and material performance. These include a simplification of the loading and the structural form of the pavement. For a complete flexible pavement this normally involves simplifying the pavement into three layers, the structural layers, the granular supporting layers and the subgrade (Croney, 1997). However even with these simplifications the analysis is still difficult due to the interaction of the layers and the complex ways in which their material properties change with applied stress (Collop, 1998). Therefore an exact solution is extremely complex to achieve as each layer will behave differently under different loading, thickness and environmental conditions. For routine analysis methods have been devised which also simplify the analysis procedure. The simplest and most widely used is to simplify the material stress strain relationships to linear ones, assuming the properties of the material are constant at the applied stress through out the layer, whereby analysis at the critical design locations (i.e. layer interfaces) can be undertaken. The materials’ elastic properties are required together with applied contact stresses and layer thicknesses, and the stresses and strains can be calculated. Several programs have been written to analyse pavement structures in this way.
most commonly used are Bisar and Elsysm 5, as reviewed by Brown, 1996), and this method of analysis is appropriate for full pavements where the structural layers govern the behaviour of the structure (Brown, 1996). However for analysis of granular materials and their behaviour during construction, solution using a linear elastic analysis approach does not account for the stress dependency of the materials (Brown, 1996).

To assess fully the behaviour of granular pavements and to incorporate the materials non-linear stress dependency, a more refined approach is required. Linear elastic analysis can be used in an iterative approach. In this approach the layers of each material are subdivided, to take account of the variations in applied stress through the layers (in accordance with a stress-dependency model for the materials). The stiffness values of each layer can then be adjusted at each iteration in accordance with the applied stresses and material model until the strains at each interface are compatible. However, the effects of the horizontal stresses (confinement) in the pavement are still not accounted for in the linear elastic approach (Brown, 1996).

To accommodate these factors finite element programs such as Fenlap (Brunton and d'Almeida, 1992) and Illipave (Thompson, 1992) have been produced, which give more rigorous solutions to the analysis of granular pavements. However any solution to the computed stresses and strains can only be as good as the material models used to predict material behaviour (discussed in Section 2.5, see Brown, 1996). In addition, for back-analysis of layer stiffnesses in a system with more than two unknown layer properties analysis will result in a number of possible solutions to the problem due to the positive and negative stress-dependency of the typical materials modelled.

Analysis of the collected data from dynamic stiffness measuring devices assumes that the deformations (inferred from the measurement transducers) are elastic (i.e. fully recoverable). In addition they are measured at the surface. The deflection is measured upon loading rather than unloading (as is the case for repeated load triaxial testing). Therefore there will inevitably be some discrepancy between the two sources of data as
the small element of permanent deformation induced with loading from the dynamic plate will be included in the calculation of stiffness. This error will be greater at higher applied stress (Fleming, 2000).

2.12 Conclusions

Current UK pavement foundation design is empirically based on the CBR test, and this has limited the use of analytical pavement design since the functional performance parameters of stiffness and resistance to permanent deformation are not directly measured. This has therefore resulted in inefficient design. If a performance approach is to be adopted, then these performance parameters must be measured at appropriate stress and environmental conditions so that test data can be used directly in design and then checked on site by measuring similar parameters. Therefore appropriate laboratory and field tests to measure these performance parameters are required and correlations between the field measuring devices themselves and between the field and laboratory tests need to be established.

The loading applied to a pavement foundation during construction is extremely complex, varying in both magnitude and duration of stress pulse. Construction plant and vehicles when moving induce a rotation of the principal stresses within the materials, the magnitude of which varies with the depth within the pavement and the stage of construction that is being considered. To model this loading for testing, simplification of the loading regime and of pavement structure is required.

The load response of materials used in pavement foundations varies with changes in load rate, applied stress, degree of confinement, water content, material type and stress history, and for fine grained soils suction. The stiffness of granular materials has been shown to be directly stress-dependent, whilst the stiffness of cohesive materials is inversely stress-dependent. Materials subject to repeated loads in the laboratory have shown that the
amount of permanent deformation sustained increases exponentially above a value of stress regarded as the threshold or shakedown limit.

The behaviour of the materials in the field under trafficking is different to the response seen in laboratory testing. This is due to the composite structure of the pavement and the effects of transfer of load through the layered structure. Composite behaviour leads to changes in the stresses experienced in other layers and variation in material properties of the layer itself due to its stress dependency and confinement provided by adjacent materials. Laboratory test data must therefore be used in a model of composite pavement response, to enable prediction of measurable parameters for field assessment.

The stiffness of a pavement foundation improves with thickness and the strength/stiffness properties of the materials used to construct it. A minimum value of stiffness is required of a supporting layer to ensure that adequate compaction of the subsequent layer can be achieved. The parameters of stiffness and resistance to permanent deformation are indirectly linked, i.e. the stiffness response of a material can influence the material's resistance to permanent deformation, which in-turn can affect the stiffness due to changes in stress state and history.

Methods to predict the changes to the materials caused by the environmental conditions that prevail in a pavement foundation, and methods of preparing samples in these states, are required in order to enable assessment of the long-term performance. Determination of the likely change in parameters as a result of the establishment of equilibrium conditions and preparation of suitable samples presents considerable difficulties. The weather experienced during construction and the degree to which a material is reworked (i.e. disturbed) have a significant influence on the effect of the changing environment.

Currently no suitable routine laboratory tests exist that are capable of measuring the performance of stiffness and resistance to permanent deformation for granular materials with large particle size, typical of those used in pavement foundations.
No accurate model of the permanent deformation behaviour of a pavement foundation currently exists, due to the complex loading and layer interaction and the stress-dependency of the materials. No single test exists which has been shown able to predict permanent deformation. The only method to assess resistance to permanent deformation accurately is wheel tracking tests on a model or full-scale pavements.

The shear strength of materials has been shown to play an important role in rut formation. Hence it is suggested that rut formation can be controlled by limiting the stress applied to the subgrade. It is believed that this threshold value can be predicted from material behaviour and possibly inferred from a strength test.

Measurement of the stiffness of fine grained soils at appropriate stresses can be undertaken in the repeated load triaxial test using on-sample instrumentation. However the effects of rotation of principal stresses cannot be modelled in this apparatus, which affects the relevance of permanent deformation data obtained. Apparatus does exist that can simulate rotation of principal stresses, but its use is limited to carefully prepared laboratory samples rather that undisturbed soil samples.

A variety of field equipment is available which measures 'stiffness' of materials under test. They range in size and complexity, and some are still under development. It would appear, in general, that the FWD has become accepted as a benchmark test against which to compare others, and that impulse type test devices can apply stresses of appropriate duration and magnitude that mirror the true loading sustained. Significant field testing has occurred in the UK on granular capping materials using the FWD and the TFT, and to a much more limited degree using the GDP.

Trafficking trials have been performed to assess the traffickability of various materials and foundation designs. Attempts have been made to correlate rutting to a measurement of strength via the DCP. Observations of rut formation have been also compared to top of
foundation stiffness. Adequate performance has been found if a certain level of foundation stiffness is achieved, the level of this acceptable stiffness varies depending on the device used to measure it. Although trials utilising in-situ parameter measurement have been performed little assessment during the construction process has been made, thus the affects of the changing influence of the subgrade on the performance of the foundation is unknown.

Many significant research developments have occurred in recent years, both in terms of field assessment techniques, laboratory testing and material modelling. However the two sets of developments have not been well integrated. Attempts to correlate laboratory and field derived performance data have been made but the correlations tend to be poor due to the nature of the testing performed and the methods used to measure or infer the parameters. Few directly comparable data have been assessed to date.
Table 2.1. Summary of Target Properties for Capping and Sub-base (after Nunn et al 1997)

**Table B12 Initial end-product acceptance criteria**

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<th>Dry density</th>
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<td>Formation</td>
<td>Foundation</td>
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<td>% of vibrating</td>
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<td>hammer test result</td>
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<td>&gt; 90 %</td>
<td>&gt; 92 %</td>
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<th>Elastic stiffness</th>
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<td></td>
<td></td>
<td>Formation</td>
<td>Foundation</td>
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<td></td>
<td>Test method</td>
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<td>FWD</td>
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<td>Stiffness (MPa)</td>
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<td>40</td>
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<tr>
<td></td>
<td>PDPBT</td>
<td>65</td>
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Note. PDPBT = German Dynamic Plate Test (GDP)
Formation = Top of Capping
Foundation = Top of Sub-base
Figure 2.1. Stress Conditions Under a Moving Wheel Load (a) Stresses on a Pavement Element; (b) Variation of Stress with Time (After Brown 1996)

Figure 2.2. Stresses on a Pavement Element (a) Principal Stresses- Element Rotates; (b) No Rotation – Stress Reversal (After Brown 1996)
Figure 2.3. Variation of Equivalent Vertical Stress Pulse Time with Vehicle Velocity and Depth (After Barkesdale 1971)

Figure 2.4. The Load Spreading Capabilities of a Pavement (after Brown 1998)
Figure 2.5. Vertical Subgrade Stress Below a 165mm Asphalt Construction Subject to a Vehicle Pass (After Brown 1996)

Figure 2.6. Vertical Subgrade Stress Below a unsurfaced 350mm Granular Layer Subject to a Vehicle Pass (After Little 1993)
Figure 2.7. Stresses Measured at 75mm Depth Beneath a 1 Tonne Double Vibrating Roller Compacting a Well-Graded Sand (After Parsons 1992)

Figure 2.8. Stresses Measured at 130mm Depth Beneath a 7 Tonne Double Vibrating Roller Compacting a Well Graded Sand (After Parsons 1992)
Figure 2.9. Critical Stresses in a Pavement (After Powell et al 1984)

Figure 2.10. The CBR Test (After Croney 1992)
Figure 2.11. Resilient and Plastic Strains During one Cycle of Stress Application (After Thom 1988)

Figure 2.12. Graph of Shear Stress against Shear Strain for a Typical Granular Material (After Thom 1988)
Figure 2.13. Influence of Shear Stress Reversal on Accumulation of Plastic Strain in a Dry Crushed Rock (After Chan and Brown 1994)

Figure 2.14. The Influence of Stress History on Permanent Strain Accumulation (After Monismith et al 1975)
Figure 2.15. The Influence of Stress Sequence on Permanent Strain Accumulation (After Monismith et al 1975)

Figure 2.16. Variation of Density with Depth (After Thom 1988)
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Figure 2.24. The German Dynamic Plate Bearing Tester (GDP)

Figure 2.25. The TRL Foundation Tester (TFT)
Figure 2.26. The Clegg Hammer

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Figure 2.28. Density Measuring Apparatus
Figure 2.29. Dependence of Foundation Stiffness on Test Device for the FWD, TFT, and Loadman (After Chaddock and Brown, 1995)
Figure 2.30. Correlation of the FWD, TFT and GDP for Many Foundations (After Shahid et al, 1997)

Figure 2.31. A Modified Sherby-Dorn Plot for Limestone (After Lekarp and Dawson 1997)
3.0 RESEARCH PHILOSOPHY

3.1 Introduction

This chapter aims to explain the philosophy behind the experimental methodologies adopted to investigate the hypothesis. Deriving from the literature review, it aims to explain the selection of the tests and programme of work performed to satisfy the aims and objectives of the thesis. The precise details of the methodologies and apparatus used are described in Chapter 4.

3.2 Philosophy of Research

In order to assess the influence of the subgrade on pavement foundation performance during construction, the two performance parameters of stiffness and permanent deformation must be assessed under realistic construction conditions. Therefore, it was proposed to carry out a series of controlled full-scale field trials, in which trial sections were constructed using appropriate plant and materials and subjected to appropriate loading. By varying the construction thickness and the materials used, and measuring the performance parameters during construction (using various techniques identified in Chapter 2), the change in the composite performance of the foundations and hence subgrade influence was assessed. Only the subgrade could be tested in isolation during the trials since field testing of the materials that form the overlying layers is influenced by the supporting substrate. By measuring the performance of materials in the laboratory at comparable conditions, and attempting to define links between the laboratory and field measurement techniques for the parameters of stiffness and strength, the individual material behaviour could be related to the composite foundation performance, and hence the influence of the subgrade on the composite performance assessed by back-analysis.
3.3 Laboratory Performance Testing

3.3.1 Test Selection

3.3.1.1 Granular Materials

Testing of granular materials with large particle sizes in the laboratory to simulate the conditions *in-situ* and material states with an appropriate applied stress/strain regime is very difficult. No routine test apparatus has yet been developed that can test the performance of granular materials with large particles, although some index tests (such as the PSSR) are available for materials containing particles up to 37.5mm in size. The testing of granular materials therefore remains an area requiring research and lies outside the scope of this thesis. For these reasons, testing of the granular materials used has been limited to classification tests only.

3.3.1.2 Fine Grained Soils

Repeated load triaxial test apparatus (using lightweight on-sample axial strain measuring devices and non-contact radial strain transducers) has been developed for assessing the performance of fine grained soils specifically to obtain data for inclusion in an analytical pavement design (Cheung, 1994). This apparatus has been used for the research reported herein for testing fine grained (cohesive soils) typical of those found as UK subgrades. The apparatus is described in detail in Appendix A.

However, little information has been published on the accuracy and data variability of this equipment, and practical use of the data has not as yet been attempted. The apparatus was validated during its development by comparing the collected data to results from other triaxial testing on similar materials. Therefore assessment of the methodology of testing and error analysis of the data collected from the apparatus forms a part of this research.
3.3.2 Sample Preparation

It is important to test the soils at the anticipated environmental conditions that will be experienced on site. To facilitate this samples need to be modelled at their anticipated field conditions.

Samples for testing at the in-situ water content (to mimic the conditions found at the time of construction) can be prepared from U100 samples of the subgrade. An undisturbed sample can be prepared directly from a U100 sample by trimming. If a test on a remoulded sample (to mimic the situation on an embankment) is to be performed, then the sample can be broken down and re-compacted.

Various sample sizes have been tested in repeated load triaxial test equipment, and various sample preparation methods have been suggested. 100mm diameter disturbed samples are typically prepared by compaction (Mohammad, 1995, Lee et al, 1997, Bohra et al, 1997) either to achieve a target density or by application of a specific energy. Cheung (1994) proposed a method of compaction similar to the light-weight Proctor Compaction test (BS1377 Part 3, 1990) and to that proposed by Head, (1992) for shear strength testing. This incorporates a kneading type of compaction which has been shown to produce resilient properties similar to those of samples compacted in the field to the same degree of saturation (Seed et al, 1962). Therefore this method of compaction was adopted herein.

In order to determine the behaviour/response of subgrade that has been subject to detrimental changes in its environmental conditions, samples need to be prepared at predicted worst case water contents. Methods of predicting equilibrium water content and the problems of preparing samples at the predicted water content have been discussed in Section 2.6. This issue is peripheral to the work reported herein since no fieldwork on trials at equilibrium water content was planned. However for completeness some laboratory testing at predicted equilibrium water content was performed.
The method suggested by Black and Lister (1979) for predicting equilibrium water content is included in the current UK Pavement Design Manual (Vol. 7, DMRB, 1992), and was adopted in this research. Although few field studies exist to validate this method, the authors suggest that it provides a conservative estimate for equilibrium water content. Since this method is based on the plasticity data for the soil, and this is established on material passing a 425μm sieve, it was anticipated that mixed soils could present a problem. However the other prediction methods available have been shown to be inappropriate for this research (Section 2.6).

Because the techniques of preparing undisturbed samples at equilibrium water content under appropriate stress conditions are complex, and are of secondary importance to the present research, a simple method of sample preparation was proposed. Equilibrium in-situ water content samples were not created by allowing undisturbed U100 samples to wet to equilibrium. Although it is possible to manufacture laboratory samples by consolidation from a slurry, these take a considerable time to produce and would not give suitable information to compare to the field data. Appropriate triaxial samples were therefore produced by mixing sufficient water to the soil to bring samples to equilibrium water content. The soil was then compacted to form a sample as described above. It is appreciated that the behaviour of these samples would be different to samples which have been remoulded and then brought to equilibrium, since the addition of water will affect the compaction of the remoulded samples. Nevertheless it was hoped that this remoulding to equilibrium (which is allowed for in conventional CBR design) would create a worst case equilibrium condition (because of destruction of the soil structure), thus giving an appreciation of worst case soil properties.
3.3.4 Test Methodology

Previous work with the repeated load triaxial test apparatus used for this research has focused on the equipment, with proof testing concentrating on recompacted/reconstituted laboratory prepared samples. For the collection of data for analytical design, as proposed by Cheung (1994), separate nominally similar samples have been used to derive resilient and permanent deformation behaviour. The methodology included conditioning the samples to 1% permanent strain, prior to obtaining resilient strain data (the 1% strain level was proposed as a threshold, where the onset of permanent deformation was observed to become unstable). Resilient data were then collected at fractions of the deviator stress at which 1% strain was reached (e.g. $\frac{1}{6}$, $\frac{1}{3}$, $\frac{1}{2}$, $\frac{2}{3}$ and $\frac{5}{6}$). This conditioning was similar to that used in tests developed in the United States (Barkesdale et al., 1975).

Because of the anticipated variability between data collected from undisturbed samples of soil obtained from the same site (due to differences in water content, soil type and stress history as well as inhomogeneity), it was decided that resilient and permanent deformation data should be collected concurrently from the same sample. Although conditioning is not thought to affect the resilient behaviour (Cheung, 1994), the stresses at which the stiffnesses are measured (using Cheung's methodology) may not be representative of the stresses imposed during the life of the pavement. In addition, conditioning is expected to affect the permanent strain data collected (Monismith et al., 1975). Therefore, for standard testing it was proposed to test samples without conditioning, starting at a low deviator stress for 1000 cycles (to mimic the expected maximum trafficking on site) and progressively increasing the deviator stress for additional sets of 1000 cycles. The stress pulse should be applied at a frequency that represents a load pulse similar to that applied by the passage of a vehicle (Section 2.2), thus a representative frequency of load pulse of 2Hz has been suggested by Cheung (1994) and was considered acceptable for this research.
The confinement of materials within the pavement has been shown to vary according to a number of factors including vehicle loading (Section 2.5), increased confinement improving the performance of materials. For conservative testing no confinement could be used during testing (Cheung, 1994). However this would be unrepresentative of the true situation and would not be practical for non-cohesive materials that require some confinement to maintain sample shape/structure. To allow comparison with in-situ tests comparable confinement should be used. However, the amount of confinement provided during testing with a dynamic plate is difficult to assess. For testing subgrade beneath a foundation the basic confinement at the subgrade interface is provided by the pavement overburden. As it is not possible to cycle the confining stress to mimic traffic loading it was proposed to use a low level of confinement similar to pavement overburden, which will give conservative test data. A confinement of 20kPa was adopted based on 1m of pavement, of unit weight 20kN/m³ with earth pressure at rest K₀=1.

These test regimes enabled resilient and permanent strain data across a full range of stresses typical to those experienced during the life of a pavement to be obtained. However, since this test methodology must be evaluated it was anticipated that some revision of the methodologies employed would be undertaken in the light of the findings during the course of the testing.

3.3.5 Test Programmes

In order to obtain performance data for the various trial sites, a number of undisturbed samples were collected and tested. This formed the bulk of the testing performed. Remoulded samples were also made and tested, although the degree of remoulding of the subgrade during construction of the field trials was small, since the subgrade would not be accessed by plant during construction. In order to assess the worst case performance of material when subject to water content changes, a limited amount of testing was performed on samples of soil remoulded at equilibrium water content. The influence of the construction operations of remoulding and the effects of long-term water content
change could therefore be assessed relative to in-situ performance (i.e. where no remoulding has occurred).

In addition to the testing of samples collected from the field trials it was also proposed to undertake limited testing on soils from other sites. Hence, the apparatus was evaluated by testing a range of fine grained materials having a variety of properties (i.e. from soft to very stiff), and a full range of material behaviour assessed.

Little work has been undertaken to assess the effects of conditioning upon the test data collected, especially at the low deviator stresses experienced beneath a thick granular layer. If very limited conditioning is applied, this should not affect the permanent strain behaviour at the critical area where it becomes unstable (i.e. at the threshold, see Monismith et al, 1975). A small element of conditioning may reduce potential errors since conditioning is frequently used to ‘bed in’ the sample instrumentation. To assess the effects of the conditioning a short programme of conditioned tests was performed.

In order to assess variability and repeatability of the tests due to instrumentation errors and precision, a series of repeat tests was performed upon samples of the same soil at the same water contents using the same sample preparation techniques. Further tests on dummy triaxial samples of various stiffnesses were also proposed in order to assess the variability of the data collected from the apparatus relative to that collected from soil samples. This allowed assessment of the sources of any errors with the equipment.

3.4 Field Testing

3.4.1 Test Equipment

The field assessment of resilient and permanent deformation properties of materials should be performed separately. Methods of assessing stiffness and permanent deformation are discussed below.
3.4.1.1 Stiffness Measurement

Similar to the laboratory testing, it is important that field testing assesses material parameters at comparable stresses to that experienced during construction (i.e. applied stress, pulse duration). In Section 2.8 various devices were reviewed that measure, or infer, the stiffness of highway foundation materials. Some devices have already been discounted since they are either unavailable in the UK or have been shown to be inappropriate for use on coarse unbound materials. Four devices were identified for further consideration: the Falling Weight Deflectometer (FWD), German Dynamic Plate (GDP), TRL Foundation Tester (TFT) and the Loadman.

Of these four devices the FWD is the most widely recognised and is regarded as a benchmark for the other devices, and has therefore been selected for inclusion in this research. Positioning the FWD for testing at all points within each layer during construction of the trials could be difficult due to their stepped structure, the size of the device and need for a vehicle to move it. It was therefore proposed to use the FWD to assess final stiffnesses only on the top layers of the trials, where easy access could be obtained. The FWD was used to give assurance of the accuracy of the other devices, which are portable and could therefore be used throughout.

The Loadman has been shown to stress to a higher level at higher frequency than the other three devices, and has a maximum plate size of only 200mm. It was therefore decided that this device would not be used since results cannot be easily compared with those from the FWD, TFT or GDP, which all have similar plate sizes and have overlapping ranges of applied stresses.

Only the FWD and TFT measure both the applied stress and deflection. In addition the operator can control the applied stress with these devices, which is important when testing stress-dependent materials. The stress-dependency of the trial foundations was
assessed over a range of stresses with the TFT. The GDP is calibrated such that the interpretation of a stiffness value is based on an assumed contact stress (similar to the Loadman). Although problems with the accuracy of the GDP have been highlighted (Shahid et al, 1997), it is a simple and quick test to perform and was therefore used with the TFT to provide a comparison throughout the testing.

Since the GDP assumes a contact stress of 100kPa, the FWD and TFT were used at a similar stress level for direct comparison. 100kPa is at the lower end of the range of stresses that the FWD can apply. The TFT applies a maximum stress of approximately 125kPa using a 300mm plate and the greatest drop height on a relatively stiff subgrade. There is consequently little overlap of contact stresses between the FWD and TFT. It was therefore proposed to take three readings at 100kPa with the FWD, in order to obtain correlations between all three devices at this contact stress. Limited stress-dependency testing at higher stresses was undertaken with the FWD only at the end of the trials to provide data for the completed foundations. This would remove any possible influence of the high contact stresses used by the FWD affecting the response of the materials at the lower stresses applied by the TFT and GDP. The appropriateness of the magnitude of the applied stresses used for testing, relative to the back-analysed applied stress beneath the completed foundations relative to the applied loads, was assessed. In order to assess the variability and repeatability of the stiffness measuring devices, a series of tests were undertaken on rubber sheets.

3.4.1.2 Permanent Deformation

Due to the complex nature of the formation of rutting, assessment of the permanent deformation characteristics of materials in isolation cannot be considered to be adequate. Consequently assessment of behaviour under a wheel load has been suggested as the only acceptable way to assess rut formation (Chan, 1990). Therefore, some form of trafficking should be undertaken on the trials during which the formation of rutting should be monitored. Strength has been defined as a critical parameter of assessing the tendency of
materials to rut. Of all the devices reviewed to assess strength, only the DCP causes minimal disturbance to the constructed foundation. However, its use has been shown to be unreliable in rut prediction since it does not allow for interlayer effects, although it does allow for assessment of the in-situ strength of materials which may give an indication of acceptable performance.

3.4.1.3 Ancillary Testing

To assess the adequacy of the compaction of the trials it was proposed to measure in-situ compacted density of the trial materials using the Nuclear Density Gauge (NDG). By comparing field density to the maximum value of laboratory compacted density for the materials, the stiffness of support required to enable adequate compaction of the subsequent layer can be assessed. Ensuring adequate compaction should prevent excessive rutting due to further densification of the materials under wheel loads.

3.5 Field Trials

Each trial needs to be designed to provide specific information about the performance of pavement foundations. The philosophy of design for each trial therefore varied according to the aims of that trial. Each trial was monitored during its construction, and was subsequently trafficked by a laden lorry.

3.5.1 Trial 1 (The Hathern Trial)

The first trial was conducted to investigate the performance of a foundation constructed using current design standards (Vol 7. DMRB, 1992) and materials that comply with the current UK specification (Vol 1. MCHW, 1992). The aim was to assess what level of performance can be obtained upon a series of standard cappings. The trial was constructed in accordance with the current method specification on a relatively poor subgrade to assess lower bound performance, and assessed how performance varied
between different material types and the amount of compactive effort applied to them. The trial was then trafficked with a laden lorry to assess its rutting potential. Assessment of the properties of the subgrade was made before construction of the capping, and the capping assessed before and after trafficking, and in the long term after a period of poor weather. This gave a basis to compare performance, since it would be indicative of the likely parameters that would be found on live construction sites.

3.5.2 Trial 2 (The Bardon Trial)

To assess the minimum level of top of capping stiffness (measured insitu) required to enable satisfactory compaction of sub-base a second trial, the Bardon Trial, was proposed. Formations of different stiffness were prepared by constructing different thicknesses of capping, the performance of which was assessed during construction. Sub-base was then laid and compacted upon the formation, and again its performance was monitored during construction using measurements of stiffness and density. Acceptable compaction was defined as a proportion of the maximum laboratory achieved dry density. Again a relatively poor subgrade was sought in order to enable a wide range of stiffness formations to be prepared. The trial’s performance was monitored in the long term, and subsequently trafficked to assess the stiffness required to limit rutting.

3.5.3 Trial Three (The Mountsorrel Trials)

Two concurrent trials (the Mountsorrel Trials) were constructed to assess the performance of two marginal capping materials, with designs being based upon a performance approach using the laboratory triaxial test subgrade data. The trials were constructed with areas of capping thicker and thinner than the design to allow assessment of a lower bound design value. The change in performance parameters was again measured during construction, and the capping was trafficked prior to the installation of sub-base, which was assessed to validate the target values found from the previous trial. Again the trial was monitored over a period of time to assess weather effects.
3.6 Additional Laboratory Testing

Standard laboratory based classification tests were proposed to characterise all the materials tested in the laboratory performance tests and the field trials, to allow a simple assessment of the relative characteristics of the materials.
4.0 METHODOLOGY

4.1 Introduction

This chapter describes the experimental work carried out to test the hypothesis (Section 1.3). This includes the classification tests performed upon the materials used, the specimen preparation and test regimes used for the repeated load triaxial testing, and the construction and in-situ assessment of the three field trials.

4.2 Classification Tests

Standard laboratory tests have been carried out on the subgrade, capping and sub-base material collected from various sites assessed as part of the research.

4.2.1 Aggregate Classification Testing

4.2.1.1 Grading

Samples of the capping and sub-base materials collected from the field trials have had grading tests performed upon them in accordance with BS 1377 Part 2 (1990). These tests were performed to assess the compliance of the granular materials relative to the specified gradings included in Volume I MCHW (1994) Tables 6/2 and 8/2.

4.2.1.2 Peak Shear Stress Ratio

Peak Shear Stress Ratio (PSSR) tests were performed on the aggregates used in the field trials. The tests were carried out in a 300 mm shear box using the method described by Earland and Pike (1984).
4.2.1.3 Compaction Tests

Vibrating hammer compaction tests in accordance with BS5835 (1985) have been performed on the capping and sub-base materials collected from the field trial sites in order to establish the laboratory maximum compacted densities and optimum water contents.

4.2.2 Subgrade Classification Testing

4.2.2.1 California Bearing Ratio (CBR) Tests

Various laboratory CBR tests have been performed on the subgrade samples collected from the field trial sites. Typically remoulded tests have been performed, although a small amount of testing of soaked remoulded samples has also been conducted. All tests were performed in accordance with BS 1377 Part 4 (1990), using three surcharge rings to provide a nominal confinement of 4 kPa to the samples.

4.2.2.2 Index Tests

Atterberg Limit tests in accordance with BS1377 Part 2 (1990) were carried out on representative samples of the subgrade retrieved from the field trial sites. The specific gravity of the subgrade soils were determined in accordance with BS 1377 Part 2 (1990).

4.3 Repeated Load Triaxial Testing

4.3.1 Introduction

Repeated load triaxial testing has been undertaken on samples of subgrade soil collected from the three field trial sites, from various road construction sites and from local clay pits. The samples have been prepared in different ways, as described in Section 3.3. The
samples were collected by hand driving U100 tubes into the subgrade. A full description of the apparatus, (shown in Figure 4.1), sample instrumentation, data collection system and sample set-up procedures are presented in Appendix A.

4.3.2 Specimen Preparation

Undisturbed samples were extruded from a U100 tube by static extrusion and prepared directly by trimming to length (200mm).

Remoulded samples were prepared from soil broken up in a mixer until it had formed lumps that would pass through a 5mm sieve. The soil was then compacted (BS 1377, Part 4, 1992) into a 100 mm diameter, 250 mm high Procter compaction mould in five equal layers, using a 2.5 kg hammer falling through 300 mm, 27 blows per layer (Head, 1992). The sample was then extruded from the mould and trimmed to length.

Remoulded samples at equilibrium water content were manufactured by mixing the amount of water predicted from LR889 (Black and Lister, 1979) into a remoulded sample, prior to compaction and preparation as described above.

4.3.3 Test Methodologies

4.3.3.1 Load Regimes

Samples were tested in the undrained condition with an applied cell pressure of 20 kPa, in order to simulate the confining stress beneath a typical road pavement (Section 3.3.4) and were then subjected to an axial seating deviator stress of 5kPa. A seating stress is required to prevent the load platen from lifting off the sample during cycling and affecting the strain readings.
The samples were loaded with sinusoidal load pulses at a rate of 2 Hz. The load phase of the pulse was 0.25 seconds, followed by a sample rest period of 0.25 seconds, to simulate the load pulse from the passage of a wheel. The samples were loaded with increasing levels of deviator stress for 1000 sinusoidal cycles applied at each deviator stress. The deviator stress was typically increased in increments of 10 kPa until a repeated deviator stress of 100 kPa was applied. Increments were then increased to 20 kPa. Once 5% permanent strain was sustained by the sample, the on-sample instrumentation was removed. The sample was then loaded monotonically at a rate of 5 mm/minute until failure occurred (BS 1377 Part 7, 1990) so that the undrained shear strength \((C_U)\) could be established.

4.3.3.2 Analysis Procedure

During a set of 1000 cycles, data from the last 5 cycles of every 100 cycles were collected. The resilient moduli at each level of deviator stress were calculated from the 999th cycle, with the final 5 cycles being investigated to ensure that the 999th cycle was representative. The resilient modulus was calculated as the repeated deviator stress upon unloading (adjusted for the change in area of the sample, due to resilient and permanent deformation at the peak value of applied deviator stress) divided by the resilient strain upon unloading.

Area correction to calculate applied stress was determined from either the radial proximity transducer data or the constant volume area correction (BS 1377 Part 2, 1990). Permanent strain was calculated from the overall change in length of the sample during the test.

4.3.3.3 Standard Tests

The sample was initially subjected to cyclic loading with a repeated deviator stress of 10 kPa for 1000 cycles (excluding the 5 kPa seating stress). The load was then cycled to
produce a repeated deviator stress of 20 kPa for 1000 cycles, and then at increasing increments of deviator stress as described previously.

Early assessment of these results indicated that further stiffness measurements at very low deviator stress would be required. Readings at 5 kPa were therefore proposed. Due to the method of sample instrumentation connection, relatively high variability between strain gauge readings was observed at low deviator stress, which reduced at higher stresses. This was initially attributed to the soil sustaining a small amount of permanent deformation around the instrumentation that improved the connection of the on-sample instrumentation for subsequent readings. It was felt that full cycling at 5 kPa would not achieve this improvement in the instrumentation connection since limited permanent deformation would be generated at this stress (which is at the limit of the lower level of cycling that can be provided by the apparatus). Therefore the sample was tested with the 10 kPa cyclic stress prior to testing at the 5 kPa deviator stress. It was felt that no significant change in sample behaviour would occur due to preconditioning with the 10 kPa deviator stress due to its low magnitude and hence limited effect on permanent deformation response, and due to the insensitivity of stiffness to stress history (Sections 2.5.5. and 2.5.6). Only 10 cycles at 5 kPa were applied, since minimal permanent deformation was expected to occur due to the fact that the sample has already been stressed to a level above this value, and this should therefore not affect the resilient response. The sequence of loading is presented in Table 4.1.

4.3.3.4 Sample Conditioning

Tests using conditioning were undertaken to evaluate the effects of conditioning and the improvements in accuracy and repeatability of measurement achieved with the on-sample strain gauges by improving the connection between the cruciform studs and the soil. The amount of conditioning applied was initially limited to a low level where the applied deviator stress would not significantly change the response of the sample when subjected to its threshold stress. Series of both standard and conditioned tests were undertaken to
compare the effects of conditioning. Stress-controlled conditioning was proposed as a result. The soil samples were subjected to 1000 cycles of 10, 20 and 30kPa repeated deviator stress, prior to being tested according to the standard loading regime (Table 4.1).

This stress-controlled conditioning was found to have little effect on stiffer samples since the amount of permanent deformation generated was insufficient to “bed down” the on-sample instrumentation. Consequently, strain-controlled conditioned tests were performed whereby the material was loaded with increasing 10 kPa increments of deviator stress until 1% permanent strain was sustained, similar to that used by Cheung 1994. The loading regime is presented in Table 4.1.

4.3.4 Repeatability and Error Assessment Tests

4.3.4.1 Repeatability Tests on Subgrade

For assessment of the repeatability of the apparatus, a bulk sample of weathered Mercia Mudstone subgrade collected from the Mountsorrel field trial was broken down in a mixer and all the gravel particles removed. A triaxial sample was then manufactured using the methodology described in Section 4.3.2. The sample was then re-tested using the methodology described in Section 4.3.3. The sample was broken down again, re-mixed and re-manufactured and re-tested. This process was repeated four times, after which the soil was then broken down again and de-ionised water added and mixed. Another series of four tests, as described above, were performed. In all three series of four, repeat tests were performed at water contents of approximately the plastic limit, and 2% and 4% above plastic limit.

4.3.4.2 Tests on Synthetic Dummy Triaxial Samples

Dummy triaxial samples 100 mm in diameter were manufactured for testing to assess the accuracy and repeatability of the triaxial test measurements. Samples were manufactured
from two pieces of solid 100mm diameter steel bar, with 100mm diameter rubber disks of 22 mm thick rubber (Shaw hardness 60) glued between them. One dummy specimen was manufactured with one thickness of the rubber, and a second specimen was made with two disks of rubber. The steel bars were cut to length to provide an overall sample length of 200 mm. The steel sections were drilled and threaded to accept direct connection of the on-sample strain gauges across the middle third of the sample, thus eliminating the cruciform studs.

The main area for concern with regard to accuracy of the repeated load triaxial testing was small strain measurement at low deviator stress. The test regime used was designed to assess this. A confining stress of 20 kPa and a 5kPa seating stress was applied. The samples were loaded with 100 cycles of increasing deviator stress (5, 10, 15, 20, 30, 40, 50, 75, 100, 125 and 150 kPa) and readings from 5 cycles every 10 cycles were stored for analysis. Three tests using this regime were undertaken on each sample.

4.4 Field Trials

4.4.1 Trial One (The Hathem Trial)

4.4.1.1 Trial Layout and Construction

The site was located on pasture and at the edge of the flood plain of the River Soar at Hathem, Leicestershire. The road foundation was designed to remain after completion to form an access track, and consequently the layout of the trial was subject to a number of constraints.

A site investigation consisting of trial pits, sampling and in-situ testing, including extensive DCP testing, was carried out. Generally the ground was found to consist of 0.2m of topsoil overlying 1m of very soft sandy silty clay with occasional gravel, overlying a wet loose silty sand with occasional gravel. The purpose of the trial was to
assess a series of standard foundations designed in accordance with the current UK design standard (HD25/94, DMRB). Therefore from the laboratory CBR tests on the subgrade and consideration of the DCP test data (related to CBR), a design thickness of 450 mm was chosen for the trial. Further details of the site investigation and the derivation of the design thickness are included in Appendix B.

The road foundation was designed as two separate 18m lengths linked by a turning area, to prevent vehicle braking and turning disturbing the materials and influencing the measurements made during trafficking on the monitored bays. Each 18m length was divided into six 3m long bays. The first three bays (bay numbers 1 to 3) were constructed to form a wedge varying in thickness from 600 to 300mm (i.e. the design thickness ±150mm). The remaining 9 bays were built at the design thickness (bays 4 to 12). The trial sections were constructed 4m wide to ensure both that the wheel loads would be transferred directly to the formation and ensure minimum overlap of the roller runs during compaction. The layout of the trial is shown in Figure 4.2.

Three test points on the three rows (Rows A, B and C), were located in each bay, giving 9 tests per bay. Row B was positioned on the centreline of the road, while Rows A and C were positioned in the outer wheel path of the trafficking vehicle.

Two standard Type 6F2 cappings were used, Porphyritic Andesite from Bardon Quarry and Oolitic Limestone from Great Ponton Quarry, together with one waste material, Porphyritic Andesite Quarry waste (40 mm Crusher Run, normally used as general fill) from Bardon Quarry. These materials were selected to assess two different types of coarse capping of different mineralogy and to assess two similar materials with different gradings.

The trial area was excavated to reduced level along the entire length of the road. The formation was proof rolled with one pass of the roller (a Benford 800 TV vibrating roller with the vibrator turned off, Figure 4.3). Table 6/4 MCHW states that this roller can
adequately compact capping with 8 passes applied to a layer 110 mm thick. A slotted drain pipe was installed in gravel along the edge of the low side of the formation to provide drainage.

The capping materials were placed in three bays at a time in 110 mm thick layers and compacted using the roller with vibration. The thickness of the top layer was adjusted to make up the required target construction thickness. Three levels of compactive effort (good, medium and poor), using 16, 8, and 4 Passes respectively.

Each capping material was placed 20 mm thicker than the target thickness, in order to allow for volume reduction due to compaction, trimmed and compacted with half the required number of passes. The material was then re-trimmed and the compaction completed. The location of materials and the levels of compaction applied in each bay are shown in Figure 4.2.

4.4.1.2 Assessment of Trial Sections

Bulk samples of the excavated subgrade and granular materials used in the trials were taken for laboratory characterisation. U100 samples for repeated load triaxial testing were taken at the centre test point of Row B. The depth of excavation to formation resulted in some of the U100 sample tubes being driven through the sandy clay to the interface with the underlying silty sand. Consequently only six full length U100 samples that were suitable for testing could be recovered.

Stiffness measurements on the subgrade and capping were performed using the FWD, TFT and GDP, as proposed in Section 3.4.1.1. In order to establish the effects of multiple stiffness measurement at a single location, a single discrete test was performed with each device on an individual location in each row in each bay, making three single tests with each device within in each bay. Additional tests were then performed using the other two devices on the middle of point in each bay, providing three multiple test points in each
bay. The layout of the test positions and order of use of each device within each bay is shown (Figure 4.2).

Tests were performed using the following methodologies. The GDP was used in accordance with the manufacturer’s instructions using three conditioning drops and three measurement drops. The TFT test regime used three conditioning drops at an applied stress of approximately 100 kPa (the maximum drop height). Three measurement drops (one each of 0.25m, 0.5m and 0.75m respectively) were then performed to ensure the full range of stress was applied and allow interpolation to yield a stiffness at a stress of 100kPa. The FWD test regime consisted of six drops, three conditioning and three measurement drops. The target contact stress was set to 100 kPa for all drops. For the TFT and FWD the stiffness is calculated using the equation in Section 2.8.2.5. Poisson’s ratio values of 0.4 for subgrade and 0.3 capping have been assumed leading to a value of 0.35 for the composite foundation. These values are similar to those values given by Brown et al (1993) and used for the development of the TFT and by Croney (1997) (Section 2.5.2).

DCP tests were performed on the subgrade at the centre test point of each row in each bay following the stiffness measurements. No DCP tests could be performed through the cappings since the DCP could not penetrate them. NDG measurements were made on the compacted capping. One reading was taken in each wheel path, giving two readings per bay. Measurements were made using the direct transmission mode with a maximum probe penetration of 200 mm, to give a representative density of the top layers of the compacted materials.

The trial sections were trafficked with 1000 passes of a two axle lorry having a gross weight of 15.54 tonnes (Figure 4.4), in order to generate rutting. The load and dimensions of the lorry used are presented in Figure 4.5. The formation of rutting was monitored by levelling the test points in the wheel paths (Rows A and C). Additional points were then marked 300 mm to either side of the wheel paths to monitor the formation of rut
shoulders during trafficking. Monitoring of rutting was initially performed every 10 passes, and then every 100 passes as the rate of propagation reduced.

During construction the weather was very hot, so to assess the change in stiffness caused by drying, additional readings of stiffness were taken on the exposed areas of subgrade that were the last to be covered with capping.

Ten days after completion of the trafficking further series of stiffness measurements were made on the capping. Tests with the GDP were performed on the single GDP test locations and on the TFT test locations. TFT testing was only undertaken in Bays 1 to 6 before the TFT malfunctioned.

Following completion of the initial phase of the trial, the sections were left for eight months (August 1997 to May 1998). Stiffness measurements were then repeated to determine if any further changes had occurred. The road had been damaged during this period, therefore only limited testing could be performed. Stiffness measurements were repeated along the trial using the TFT and the GDP. Row A was tested along its full length, while Row C was tested in Bays 7 to 12.

4.4.2 Trial Two (The Bardon Trial)

4.4.2.1 Trial Layout and Construction

The site consists of an area of undisturbed waste ground adjacent to a quarry waste tip at Bardon Hill Quarry, Leicestershire. Only a limited site investigation was undertaken prior to construction. Two small trial pits were dug adjacent to the area of the trial. They showed the subgrade to be a weathered Mercia Mudstone, a silty clay containing regular 15 to 20mm Porphyritic Andesite gravel. As the purpose of the trial was to provide bays of different capping stiffness upon which to compact a 150 mm layer of sub-base, no defined design thickness was required. Five 3m long bays varying in thickness from no
capping to 400 mm of capping, in 100 mm increments, were constructed to provide a wide range of stiffness formations.

The width of the trial sections was 2.4m sufficient to allow three rows and three test points to be positioned in each bay, free from influence from the trial edges or adjacent testing (Figure 4.6). In order to provide a level top surface on which to compact the 150mm sub-base, the subgrade excavation was stepped.

The materials chosen for the trial were both standard highway construction materials in accordance with the current Specification for Highway Works. The capping used was a Type 6F1 (Table 6/4, Vol. 1 MCHW, 1994), a well-graded Porphyritic Andesite from Bardon Quarry. A material with a fine grading was selected to ensure that no large particles could affect the compaction of the layers. The sub-base used was a Type 1 (Table 8/2, Vol. 1 MCHW, 1994) well-graded Porphyritic Andesite from the same supplier.

Compaction was carried out with a Benford 1300HV vibrating roller (Figure 4.7). The 100 mm layers of capping were compacted using three passes of the roller in accordance with the Specification for Highways Works (Table 6/4, Vol. 1 MCHW, 1994). The sub-base was compacted with eight passes of the roller (Table 8/1, Vol. 1, MCHW, 1994).

The subgrade was excavated and trimmed to finished level in each bay immediately prior to installing capping, hence minimising the effects of rain or frost on the subgrade since construction took place in December. Bay 1 was excavated to finished level, the subgrade tested, and the first layer of capping was placed to achieve a thickness of 120 mm (i.e. allowing 20 mm thickness reduction for compaction). The capping was compacted with two passes of the roller and the materials were re-trimmed, prior to a further pass with the roller to complete compaction. The subgrade of the next bay (Bay 2) was then trimmed. The top of capping in Bay 1 and the subgrade in Bay 2 were tested, and the next 120mm thick layer of capping (in Bays 1 and 2) was placed and compacted.
Construction of the capping continued in this sequence until all the bays containing capping had been constructed. Finally, the subgrade in Bay 5 (the no capping bay) was trimmed and the sub-base formation tested. The sub-base was then laid to achieve a thickness of 175mm (allowing 25 mm for compaction) throughout the length of the trial. It was tested at three stages during its compaction: after 2, 4 and 8 passes of the roller. It was re-trimmed as necessary at the intermediate testing stages.

Following completion of the compaction trial and analysis of the compaction data it was decided to traffic the trial sections. Trafficking was not initially proposed at the time of construction. The trial sections therefore required widening to allow the passage of a vehicle. This work was undertaken four months after construction.

4.4.2.2 Assessment of Trial Sections

One bulk sample of excavated subgrade was collected from each bay. Bulk samples of the granular materials used in the trials were collected for characterisation in the laboratory. No U100 samples could be taken from the site, since the gravel in the subgrade made it impossible to drive the tubes.

Dynamic stiffness measurements were made during construction and trafficking was carried out using the methods described for each device in Section 4.4.1.2, with all tests being performed on the same points. The stiffness of the subgrade was measured in each bay on all points marked in Rows A and C (i.e. six tests per bay). Tests were performed using GDP and the TFT. In Bay 5 (the no capping bay) all test points in all three rows were tested. Tests on each layer of capping were performed using the GDP and the TFT. On the lower capping layers, all test points in Rows A and C (i.e. six tests per bay) were tested after complete compaction of the bay. On the final layer of capping all three rows of test points were tested. Stiffness measurements using the TFT and the GDP on the sub-base were performed on all points in Rows A and C, after two and four passes of the
roller. After eight passes all points in all rows were tested. The FWD was not available
during the initial testing of the trial. Four days after completion of the sub-base a series of
FWD and repeat GDP tests was performed on all points in all bays.

DCP tests were performed on the subgrade adjacent to all points in Rows A and C after
the stiffness measurements had been performed. The DCP was driven approximately 800
mm into the subgrade. The DCP was driven through the compacted capping following its
completion in all points in Rows A and C. For sub-base, one point in each of Rows A and
C in each bay was tested after its completion.

Density measurements were performed using the NDG in direct transmission mode.
Density measurements were performed at all test points in Rows A and C in all bays on
the final layer of capping. The NDG probe penetrations used in each bay were adjusted to
reflect their thickness to remove any effects of the subgrade, (i.e. Bay 2, 200 mm
penetration into 300mm of capping; Bay 3, 150 mm penetration into 200 mm of capping;
Bay 4, 100 mm Penetration into 100 mm capping)).

Density measurements on sub-base were made after two, four and eight passes of the
roller. The extent of the density measurement on sub-base was limited due to time
constraints. The NDG measurements were made with a probe penetration of 150 mm to
match the sub-base layer thickness. Density measurements were made on five points in
each bay spread across Rows A and C.

Stiffness measurements were made at test points in all three rows in all bays with the TFT
and the GDP prior to widening and trafficking. Further DCP measurements were made at
this stage on all points in Row A (the wheel path). Two DCP measurements were taken in
each bay of Row B.

The trial was trafficked with a laden twin axle lorry similar to that described in Section
4.4.1.1 (the lorry details are given in Figure 4.8). Monitoring of the formation of ruts was
undertaken by levelling in the wheel path (due to width constraints and the widening only Row A was trafficked). A full set of levels was taken every 10 passes. The frequency of levelling was then reduced as the rate of rut propagation reduced. When the 40 mm sub-base rut limit (Powell et al 1984) had been exceeded in each bay, the sub-base and capping in the rutted bay were carefully excavated and the transfer of rut through to the subgrade measured (relating the levels back to subgrade levels previously taken).

Fifteen months after construction, additional testing of the trial was undertaken. Only the centre row (Row B) of test points remained undisturbed since Rows A and C had been excavated during the widening and trafficking phase. All three devices were used on the trial, although slightly different methodologies were adopted.

The GDP tests were performed as described in Section 4.4.1.1. The TFT tests performed used a slightly different test regime to that previously applied. Three conditioning drops from the greatest drop height (approximately 100 kPa contact stress) were followed by three measurement drops at each of the lowest, middle and greatest drop heights, giving approximate contact stresses of 33, 66 and 100 kPa. The FWD was used with three conditioning drops at 100 kPa contact stress followed by three measuring drops at each of 100, 200 and 300 kPa contact stresses. A further set of tests at 100 kPa was then made to assess any effects of the higher contact stresses. DCP measurements were performed in each bay of the trial and on the exposed subgrade.

4.4.3 Trial Three (The Mountsorrel Trial)

4.4.3.1 Trial Layout and Construction

The trial site was located at an isolated area of undisturbed ground within the quarry stockpile area at Redland Larfarge’s Mountsorrel Quarry, Leicestershire. The site is crossed by high voltage overhead power lines. The two trials were constructed side by
side at the crest of a slope, allowing sufficient safe working distance from the power lines.

A site investigation incorporating the digging of trial pits and collection of U100 samples for triaxial testing was undertaken to establish the soil conditions of the site. The site was found to be essentially 300 mm of topsoil overlying a stiff red brown silty clay (weathered Mercia Mudstone) containing occasional Granodiorite gravel, the gravel becoming more prevalent with depth. Occasional Granodiorite cobbles were found at 1.5m and below. From the site investigation test data a design thickness of 300 mm was chosen, based on repeated load triaxial test data collected from site investigation samples. Full details of the site investigation and the derivation of the design thickness are included in Appendix B.

To determine the effect of a variation in capping thickness, and to determine the critical design load case, thicker (450 mm) and thinner (150 mm) bays of capping were constructed in addition to the 300 mm thick trial section. The layout of the trial road is shown in Figure 4.9. The subgrade was stepped to give a level completed formation surface for trafficking. The bays were 4.5m long, and 4.0m wide. “Run off” zones were provided at each end of the trials to allow the roller proposed for compaction to completely clear the test bays.

The roller used for compaction of the capping and sub-base was a Benford SP2010 Vibratory Roller (Figure 4.10). The 150 mm layers of both capping and sub-base were compacted with five passes of the roller (MCDHW Vol. 1, Table 6/4 and 8/1 1994). The roller was 2.1m wide, which produced an overlap of 0.2m when two adjacent passes were used to compact the full width of the trial sections.

Three rows of test points (Rows A, B and C) were marked on each section. Row B was positioned on the centreline of the trial sections (in the area of roller overlap), while rows A and C were on the outer wheel paths of the trafficking vehicle. Four test points were
positioned in each bay along each of the three rows (Figure 4.10), i.e. 12 tests per bay. The test points were positioned so that there was no influence from the adjacent bay(s) or test points on the readings.

A 40 mm down screened crusher run Granodiorite, from the Mountsorrel Quarry, was utilised as capping for the first set of trial sections. An Oolitic Limestone 6F2 (Cl. 613, of MCHW, Vol. I) capping from Great Ponton Quarry was utilised for the second set of trial sections (similar to the material used at Trial 1). The Type 1 sub-base (Cl. 803, MCHW Vol. 1 1994) used was a graded Granodiorite from the Mountsorrel Quarry.

Initially the first set of trial sections was constructed to the top of the final layer of capping. The second set of trial sections was then constructed adjacent to the first, similarly to top of capping level. Assessment of the bays took place as the trials were being constructed, the capping was then trafficked and finally sub-base was constructed upon the capping.

Both trials were constructed using a similar methodology to the sequential construction and testing used in Trial Two (Sections 4.4.2.1 and 4.4.2.2). The thickest bay (Bay 1, 450 mm of capping) was excavated to finished level. The exposed formation was then tested. Capping material was laid to a thickness of 175 mm (allowing 25 mm for compaction). The capping was compacted with two passes of the roller, the materials trimmed and a further three passes of the roller made to complete compaction. The subgrade in Bay 2 was then excavated and trimmed to finished level, and the capping in Bay 1 and subgrade in Bay 2 tested.

The next layer of capping was then installed in Bays 1 and 2 (to provide 300 mm thickness of capping in Bay 1 and 150 mm in the Bay 2). The subgrade in Bay 3 was excavated, trimmed, and testing performed on the capping in Bays 1 and 2 and the subgrade in Bay 3. The final layer of capping was then installed and tested at three stages during its compaction (Section 4.4.3.2).
The Granodiorite sub-base was laid upon the capping of both trials and trimmed to level to create a 175 mm thick layer (again allowing 25 mm for compaction). Testing during compaction of sub-base was undertaken after one, two, and five passes of the roller.

4.4.3.2 Assessment of Trial Sections

Bulk samples of the granular materials were taken for characterisation in the laboratory. Six U100 samples and bulk samples of the subgrade were collected, from trial pits that were dug to formation level, adjacent to each trial bay.

Dynamic stiffness measurements were performed using the devices used in the previous trials. The test methodology during construction is as described in Section 4.4.1.2. All the tests have been performed on the same points. The quantity and nature of testing undertaken on each set of trial sections was varied according to the devices available at the time of construction and is described below.

During the construction of the lower layers of the Granodiorite capping only the GDP was available. All points in Rows A and C were tested. The magnetic field generated by the adjacent power lines appeared to affect some of the GDP readings at the thinnest end of the trial sections, and as a result some of these data are missing.

Additional measurements were taken using the FWD on points in Rows A and C after completion of the second layer of capping, prior to the placement of the final layer of capping. Testing with the FWD and GDP was undertaken on the final layer of capping in Rows A and C after one and two passes of the roller, and at all points in all three rows after five passes of the roller. To assess the effects of compaction, a further 10 roller passes were applied. A series of repeat GDP measurements were taken prior to this extra compaction. The capping was tested with the GDP after an additional 6 and 10 passes (giving a total of 11 and 15 passes respectively).
The pattern of dynamic stiffness measurement and frequency of testing performed on the Granodiorite sections was repeated for the Limestone sections. Stiffness measurements were also made with the TFT in all locations where GDP measurements were made. Testing on the final layer of capping was undertaken after one, three and five passes of the roller. Testing was performed after three passes instead of two for operational reasons.

Four DCP tests were performed on the exposed and trimmed subgrade in each bay. Following completion of the construction of the final layer of Granodiorite capping, a further four DCP tests were carried out in each bay through the capping. Because of the particle size of the 6F2 Limestone capping used in the second set of trial sections, no DCP tests could be made.

NDG measurements were made on the top level of capping using the direct transmission method. After one and two passes of the roller on the Granodiorite capping (or one and three passes on Limestone capping) density measurements were made at five points in each bay (two in each of Rows A and C, and one in Row B). After completion of compaction of the final layer of capping, density measurements were taken at all points in Rows A, B and C.

Following completion of the capping, the trials were trafficked with a laden twin axle lorry similar to that described in Section 4.4.1.1, and the formation of ruts monitored. The lorry details are given in Figure 4.11. Following trafficking, the Granodiorite capping was re-profiled to fill in the ruts formed, and re-compacted with a further five passes of the roller.

The Limestone capping was affected by water ingress due to a period of wet weather between completion of construction and trafficking, with water being absorbed by the fines in the material. The Limestone, which when originally constructed formed a very stiff and competent capping, turned into a saturated, spongy material having a low
strength and stiffness. When trafficked, the material immediately developed significant rutting and trafficking was therefore stopped after two lorry passes. As a consequence the upper 150 mm of the Limestone was removed, and fresh material was laid and compacted.

A limited programme of testing was undertaken to assess the deterioration in the properties of the Limestone trial sections due to the ingress of water. Four repeat GDP measurements were taken in each bay and two DCP tests were performed. Samples of the wet capping materials were recovered for assessment of water content in the laboratory.

Further stiffness measurements were undertaken on the final layer of capping using the GDP and the TFT immediately prior to constructing the sub-base. A similar programme of assessment of the sub-base was undertaken for both trial sections. During compaction of the sub-base the stiffness was measured at the test points in Rows A and C (after one and two roller passes) using the GDP, the TFT and the FWD. After five compaction passes all the test points in all three rows were tested with all three devices.

NDG measurements were taken on the sub-base after one and two passes of the roller. Density measurements were taken at five points in each bay (two each in Rows A and C). After completion of the final (i.e. fifth) roller pass, density measurements were taken at all points in Rows A, B and C.

Five months after completion of sub-base construction repeat stiffness measurements were made in all points in Rows A and C of the two sets of trial sections. Similar methodologies to those described in Section 4.4.2.2 were used.

4.5 Error Assessment of Dynamic Stiffness Measuring Devices

A series of tests on rubber mats of three different thicknesses positioned on a cast concrete floor slab in the laboratory were carried out with the TFT and GDP. The rubber
mats were made from 22mm thick rubber sheet (similar to that used in the dummy triaxial samples, Section 4.3.4.2). Three mats, made from one thickness, two and three thicknesses respectively, of rubber glued together, were tested. The temperature of the laboratory was monitored during the testing to determine whether this influenced the results. Thirty three tests with the GDP were carried out on each rubber mat using the device in accordance with the manufacturer’s instructions (three drops to each test), giving a total of 99 test drops on each mat. The TFT was tested using three drop heights (giving contact stresses of approximately 40, 70 and 105 kPa depending on the thickness of the mat tested). The tests were carried out in 10 batches. Each batch consisted of 10 drops from each of three drop heights (i.e. 30 drops in each batch), giving a total of 100 drops on each mat from each drop height. Similar testing with the FWD was performed with the mats, positioned outside on an area of industrial block paving. Three drop heights were used to produce target contact stresses of approximately 100, 150 and 200 kPa. Thirty drops at each stress on each mat were used.

4.6 Analysis of Data

The data collected at various stages of loading during repeated load triaxial testing were analysed to assess the effects of the applied stress on sample response during cycling to assess the variability and precision of the data collected from the strain measuring transducers.

To allow the assessment of the influence of the subgrade, subgrade stress dependency data have been compared to the field measured subgrade stiffness data. Comparable subgrade stress-dependency was then used to back-analyse the capping stiffness. These analyses were performed using the Elsym 5 linear elastic analysis programme (Section 2.11).

Each analysis was performed by fitting curves to the subgrade stress-dependency (from either repeated load triaxial tests or in-situ measurements) and the composite field
measured stress-dependency data. The equations of the curve for the composite were then used to predict values of stiffness at various applied stresses, and the deflection of the composite back-calculated using Boussinesq analysis. The value of individual capping and subgrade stiffness were then estimated, and the applied stress to subgrade calculated in the program. This was then used to calculate subgrade stiffness using its stress-dependency equation. The composite deflection was then recalculated in the program and compared to that initially calculated from the composite stress-dependency equation. The process can then be repeated around an iterative loop until the deflections match and hence the capping stiffness resolved. By finding the separate stress-dependency of the materials, the influence of the subgrade on the composite performance of the trials under various loads was assessed.
Table 4.1. Loading Regimes for Repeated Load Triaxial Tests

<table>
<thead>
<tr>
<th>Load (kPa)</th>
<th>No. of Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>1000</td>
</tr>
<tr>
<td>20</td>
<td>1000</td>
</tr>
<tr>
<td>30</td>
<td>1000</td>
</tr>
<tr>
<td>100</td>
<td>1000</td>
</tr>
<tr>
<td>120</td>
<td>1000</td>
</tr>
<tr>
<td></td>
<td>Continuing at 20 kPa increments</td>
</tr>
</tbody>
</table>

1. Standard Tests

2. Revised Standard Test

3. Stress Controlled Conditioning

4. Strain Controlled Conditioning

Note: Tests stopped when sample has sustained 5% permanent strain
Figure 4.1. Repeated Load Triaxial Apparatus. (Top, Sample Under Test), (Bottom, On-Sample Instrumentation)
Figure 4.3. Benford TV800 Roller used to Compact Trial One

Figure 4.4. Photograph of a Standard Two Axle Lorry Used for Trafficking the Trial Sections
Weight:-

Gross Weight 15540 kg
Front Axle 5900 kg (two tyres 2950 kg each)
Rear Axle 9640 kg (four tyres 2410 kg each)

Lorry Dimensions:-

Wheel Base 3.7m
Width Overall front 2.3m
rear 2.34m
Tyre Width 0.22m

Tyre Pressures (kPa):-

Front Left Front Right
(564) (564)

Rear Left Outer Rear Right Outer
(536) (536) (536) (536)

Tyre Foot Prints:-

Left Front 180mm x 285mm (51300mm²)
Left Rear Outer 180mm x 245mm (44100mm²)
Left Rear Inner 180mm x 245mm (44100mm²)

Figure 4.5. Details of Lorry Used for Trafficking Trial One (Hathern)
(Mass = 3,565 kg, Width = 1.3m)

Figure 4.7. Benford 1300HV Roller used to Compact Trial Two
Weight:-

Gross Weight 15720 kg
Front Axle 5300 kg (two tyres 2650 kg each)
Rear Axle 10420 kg (four tyres 2605 kg each)

Lorry Dimensions:-

Wheel Base 3.7m
Width Overall front 2.3m rear 2.34m
Tyre Width 0.22m

Tyre Pressures (kPa):-

Front Left Front Right
(506) (506)
Rear Left Outer Rear Right Outer
(580) (580) (580) (580)

Tyre Foot Prints:-

Left Front 180mm x 285mm (51300mm²)
Left Rear Outer 180mm x 245mm (44100mm²)
Left Rear Inner 180mm x 245mm (44100mm²)

Figure 4.8. Details of Lorry Used for Trafficking Trial Two (Bardon)
Figure 4.10. Benford SP2010 Roller used to Compact Trial Three

(Mass = 10,070 kg, Width = 2.1m)
**Weight:-**

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gross Weight</td>
<td>15880 kg</td>
</tr>
<tr>
<td>Front Axle</td>
<td>4480 kg</td>
</tr>
<tr>
<td>Rear Axle</td>
<td>11400 kg</td>
</tr>
</tbody>
</table>

*(two tyres 2240 kg each)*

*(four tyres 2850 kg each)*

**Lorry Dimensions:-**

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wheel Base</td>
<td>3.7m</td>
</tr>
<tr>
<td>Width Overall</td>
<td></td>
</tr>
<tr>
<td>front</td>
<td>2.3m</td>
</tr>
<tr>
<td>rear</td>
<td>2.34m</td>
</tr>
<tr>
<td>Tyre Width</td>
<td>0.22m</td>
</tr>
</tbody>
</table>

**Tyre Pressures (kPa):-**

<table>
<thead>
<tr>
<th></th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front Left</td>
<td>(428)</td>
</tr>
<tr>
<td>Front Right</td>
<td>(428)</td>
</tr>
<tr>
<td>Rear Left Outer</td>
<td>(634)</td>
</tr>
<tr>
<td>Rear Right Outer</td>
<td>(634)</td>
</tr>
</tbody>
</table>

**Tyre Foot Prints:-**

<table>
<thead>
<tr>
<th></th>
<th>Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left Front</td>
<td>180mm x 285mm (51300mm²)</td>
</tr>
<tr>
<td>Left Rear Outer</td>
<td>180mm x 245mm (44100mm²)</td>
</tr>
<tr>
<td>Left Rear Inner</td>
<td>180mm x 245mm (44100mm²)</td>
</tr>
</tbody>
</table>

Figure 4.11. Details of Lorry Used for Trafficking Trial Three (Mountsorrel)
5.0 PRESENTATION OF RESULTS

5.1 Introduction

The chapter is divided into three sections. Firstly it presents the data collected from the laboratory classification tests describing the basic characteristics of the materials tested in the research. Secondly the laboratory based repeated load triaxial testing is presented. The results from the error assessment tests performed on the dummy triaxial samples are presented as well as an analysis of the precision of the data acquisition system. Typical data are then presented from tests undertaken using different test methodologies. Large amounts of data were collected during the course of the testing, and as a consequence only typical figures are presented within the chapter. The key parameters from all the tests are summarised in tables to allow comparison between the different materials tested and are presented in Appendix C.

Finally the field trial data are presented, starting with the results of the repeatability tests, conducted with the three dynamic stiffness measuring devices on rubber sheets. Each field trial is thereafter presented separately and in the order in which they were constructed. For each trial the stiffness measurements taken during the course of the work are presented, followed by the density measurements made on the upper layers of the trials, a description of the trafficking undertaken and a review of the stress-dependency of the materials tested. Again the data are summarised in the figures and presented in detail in tables in Appendix C.

5.2 Classification Testing

This section initially describes the classification data from the laboratory tests on the imported granular materials used to construct the field trials, and then details the test data from the classification tests performed on the subgrade samples collected.

5.2.1 Aggregate Tests

5.2.1.1 Grading

The grading curves for the eight aggregates tested in the controlled field trials are presented in Figures 5.1 to 5.3. These show the test results in relation to the specified
grading limits for each particular material type. (i.e.: Fine 6F1, and Coarse 6F2 Capping, from Table 6.4 MCHW 1994, and Type 1 Sub-base, from Table 8.1, MCHW 1994).

The grading curve for the Porphyritic Andesite and Oolitic Limestone 6F2 cappings from Trial 1 (the Hathem trial) and the Limestone 6F2 capping from Trial 2 (the Mountsorrel trial) are presented in Figure 5.1. Both of the cappings from the Hathem trial lie within the middle of the permitted grading envelope for 6F2 material, and have broadly similarly shaped grading curves, with the Porphyritic Andesite being more well graded.

The grading curve for the Oolitic Limestone capping from the Mountsorrel trial lies within the grading envelope at the coarse and finer end of the permissible 6F2 grading envelope, yet there was insufficient material in the size range between 7 and 37.5 mm to comply with the 6F2 specification. This material was from the same quarry as the Limestone used in the Hathem trial.

The grading curves for the finer graded quarry waste materials and 6F1 cappings used in the trials are shown in Figure 5.2. The grading curve of the Porphyritic Andesite Quarry Waste, used at Hathem (Trial 1) showed the material to contain a low fines fraction, and consequently lay marginally outside the grading envelope for 6F1 capping at the lower end of the curve. At the coarse end of the curve the material lay slightly above the 6F2 grading envelope.

The Porphyritic Andesite Quarry Waste capping used at Bardon (Trial 2) was a similar material to that used at Hathem and was from the same supplier. It had a very similar grading, being slightly outside the grading limits for both 6F1 and 6F2 capping.

The grading curve for the Granodiorite capping used at Mountsorrel (Trial 3) indicated the material to be uniformly graded (in the range 6-50mm) with minimal fines content, with only 15% of the material less than 10 mm in size. The material lay outside the grading envelopes for both 6F1 and 6F2 capping. However, it was closest to a 6F2 grading.

The sub-base grading curves are presented in Figure 5.3. The material grading from Bardon (Trial 1) shows the Porphyritic Andesite sub-base was on or marginally outside the coarse limit of the Type 1 grading curve. At the lower end of the curve the material lies below the grading limit, i.e. it contained insufficient fines to comply with the
specification. The Granodiorite sub-base from Mountsorrel (Trial 3) lay within the grading envelope, with the grading curve following the coarse grading limit below 20mm particle size.

5.2.1.2 Peak Shear Stress Ratio Tests (PSSR)

All of the granular materials used in the controlled field trials were tested in a 300mm shear box to establish their PSSR. The results for each of the materials are presented in Table C5.1, together with the percentage of material removed to comply with the test method. Materials with a PSSR of less than 1.9 are not considered suitable for direct trafficking, while those with a PSSR of between 1.9 and 2.8 may be suitable for trafficking and materials with a PSSR of 2.8 and above should perform adequately under trafficking (Earland and Pike, 1984). The test should only be performed on materials containing particles less than 37.5mm in size, and no more than 15% of the material should be removed to comply with the test method.

Both the Porphyritic Andesite cappings used at Hathem (Trial 1) had PSSR values above 2.8 and therefore would have been expected to have sufficient internal stability to be able to sustain trafficking. The Limestone 6F2 capping used at Hathem had a PSSR in the intermediate classification (2.6), indicating that the material should be trafficked to assess its likely performance.

Both materials used at Bardon, (Trial 2) had PSSR values above 2.8, implying they should perform adequately under trafficking.

The three granular materials used at Mountsorrel (Trial 3) all had PSSR values greater than 2.8. These materials had the highest values of all the PSSR tests performed, with the sub-base having the highest value of all.

5.2.1.3 Compaction Tests

The optimum water content and maximum compacted density data from the compaction tests performed upon the granular materials from the field trials are summarised in Table C5.1, and detailed in Figures 5.4 to 5.6. The curves of best fit (fitted using least squares linear regression) had relatively low correlation coefficients ($R^2$, an $R^2$ value of ±1 is a perfect correlation), making it difficult to predict precise values of maximum density and
optimum water content. It was noted that similar materials, such as the 6F1 cappings and the sub-base used in trials from the same source, possessed similar gradings, and yet the compaction data were quite different.

The coarse materials (Figure 5.4) all showed significant scatter. For example, when comparing the data from Oolitic Limestone from the Hathem and Mountsorrel trials, it can be seen that the materials had laboratory maximum densities of 1.829 Mg/m³ at 2.2% and 1.896 Mg/m³ at 7.0% respectively. However the quantity of material removed must be considered to have an affect.

5.2.2 Subgrade Tests

5.2.2.1 Index Tests

The results from the subgrade classification tests are presented in Table C5.2. The subgrade at the A1M site was a stiff brown clay of intermediate plasticity with significant variation in water content. London Clay from Iver was a firm grey/brown clay of intermediate plasticity. The Oxford Clay from Peterborough was a very stiff dark grey clay of high plasticity, all samples having similar water contents and bulk densities.

The subgrade at the Hathem Site (Trial 1) was a light brown silty sandy clay of low plasticity. The plasticity, bulk density and water content of the subgrade samples showed some variability along the site.

The subgrade at Bardon (Trial 2) was a stiff dark red/brown silty clay (weathered Mercia Mudstone) of low plasticity, becoming more plastic (less silty) with increasing bay number (i.e. along the site).

The subgrade at Mountsorrel (Trial 3) was a firm red/brown silty clay (weathered Mercia Mudstone) of intermediate plasticity. The plasticity was observed to be consistent along the site, although some variability in the bulk density and water content was observed.

5.2.2.2 CBR Tests

Data collected from the limited CBR testing on samples of subgrade collected from the field trial sites are presented in Table C5.2. The samples from the Hathem site (Trial 1)
produced significantly different CBR values (which do not correlate with water content). The Bardon samples (Trial 2) showed a consistently low CBR value (approximately 1 to 2 %) with only a slight reduction in CBR in soaking tests.

The Mountsorrel samples (Trial 3) again showed significant variability in CBR between tests with values in the range 5.0 to 9.5%. Although there was no pattern of changing CBR with water content or density, generally the higher the saturation of the sample the lower the CBR.

5.3 Triaxial Test Results

5.3.1 Introduction

Samples from two types of site have been tested in the repeated load triaxial test: those from controlled field trials (Hathem and Mountsorrel), and samples of different types of materials collected from various sources. The latter materials were assessed to provide a range of materials with which to evaluate the apparatus. They included London and Oxford Clay (U100 samples collected from brick pits), a silty Mercia Mudstone collected from the overburden at Bardon Hill (not from the field trial site) and samples collected from the subgrade of the A1M Motorway widening scheme (Oxford Clay). In addition, tests were performed on rubber dummy triaxial samples to assess the precision of the apparatus.

In excess of 60 triaxial tests have been performed using different specimen preparations and test methodologies. To simplify presentation only significant and typical patterns seen from each set of tests are presented and discussed herein. Tabulated data summarising the key elements from the tests performed on each specimen are presented in Appendix C. Typically the data presented (stiffness or strains) are calculated from the average readings from the two on-sample strain gauges.

The typical behaviour of the materials during the phases of cycling during a triaxial test is presented first, the results from the tests on the rubber dummy triaxial samples, together with an indication of the variability in the measurements. Thereafter the behaviour from tests using undisturbed, remoulded and conditioned samples are presented.
For each triaxial test two sets of curves are plotted, resilient modulus (stiffness) against repeated deviator stress, and permanent strain against total deviator stress (seating stress of 5kPa plus repeated stress 4.3.3). The data for tests on samples from the same site using similar test regimes are typically shown on the same figure.

5.3.2 General Observations

5.3.2.1 Typical Sample Responses

Figures 5.7 and 5.8 show typical resilient and permanent deformation behaviour of undisturbed triaxial samples during the course of each period of 1000 cycles of incrementally increasing deviator stress. Figure 5.7 shows that there was little change in the measured resilient deformation during a period of 1000 cycles at lower deviator stress and small associated sample strains. As greater deviator stresses were applied the magnitude of the resilient strain reduced slightly during each period of cycling, this effect being greater at higher deviator stresses and once the permanent strain had started to increase significantly. The LVDT, which records the complete sample response, showed greater variability in resilient strain reading than the on-sample gauges, and the variability increased at higher stresses. At each step in deviator stress there was a large jump in the resilient strain.

Figure 5.8 shows the development of permanent strain during the course of a test. This shows that at low deviator stress limited permanent strain was caused and the majority of the strain at each stress level was generated during the initial cycles. As the deviator stress increased however, the amount of deformation sustained within a period of cycling increased until the deformation rate over the first 400 cycles became almost uniform. Nevertheless, much more deformation was sustained during the first 100 cycles than in the last 100 cycles. Generally the permanent deformation sustained during a period of cycling reached an asymptote prior to the 1000th cycle of load at strain levels below 5 % (which was the maximum strain sustainable by the on-sample measuring transducers). The on-sample gauges should provide more reliable data as they measure across the middle third of the sample and hence should remove end effects from the testing.
5.3.2.2 On-Sample Strain Gauge Variability

Figure 5.9 shows the stiffness readings from the two on-sample strain gauges and the whole sample LVDT for data collected for three samples. As this example shows, there was commonly a large difference between the stiffnesses measured on either side of a sample. In addition a large difference was found between the on-sample gauges and the LVDT at low deviator stress levels. Initially the LVDT readings produced much lower values of stiffness than the on-sample gauges, whereas as the applied stress increased the stiffness values tended to converge for all three transducers. Changes in the value of stiffness at low deviator stress were commonly found to be variable. For example Figure 5.9 shows a case in which one on-sample gauge indicated a steady reduction in stiffness with applied stress, while the diametrically opposite gauge indicated a more erratic response. However, at higher values of deviator stress the initial variability reduces and data from the gauges converge to give clear patterns of behaviour, providing useful data for comparison to field measured parameters.

5.3.2.3 Precision

Appendix A describes the repeated load triaxial test apparatus and data collection system. Table C5.3 shows the possible percentage error for any given value of stiffness measured at a particular level of deviator stress for both the LVDT and the on-sample strain gauges, based upon the precision of the apparatus analogue to digital converter given the calibration range of the devices.

This shows that at low deviator stress where high stiffness is measured, the percentage error of the data collected could have been in excess of 100%, and this potential source of error must be considered for data collected at low deviator stress. However the precision improved dramatically with increased applied stress for any given resilient deformation or for a lower measured stiffness for a given deviator stress. Therefore, for the majority of stiffness measurements made during this research the data can be seen to have acceptable accuracy across an acceptably wide range of deviator stress. The precision of permanent deformation data was not a concern as the deformations measured are much greater.
5.3.2.4 Dummy Samples

The stiffness data collected from the three tests on the two rubber samples are presented in Figure 5.10. The one sheet dummy sample had a stiffness of 60 MPa and the two sheet sample 18 MPa. All three tests for each samples converged to similar values. The stiffer (one sheet) dummy sample showed greater variability at the lower range of deviator stresses and only reached a consistent value after an applied deviator stress of approximately 50kPa. The two sheet sample reaches a consistent value at 30kPa. The two sheet sample showed a very small yet progressive reduction in stiffness with increasing deviator stress.

5.3.3 Undisturbed Samples

Table C5.4 shows the key data from the repeated load triaxial tests on undisturbed samples taken from the U100 sample tubes collected at the various sites. Curves of permanent strain against deviator stress and stiffness against deviator stress for the Hathem (Trial 1) site are presented in Figures 5.11 and 5.12. Figure 5.11 shows relatively high variability between samples. All the curves show an exponential increase in permanent deformation with increasing deviator stress. Initially a low level of strain was induced which then increased rapidly beyond a certain point (threshold). Typically the stiffer a sample the higher the deviator stress required to cause a similar value of permanent deformation (Figure 5.12), as indicted by the values of threshold stress compared to the stiffness values given in Table C5.4.

Similar variability between samples was seen from the curve of stiffness against repeated deviator stress (Figure 5.12). The data showed a similar pattern of higher stiffness at a low level of repeated deviator stress, whereafter the stiffness reduced with increasing deviator stress. The magnitude of the stiffness at each deviator stress was different for each sample, and there was no consistency between samples. This can be observed from the stiffness values measured at deviator stresses of 10 and 100 kPa included in Table C5.4. However the curves do have similar shapes.

At low deviator stress data tended to show some deviation about the general trend of the stiffness reading. This was attributed to variations in the readings between the two on-sample strain gauges. Table C5.4 gives the range of stiffness values measured by the on-sample gauges at 10 kPa, and shows that this range can be in excess of 100% of the
average stiffness. In certain cases, only the readings from one on-sample strain gauge could be used due to poor connection of the gauges. The discrepancy was larger for samples of higher stiffness and at lower deviator stress. This variability was common across most of the samples tested regardless of which sites they were from. This variability was not so significant for measurements of permanent strain. Large changes in stiffness were commonly seen at deviator stresses less than \(20\) kPa, implying a sensitivity of stiffness calculation to small changes in strain at low deviator stresses.

The stiffness data from samples tended to a limiting value of stiffness at increasing deviator stress (referred to as a stiffness asymptote). Samples from the same site frequently tended to similar values for their stiffness asymptote. The asymptote was approached at a deviator stress where the onset of permanent deformation becomes unstable (i.e. at or beyond threshold).

From Table C5.4 the variability in the materials from the same site can be seen. The stiffness values and "threshold stress" values were quite different for samples that have similar water contents, strength and bulk density. For example, samples from the Hathern site (all taken from an area \(40\)m by \(6\)m), had shear strengths in the range \(40\) to \(75\) kPa, threshold stress values in the range \(17\) to \(56\) kPa, and water contents \(15\) to \(21\)%.

5.3.4 Remoulded Samples

Data from the tests on remoulded samples are presented in Table C5.5 and typical data from these tests are presented in Figure 5.13 and 5.14. Similar patterns of permanent strain and stiffness to those of the undisturbed samples were seen. The magnitude of the permanent strain and stiffness at each level of deviator stress again showed the variability between samples from the same site. The samples from the same site shown on the figures had threshold values in the range \(53\) to \(138\)kPa and stiffnesses at \(10\)kPa in the range \(91\) to \(145\)MPa. The stiffness data from samples from the same site appeared to be more convergent to a similar value of stiffness asymptote than the undisturbed samples. The variability in readings of both permanent strain and stiffness are again apparent at low deviator stress.

Comparing between Tables C5.4 and C5.5 the stiffnesses at specific deviator stress values were higher for the remoulded samples than for the tests on the same samples in their undisturbed state. Typical data are plotted on Figures 5.15 and 5.16, in general, an
improvement in the consistency of behaviour of the samples with remoulding was found relative to their performance in the undisturbed state. This appeared to be largest for the Hathem samples. Unsurprisingly changes to the density of the samples were found following re-compaction from those values seen in the undisturbed samples.

5.3.5 Equilibrium Remoulded Samples

Table C5.6 shows the sample data collected from the limited testing undertaken on samples remoulded at equilibrium water content. Figures 5.15 and 5.16, again indicate the pattern of reducing stiffness with increasing deviator stress. The increase in water content of the samples to their predicted in-situ equilibrium values was typically 3%. Yet this had resulted in a 50% reduction in stiffness and a significant reduction in the amount of stress required to cause a similar amount of permanent deformation relative to the remoulded in-situ water content samples.

Comparing the equilibrium results to the undisturbed tests, only marginally lower stiffnesses were observed and in some cases better performance in terms of both stiffness and resistance to permanent deformation was found with the equilibrium samples. They showed a much more rapid increase in permanent deformation at lower deviator stress than either the remoulded or undisturbed in-situ water content samples. The range and variability of stiffnesses measured by the on-sample gauges at low deviator stress on these softer samples was lower compared to the data from undisturbed and remoulded samples.

5.3.6 Conditioned Tests

Typical results from the conditioned tests are presented in Figures 5.17 and 5.18.

5.3.6.1 Undisturbed Conditioned Samples

The results of tests on undisturbed samples after they have received conditioning are presented in Table C5.7. The stiffness results showed a similar pattern of behaviour to the unconditioned undisturbed samples. The variability at low applied deviator stress as evidenced by the range at 10kPa, did not appear to be rectified by the small element of conditioning utilised (Figure 5.17).
The relationship between permanent strain and deviator stress (Figure 5.18) showed a reduction in the generation of permanent strain over the range of the applied conditioning stresses during the main part of the test after conditioning had been applied.

5.3.6.2 Remoulded Conditioned Samples

The data from the tests on the remoulded conditioned samples are presented in Table C5.8. The stiffness data follow a similar pattern to the other tests. Significant variability of the results at low deviator stress was still observed, despite the conditioning. The effects of conditioning again did not appear to have reduced the cross-sample variability between the readings from the two on-sample strain gauges, and the variability between samples from the same site was still apparent.

The permanent deformation behaviour (Figure 5.18) clearly shows the effect of the condition on the permanent deformation response post conditioning at stresses below those applied during conditioning, whereby the samples sustained limited permanent deformation until the sample was loaded above the stress level previously applied. The stiffness increased slightly after conditioning.

5.3.7 Repeatability Tests

The data from the repeatability tests on remoulded samples of Mountsorrel subgrade are presented in Table C5.9. Typical curves from one series of repeatability tests are presented in Figures 5.19 and 5.20. The stiffness results obtained were similar at higher deviator stress, but were considerably less so at low deviator stress (less than 40 kPa). The stiffest two sets of samples showed reasonable repeatability in stiffness measurement, while the second set of samples (water content, PL +2%) showed the most comparable behaviour between tests (as evidenced by the threshold stress and stiffness values given in Table C5.9). The wetter samples (water content, PL +4%) showed very poor repeatability. Each set of samples converged upon a stiffness asymptote, the value of which reduced with increasing water content. The deviator stress at which this asymptote was reached varies from sample to sample within each group, with stiffer samples reaching the asymptote at higher deviator stress.

The variability between the permanent deformation for different sets of samples increased with higher deviator stress. This was greater at higher values of permanent strain. Some
variability is seen at low deviator stress, with one sample showing negative permanent strain over the low strain range. The samples that show higher re-compacted density tend to show slightly better performance.

5.4 Dynamic Plate Device Repeatability Tests

Figure 5.21 and Table C5.10 summarise all the data from all three devices for the tests performed upon the rubber sheets. The GDP results (all at an applied stress of 100kPa) showed limited scatter for the two and three sheet tests. The three sheet test gave similar values to the TFT (10MPa). The two sheet test stiffnesses are 30% lower than those for TFT (GDP 22MPa, 32MPa TFT). For the one sheet test the GDP measured a much lower stiffness (43 MPa) than the TFT. The GDP showed a scatter of 10% (standard deviation expressed as a percentage of the mean) on the thinner, one and two thickness sheets, and 25% on the thicker sheet, showing the GDP to produce repeatable results.

The TFT tests on three sheets showed a very slight increase in stiffness within increasing applied stress (2 MPa), but within the error band shown on the figure (approximately 10%). The two sheet test showed a slight increase in stiffness between the lowest two applied stresses (2 MPa), followed by no change to the highest applied stresses. The standard deviation of measured stiffness was 10 to 20% of the mean stiffness, but there was an increase in standard deviation with increasing applied stress. The one sheet tests showed significant stress-dependency, with a large increase in the measured stiffness with increasing applied stress, and a large increase in standard deviation (20 to 30%), more so than the tests on the other sheets. Figure 5.22 presents the data from the TFT tests on two sheets of rubber, these results were typical of those collected and indicate the scatter in the data was for both the applied stress (due to drop height) and the calculated stiffness. This scatter was thought to be partially attributable to the device bouncing slightly during the testing, and potentially due to the influence of the floor beneath, especially for the thinnest sheet. However, for the one and two sheet tests the TFT was shown to produce relatively repeatable results.

The FWD was used at higher stresses than the other two devices and there was some apparent discrepancy in the results. The FWD measured the stiffness of all the sheets to be in the range 80 to 100MPa at the lowest applied stress, which is four times the stiffness measured by the TFT on two sheets. The single sheet test showed a similar value of stiffness as the TFT. The FWD showed the two sheets to be less stiff than the three sheets
at the lower two applied stresses and stiffer than one sheet at the highest applied stress. The standard deviation for the data was low, typically 1 to 2% of the mean, hence being highly reproducible.

5.5 Trial One (The Hathern Field Trial)

5.5.1 Subgrade Stiffness Measurements

The data have been grouped to give average stiffness measurements in each bay for both single tests at adjacent points and three tests at a point for all three dynamic stiffness devices used. These data have then been plotted against bay numbers to show the variation in subgrade stiffness along the site (Figure 5.23). The data are summarised in Table C5.11, which shows the differences in the subgrade stiffness along the length of the trial.

The GDP showed relatively little variability (range 6 to 12 MPa) in its readings along the length of the trial, with little scatter as observed from the standard deviations (in the range 10 to 30% of the mean) compared to the other devices. A marginal reduction in stiffness from Bay 1 to 3 was indicated. The subgrade stiffness in Bay 7 was noticeably lower than the values observed in Bays 11 and 12. The TFT shows a larger reduction in stiffness from Bay 1 to Bay 3, and indicated higher stiffness in Bays 5 and 6, and again in Bays 10 to 12, where subgrade stiffnesses in the range, 10 to 30 MPa were measured. The FWD indicated a higher stiffness in Bays 1 to 3, a lower stiffness in Bays 5 and 6 with steady improvement to Bay 10, and a high stiffness in Bays 11 and 12, which is more comparable to the trends seen with the TFT. The FWD measured a range of stiffness of 10 to 41 MPa. The values of stiffness measured with the TFT and FWD were generally higher than with the GDP (typically by 50%) although some values are much higher. The degree of scatter was significantly greater for the FWD and TFT than for the GDP, with the standard deviation being up to 50% of the mean value.

Comparing data for the GDP and FWD for the groups of single and three tests at a point shows that they are very similar, when considering the variability of the data collected within each bay. Therefore there were no significant difference between single and multiple tests at a point on subgrade when the standard deviation of the data was considered.
The limited set of repeat subgrade stiffness measurements taken during the warm weather (Table C5.11) to assess the change in subgrade stiffness due to drying, showed an approximate doubling in the subgrade stiffness measured with the GDP, when compared to the initial readings taken after initial excavation.

5.5.2 Subgrade Dynamic Cone Penetrometer Testing

The results of the DCP tests (Figure 5.24) have been averaged throughout their depth for each of the three tests in each bay and the data combined to give the average values of penetration (Table C5.12). There was a substantial variation in DCP penetration along the length of the site. The first three bays showed relatively low penetration rates, (i.e. highest strength), with the trend for Bays 1 to 4 being a consistent increase. The central bays (Bays 4 to 10) showed broadly similar values (although with a soft spot in Bay 8), and there was then a reduction in penetration rate through Bays 11 and 12. The variability in DCP penetrations in each bay were themselves relatively large, with variability between the three tests being between 10% and 50%, with a typical variability of approximately 25% of the mean.

5.5.3 Capping Stiffness Measurements

Capping stiffness measurements are presented in Table C5.13, and Figures 5.25 to 5.27. The GDP indicated a small and progressive improvement in material stiffness with increasing compaction for all three materials. The FWD showed a progressive increase in stiffness with increasing compaction for the 6F2 Porphyritic Andesite (Figure 5.25). A similar increase was shown between 4 and 8 passes on the Porphyritic Andesite 40mm Crusher Run (Figure 5.27), although a slight reduction in stiffness after 16 passes then occurred. The FWD showed very little variation in stiffness with compaction on the 6F2 Limestone (Figure 5.26), with its maximum value (68MPa) being reached in the bay that received 4 passes. The TFT showed a similar pattern to the FWD on the 6F2 Limestone, and 6F2 Porphyritic Andesite (maximum stiffness after 4 passes). It showed a progressive increase in stiffness with number of compaction passes on the Porphyritic Andesite 40 mm Crusher Run.

Considering stiffness values between material types, the 40 mm Crusher Run had a lower composite stiffness than the other two materials when tested with the GDP and the FWD, but higher values after more than 4 passes of the roller when measured with the TFT. The
two 6F2 capping materials showed similar stiffnesses in the bays that received 8 and 16 passes of the roller for single measurements with any device.

Considering data for single tests at adjacent points and three tests at the same point for the GDP and FWD, the GDP consistently measured slightly higher stiffnesses at the multiple test points than at the single test points for all three materials. This increase was generally quite small (typically less than 5 MPa), although a larger increase was measured on the Limestone. The FWD did not show any significant trend of change in stiffness between the test points. As with the subgrade the scatter in the readings showed no significant difference between single and multiple tests at a point, except for the GDP on Limestone after 16 passes. The GDP measured values of stiffness that were approximately 30 to 50% of those with the other devices. Again, significant scatter in data was observed. The scatter about the mean of the GDP data became more regular at about 10%, whilst the other devices showed scatter in the approximate range 20 to 50% of the mean.

Figures 5.28 to 5.30 show the stiffness measurements taken at the end of construction, after trafficking, and eight months after completion of the trial. All the stiffness data collected within each bay have been combined (both single and multiple test point data) within each bay to give the averages and standard deviations presented in Table C5.14.

The GDP measured an improvement in stiffness for the 6F2 Porphyritic Andesite (Figure 5.28), with addition of capping of approximately 20 to 30 MPa, followed by a further increase after trafficking of approximately 10 MPa, and a further small increase 8 months after completion. The TFT showed a considerable increase over sub grade stiffness with capping of approximately 40 MPa followed by a further increase of 40 MPa to approximately 100 MPa from the end of construction to 8 months after completion.

The 6F2 Limestone (Figure 5.29) showed an improvement in stiffness over subgrade, when measured with the GDP of 20 to 30 MPa with addition of capping. This was followed by a further small increase between the time of construction and after trafficking, giving a similar stiffness for all bays of approximately 50 MPa (similar to the 40mm crusher run). This improvement was greater in bays that received less compaction. Comparing the readings taken 8 months after construction with those taken before trafficking revealed a small and consistent improvement in stiffness along the length of the trial of approximately 5MPa. The TFT data showed a considerable and consistent improvement in limestone stiffness along the trial (of 50 MPa with capping construction.
and in excess of 60 MPa between the time of construction and 8 months later), approximately a doubling in the measured stiffness to 120 MPa. This was a much larger increase up to 8 months than the other two materials.

The Porphyritic Andesite 40 mm Crusher Run (Figure 5.30) measured with the GDP showed an improvement in stiffness immediately after trafficking, which was largest in the bay receiving only 4 roller passes (25 MPa) (Bay 12), and reduced in the bays that received more compaction. A further increase of approximately 10 MPa over the initial readings was seen after eight months. The TFT showed an approximate doubling in stiffness from the time of construction to eight months for the bay that received least compaction, and a small reduction in stiffness in the 16 pass bay.

Figure 5.31 shows the change in stiffness for all tests combined for the change in capping thickness through the wedge constructed using 6F2 Porphyritic Andesite. The GDP readings indicated no change in stiffness with thickness over the length of the wedge. The TFT showed a marginal increase in stiffness between Bays 1 and 2, and a larger increase in stiffness in the thicker portion (Bay 3). The FWD indicated a similar pattern in Bays 1 and 2, but with higher absolute values, followed by a considerable reduction (converse to the TFT) in Bay 3, the thickest bay.

The scatter of the averaged data (standard deviation as a percentage of the mean) for all devices in all bays showed a substantial increase, from approximately 10 to 20% at the time of construction to in excess of 50% eight months after trafficking and in the long-term.

5.5.4 Capping Density Measurements

Figure 5.32 and Table C5.15. shows the change in NDG measured capping dry density increases with increasing compaction. There was a limited change between the densities measured in the bays that received 4 and 8 passes of the roller (with the limestone showing a slight reduction), followed by a large increase for all materials in bays that received between 8 and 16 passes. The 40 mm Crusher Run Porphyritic Andesite gave a higher density than either of the 6F2 cappings. Both 6F2 cappings had been compacted to a density of 95% of their laboratory maximum (Table C5.1) after 4 passes. The Porphyritic Andesite 40 mm Crusher Run did not achieve 95% of its maximum laboratory density in any bay. The range of values of density was small with a change of only 0.121
Mg/m$^3$ between different readings with a standard deviation of 3 to 4% of the dry density measured, which was a scatter of a minimum of 25% of the changes seen.

5.5.5 Rutting

The rut formation data presented were the averages of rut depths measured in both outer wheel paths at all test point in each bay (Table C5.16). These have been plotted on logarithmic scales (Figures 5.33 to 5.35). The propagation of surface rutting proceeded very slowly, with no more than 30 mm of rutting occurring in any location throughout the duration of the trafficking.

The outer wheel path received a greater degree of trafficking than the inner wheel path, as both the vehicle's single front wheel and outer back wheel pass along the same line, and only the inner back wheel passes along the inner wheel path. The level of rutting observed in the inner wheel path was thus found to be the minimum and was quite variable. The depth and rate of rutting were greatest in the outer wheel path, and the magnitude of rutting was small overall. Only the rutting from the outer wheel path is considered in detail below.

The 6F2 Porphyritic Andesite (Figure 5.33) produced a similar pattern of rutting to the Porphyritic Andesite 40 mm Crusher Run. The bay that received 16 compaction passes (Bay 4) exhibited very little rutting. The bays that received 4 and 8 passes of the roller again produced remarkably similar levels of rutting, but showed a larger initial rut than the other materials (8 mm) and then a small but linear increase (on the logarithmic plot) over the rest of the trafficking.

The 6F2 Limestone (Figures 5.34) showed the least rutting, similar levels of rutting were observed across the three bays regardless of their compaction. The bay that received greatest compaction (Bay 5) showed the best overall performance.

The Porphyritic Andesite 40 mm Crusher Run (Figure 5.35) again exhibited a low level of rutting, and the bay that received 16 compaction passes (Bay 6) showed less rutting than the other two bays. Because of the small level of the rutting observed (less than 10 mm) there was significant variability of results as the number of passes increased, although with a consistent general trend. The bays that received 4 and 8 roller passes...
showed remarkably similar levels of rutting, with a progressively increasing rut propagation, as exhibited by the linear relationship.

Generally there was significant variability in the rut depths over approximately the first 150 to 250 passes, where after the variability reduced to give broadly smooth lines. All the ruts formed basic depressions in the granular materials with no rut shoulders forming.

5.5.6 Stress-Dependency Data

The subgrade stress-dependency are given in Table C5.17 and shown on Figure 5.36, which shows the variability of the stiffness along the length of the trial, and the variability of the response of the subgrade to different applied loads. The majority of the bays showed limited stress-dependency of the subgrade with similar values of stiffness being measured regardless of the applied stress at which the measurements were made. However the higher bay numbers did show more sensitivity to applied stress (which tend to be stiffer) and showed inverse stress-dependency, i.e. the higher the applied stress the lower the measured stiffness. The values of stiffness measured at approximately 70kPa tended to be similar to those measured at approximately 110kPa, however there was a greater change in stiffness measured at applied stresses of 40 to 70 kPa.

Figure 5.37 (Table C5.18) shows a change in the stress-dependency with compaction for the 6F2 Porphyritic Andesite. The bay that received 4 passes showed a progressive increase in stiffness with applied stress. The 8 pass bay showed a similar level of stiffness regardless of the applied stress. The 16 pass bay showed a very high value of stiffness at low applied stress. This reduced to a similar value for the readings taken at 75 and 135 kPa. The Limestone 6F2 and 6F1 Porphyritic Andesite capping (Figures 5.38 and 5.39 respectively) showed similar patterns, except that the changes in stiffness between applied stress were smaller.

5.6 Trial Two (The Bardon Field Trial)

5.6.1 Subgrade Stiffness Measurements

The subgrade stiffness along the trial is shown in Figure 5.40. (Bay 1 is underlain by 400mm capping, Bay 5 is the no capping bay). The data is presented in Table C5.19. The GDP showed a slight reduction in stiffness along the length of the trial from Bay 1 to Bay
5 (from 15 MPa to 10 MPa). The TFT values of subgrade stiffness are almost twice the GDP values. Similar TFT stiffnesses were seen in Bays 1 and 2 (25 MPa), with the stiffness reducing to a value of approximately 15 to 20 MPa, between Bay 3 to Bay 5.

The readings taken at the time of the trafficking showed considerable variation from those taken at the time of construction (Figure 5.43). The TFT and GDP readings in the thinnest bays (Bays 4 and 5) showed a significant increase in subgrade stiffness at the time of the trafficking phase, whereas those in bay 2 were shown to remain approximately constant. In the long-term there appeared to be limited change in the stiffness of the subgrade, within the general pattern of variability observed along the length of the trial. All the subgrade data showed quite large scatter in results with variability (as a percentage of the mean) for the subgrade typically being in the region of 30% for the GDP and 30 to 50% for the TFT.

5.6.2 Capping Stiffness Measurements

Figures 5.41 and 5.42 show a generally progressive improvement in measured stiffness in each bay with increasing capping thickness. The TFT data for the thickest bay (400mm capping) initially showed a small increase in stiffness with the addition of the first three layers of capping followed by larger increases (approximately 10MPa) with the addition of the final layer of capping, giving a completed stiffness of 33MPa. A similar pattern was seen for the thinner bays, with typically a limited improvement (even a reduction in some cases) with the addition of the first layer of capping followed by an improvement of approximately 10MPa with the installation of the subsequent layers.

The GDP showed a similar improvement with the addition of the final layer of capping, as observed with the TFT. However it did not show a progressive improvement in stiffness over the subgrade values seen with the TFT. The thinnest three bays showed a reduction in stiffness with the addition of the first layer of capping of approximately 5MPa, followed by small increase with additional capping back to the level of the subgrade stiffness.

Figures 5.43 and 5.44 show a minimal improvement in composite stiffness due to the addition of capping over the subgrade stiffness, except for the 400mm thick bay with the GDP and in the 400 and 300mm thick bays when measured with the TFT. There was a rapid increase in stiffness with the two thickest cappings.

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The GDP showed a standard deviation of approximately 10% of the mean values presented. The standard deviations of the TFT data varied between 10 and 50% of the mean but was typically 20%.

5.6.3 Capping Density Measurement

The capping dry density data are presented in Table C5.20 and Figure 5.45. The capping showed a small increase in density for increase in supporting capping thickness. However, significant scatter was observed relative to the change in density across the bays. The density measurements did not exceed a value of 95% of the maximum laboratory density (Table C5.1) in any bay.

Figure 5.45 shows the average capping density plotted against the stiffness of supporting capping (i.e. for the 200, 300 and 400mm thick capping bays). This shows a general trend of an increase in capping density with stiffness, although there is a slight reduction in the 300mm bay relative to the 200mm bay. This shows a large change in stiffness correlates with only a small change in density, however this is masked by the scatter in the data.

5.6.4 Sub-base Stiffness Measurements

The stiffness measured with the TFT showed no significant increase in stiffness with increasing compactive effort, although a small change (5 MPa) was seen in the thicker bays (Figure 5.46 and Table C5.21). Both the TFT and GDP showed a well-defined pattern of increase in sub-base stiffness with increasing thickness of supporting capping, the curves after 8 roller passes were approximately linear for both devices. The exception to this pattern occurred with the thinnest (100 mm thick) layers of capping, which exhibited a stiffness that was little better than that of the subgrade after 2 roller passes and fell below the linear trend after 8 passes. The GDP data showed no significant change in stiffness with increasing compaction. Again an approximately linear change in stiffness was seen with increasing supporting thickness.

There was no significant improvement in stiffness, over that achieved with capping with the addition of sub-base, until the 200 mm bay (Figures 5.43 and 5.44). The GDP (Figure 5.43) showed an improvement in stiffness (approximately 5MPa) in the 200 and 300 mm bays over the top of capping value, but in the 400 mm bay the sub-base stiffness was
similar to the top of capping stiffness. The TFT (Figure 5.44) showed a significant improvement with the addition of sub-base over the stiffness measured on top of capping in the 200, 300 and 400 mm bays of approximately 10MPa. The increase in stiffness with sub-base placement was slightly larger for the thicker bays which reduced with supporting thickness, such that the sub-base in the no capping bay showed no improvement over the original subgrade stiffness.

The GDP showed an increase in stiffness of approximately 10MPa in the thicker bays (400 to 200mm bays) and slightly less for the thinner bays, between the readings taken at the time of construction and 4 days later (Figure 5.43). The 100mm and no capping bays then showed no further improvement measured by the GDP up to the time of trafficking. However, the 200, 300 and 400mm bays showed a consistent improvement of approximately a further 5MPa. Eight months after construction a further increase in stiffness was then seen, while conversely the thickest two bays showed an equivalent reduction in stiffness, and no change in the central 300mm bay.

The TFT data monitoring the long-term changes in stiffness (Figure 5.44) did not show such specific changes as the GDP data. The TFT generally showed an improvement in stiffness between the readings taken at the time of construction and those taken at the time of the trafficking, which was generally double that that seen with the GDP. A smaller increase was seen in the no capping bay (6MPa) than in the other bays, which showed slightly larger increases (20 to 30MPa) to the time of trafficking. When the final readings eight months after construction were made a further increase in stiffness was seen in the thinnest three bays. The magnitude of this increase was smallest for the thinnest bay (6MPa) and greatest for the 200mm Bay (27MPa). The 300mm and 400mm bays, similar to the GDP, showed a reduction (5 to 10MPa) in stiffness to a level below that seen at the trafficking phase, but still above that measured at the end of construction.

The FWD stiffness measurements taken four days after completion of the trial (Figure 5.44) were slightly higher than those of the TFT taken at the end of construction (by 5 to 10 MPa). The readings taken 8 months after construction showed a similar pattern of change in stiffness as the other two devices. The thinnest three bays showed a large increase of approximately 25MPa in stiffness over the values previously measured, while the 300mm bay showed a smaller increase of 10MPa. The thickest bay showed a similar stiffness to that previously measured. The FWD showed a similar general pattern of
increase in stiffness with capping thickness to the other two devices, although the 300mm bay value lies slightly below the trend.

As with the previous trial quite large scatter about the mean values calculated in each bay were observed. Generally the GDP exhibited relatively small and consistent scatter, typically 10% of the mean, while the TFT readings had a greater range of error (10 to 40%). The FWD showed a comparable scatter to the TFT of about 10 to 50%. The magnitudes of scatter in the data for the GDP and TFT were higher for the readings taken after completion of construction than at the time of construction.

5.6.5 Sub-base Density Measurement

A large increase in sub-base density with the initial compaction passes was observed, followed by a limited improvement for the final few passes of the roller (Figures 5.47 and Table C5.20). Figure 5.47 also shows an improvement in the density achieved with increasing supporting capping thickness for the central bays. The no capping bay showed a higher sub-base density than the thinnest three bays, the thickest bay 400mm capping bay showed the highest density for the bays with capping.

Figure 5.48 shows the average top of capping stiffness in each bay plotted against the average compacted sub-base dry density. A similar pattern to that seen with capping is observed (i.e. a slight increase in density with increasing supporting stiffness). Again significant error in the data was observed relative to the change in measured density.

The sub-base achieved a compacted density of 97% of laboratory maximum dry density (2.015 Mg/m³) (Table C5.1) in all bays, after 4 passes. The capping bay achieved this density after two compaction passes.

5.6.6 DCP Testing

The DCP rates of penetration from the various points tested in each bay have been averaged over the depth of each test and then the tests averaged within each bay. These results for various stages during the trial are presented in Table C5.22.

Figure 5.49 shows a broadly similar level of average penetration rate through subgrade along the trial (20mm/blow). Bay 3 (the 200 mm thick capping bay) showed a slightly
higher penetration rate (25 mm/blow) than the other bays with a much larger error. (This was due to a very high average penetration rate from one test in this bay, leading to a high bay average and standard deviation). The other points in this bay showed similar penetration rates to the other bays.

Figure 5.49 shows a high penetration rate through the top of the capping, followed by an increase in resistance to a consistent value through the remaining capping thickness. A subsequent slight reduction in penetration rate was then seen through the subgrade. There was some variability between the resistance of the capping along the length of the trial. The thickest bay shows a very poor resistance which was significantly lower than the 300 mm bay. Ignoring the 400 mm bay there was generally an increase in penetration rate with reducing capping thickness.

The data from the DCP tests through sub-base, performed at the time of the trafficking phase of the trial are plotted in Figure 5.50. These showed a high penetration rate through the sub-base, followed by a slight reduction in penetration rate through capping and then a further increase into the subgrade. No improvement in penetration resistance seemed to occur with increasing thickness of construction. The sub-base showed a similar penetration resistance across all bays and with depth. The rates of penetration through the subgrade and sub-base showed no change to those previously observed. However the capping shows a large improvement of approximately 20 mm/bay with the addition of sub-base.

The DCP results taken eight months after construction (Figure 5.51 and Table C5.22) show the sub-base penetration resistance to have increased slightly, relative to that previously observed. The capping showed a reduction in resistance to penetration over that previously observed at trafficking, back to the level previously observed at the time of capping construction. The subgrade showed a reduction in its penetration resistance along the trial.

5.6.7 Trafficking of Sub-base

The monitoring of the formation of ruts is presented in Figures 5.52 to 5.53 (on logarithmic plots) and Table C5.23 for the outer and inner wheel paths, the data presented are the averages of rut depth measured within each bay.
These indicate an approximately linear relationship beyond 10 lorry passes (on log scales) implying that the amount of rutting reduced with increasing vehicle passes. However, the progressive build up of rutting in the 400 mm thick bay tended towards stability (after 32 passes), whereas the sub-base in the 300 mm capping bay showed an increasing tendency to rutting, exhibited by the steeper downward curve. In the thinner bays, both the zero capping and 200mm capping bays exhibited serious rutting after 20 lorry passes. The sub-base in the bay containing 100mm of capping rutted far less, and appeared to reach a stable state after 50 lorry passes, although trafficking was then discontinued due to the damage to the adjacent bays.

In the outer wheel path after 22 passes the 400 and 300mm bays showed a reduction in rate of rutting, whilst the 200 mm and no capping bays continued to develop rutting at a rapid rate. The 100mm bay showed a similar rate of rutting as seen between 2 and 12 passes. After 22 passes the 200mm and no capping bays had rutted excessively, and the bay with no capping was excavated.

A further 10 lorry passes were administered, after which the 200 mm bay showed a continuing tendency to rut and was excavated (i.e. after 32 passes). The 100 and 300 mm bays rutted at a similar rate to that previously observed.

Over the next 20 lorry passes (total 52 passes) the 100 mm bay showed no further rutting, as it reached and maintained a constant value. This bay was then excavated. The 300 mm bay had been affected by the lorry having to 'climb out' of the excessive, premature rutting seen in the 200 mm bay. Trafficking was continued in the 300 and 400 mm bays to a level where the 300 mm bay could no longer be trafficked. The trafficking was ceased prior to the 400 mm bay reaching the limit.

Between the 52nd and the final (250th) pass the 400 mm bay showed a very slow rate of rutting with almost no change in rut depth being observed over the final 100 passes. The 300 mm bay showed a reduction in the rate of rutting between the 50th and 150th pass but then showed a progressively increasing trend of rut accumulation.

The outer wheel path received twice as much trafficking as the inner wheel path. Therefore the amount of rutting seen in the inner wheel path is less. In the inner path very little rutting was observed in the two thinnest bays over the duration of the trafficking. The no capping bay showed 4 mm change in rut depth between the second pass and the
time that trafficking was ceased. The 100 mm capping bay similarly showed little rutting in the inner wheel path, with no change in rut over the 52 passes prior to excavation. (This was attributed to the build up of rutting in the outer wheel paths causing heave in the line of the inner wheel path). A similar pattern was observed in the 200 mm capping bay, showing approximately 10 mm of rutting over the first 22 passes. There was subsequently a rapid increase over next 10 passes (30 mm), after which the bay was excavated.

Both the 300 mm and 400 mm capping bays showed a similar pattern and magnitude of rutting to that seen in the outer wheel paths, whereby a fast initial rate of rutting, slowed with increasing trafficking. Similar to the outer wheel path the inner path in the 300 mm bay showed a small increase in rut depth between 50 and 150 passes and then a faster increase to failure. The 400 mm bay shows little change in rut depth after 100 passes.

The initial rutting generated over the first two passes was generally similar in magnitude to that seen with the outer wheel path. All the bays except the 300 mm bay showed a similar amount of rut after two passes (approximately 8 mm), whereas the 300 mm bay showed approximately 15 mm after two passes. This latter observation should be compared with the minimal rutting (<2 mm) observed in the outer wheel path after 2 lorry passes.

Heave at the outer edges of the ruts did not occur in the thicker (300 mm and 400 mm capping) bays, whereas the thinner bays did show evidence of heave at the edges after 10 passes. This was assumed to occur when a bow wave in the base of the rut could be seen to form at the approach of the vehicle and cracks appeared at the top edge of the rut.

5.6.8 Excavation of Subgrade Sub-Surface Rutting

The excavation in the no capping bay indicated that approximately 50% of the rut was transferred through from the running surface to the subgrade. The 200 mm capping bay was the next to be excavated with between 33% and 50% of the rut transferred from the surface, although this was difficult to determine accurately. The 100 mm bay showed typically 40% of the rut transferred through to the subgrade, although one of the three profiles (that adjacent to the 200 mm bay) showed little transfer of rut.
The 300 mm bay only showed rut transfer to the subgrade at one point, which was adjacent to the soft area of the 200 mm bay (Bay 3), with approximately 70% of the surface rut depth being observed in the subgrade. The other points in the 300 mm bay showed no transfer of rut through to the subgrade.

Since no transfer of rut through to the subgrade was observed in the 300 mm bay and relatively little deformation occurred to the surface in the 400 mm bay, this was not excavated. It was felt that due to the thicker granular material in this bay all the rutting observed would be contained within the granular materials, as had been observed in the adjacent 300 mm thick bay.

5.6.9 Stress-Dependency

The stress-dependency data for the subgrade capping and sub-base collected with the TFT and FWD are presented in Table C5.24.

The limited stress-dependency of the materials over the range of stresses measured with the TFT was implied by the generally flat nature of the curves plotted from the data (Figure 5.54). There was slight evidence of increasing measured stiffness with increasing applied stress, confirmed by Bay 1 which showed a significant stress-dependency.

The TFT capping stress-dependency data (Figure 5.55) showed a slight increase in the stiffness measured, with increasing applied stress over that seen with the subgrade, although Bay 1 showed a small reduction in change in stiffness with applied stress over that which was previously seen for subgrade.

Figure 5.56 shows the TFT stress-dependency for top of sub-base. This shows an increase in the stiffness with applied stress for the thicker bays (the 400 and 300mm bays), especially over the lower stress range, Bay 2 showed the biggest change. The thinner bays show a similar pattern of stress-dependency to that seen on the capping.

The repeat stress-dependency measurements taken with the TFT and FWD eight months after construction (Figure 5.57) showed a further change in the patterns of stress-dependency in bays three and four, which showed a greater increase in stiffness with increasing applied stress than seen previously. The thickest two bays and the thinnest bay, although showing a change in stiffness, did not show a change in the pattern of stress-
dependency. The stress-dependency testing with the FWD (undertaken at higher applied stresses than the TFT), showed very flat stress-dependency with no change in stiffness for increasing applied stress.

**5.7 Trial Three (The Mountsorrel Field Trial)**

The data collected on the two Mountsorrel trial roads are presented separately. Initially the results from the Granodiorite trial are presented followed by the data from the Limestone trial.

**5.7.1 Granodiorite Trial Results**

**5.7.1.1 Subgrade Stiffness Measurements**

The Subgrade showed a reduction in stiffness of approximately 5MPa from 18 MPa in the 150mm to 13 MPa in the 450mm (MPa) bay, with a standard deviation of 15 to 30% of the mean (Table C5.25 and in Figure 5.58).

**5.7.1.2 Capping Stiffness Measurements**

A consistent improvement in stiffness of approximately 7 MPa (measured with the GDP) with the placement of the first layer of capping over the subgrade measurements was seen along the trial (Figure 5.59 and Table C5.25). With the addition of the next layer a small reduction in stiffness was seen to occur, and there was no further change with the placement of the final layer.

During compaction of capping the GDP (Figure 5.59 and Table C5.26) showed the thinnest bay (150mm) to progressively increase in stiffness with compaction. The middle bay (300mm) showed a limited improvement over the first two passes of the roller and an improvement of 3 MPa with the final three passes. The thickest bay showed no improvement in stiffness with the initial two passes, followed by a larger increase of 7 MPa with the final passes. The FWD (Figure 5.60) measured a progressive increase, again of 7 MPa in capping stiffness in the thickest and thinnest bays with increasing compaction. However, the measurements in the central bay (300mm) appear erratic, with a higher stiffness being observed after 1 pass (41 MPa) than after two, followed by a larger increase (11 MPa) after five passes to give a stiffness greater than that in the 450
mm thick bay (49.5 MPa). Ignoring the central bay with the FWD, both the GDP and FWD showed a small overall increase in stiffness with thickness of capping, although this may only reflect the differences in the underlying subgrade stiffness. However, the standard deviations of the mean data presented are typically larger than the difference in stiffness between bays.

The repeat stiffness measurements taken prior to the additional compaction (measured with the GDP) showed marginally lower values of stiffness than those measured at the end of the construction, except in the central bay which showed a higher value similar to the trend seen with the FWD (Figures 5.59 and Table C5.27). After an additional 6 passes of the roller (11 Passes) the stiffness on top of capping reduced considerably in all bays to the level previously measured on the subgrade. A further 4 passes of the roller (i.e. total of 15) produced no change in the stiffness of the 150 mm bay, a small increase in the central 300 mm thick bay and a slightly larger increase (5MPa) in the 450 mm bay. Nevertheless the stiffness did not return to the level achieved after the initial 5 passes. The thinnest bay showed an overall reduction in stiffness of 20%, the central bay a reduction of 25% and the thickest 10%.

The repeat GDP and TFT stiffness measurements taken on the Granodiorite capping prior to the construction of sub-base (after trafficking, re-grading and a period of rain), showed a linear increase in capping stiffness with capping thickness for both the TFT and GDP (Table C5.27 and Figures 5.59 and 5.61). The GDP showed a further change in capping stiffness from the time of additional compaction, while the thinnest bay reduced in stiffness to a value lower than the original subgrade stiffness, and the central bay showed a similar stiffness to that measured after additional compaction. The thickest bay showed an increase back to the level previously seen at the end of initial capping construction. The TFT reads twice the value of the GDP.

5.7.1.3 Capping Density

The 150 and 300mm thick capping bays showed a progressive increase in capping density with compaction (Figures 5.62 and Table C5.28). The density in the 300 mm bay being marginally greater than that in the 150mm bay. The thickest bay (450mm) showed a much greater increase in stiffness after the second pass of the roller and little improvement thereafter. Similar values of final compacted density are observed on the thicker two bays.
Figure 5.63 shows that a large change in supporting stiffness is required to achieve a small change in density. As with the previous trials the standard deviation to the data is large in comparison to the changes in measured density.

A compacted density of 95% of the maximum laboratory compacted density (1.62 Mg/m\(^3\)) was achieved in all bays. The 300 mm and 450 mm bays all achieved the target density after two passes, while the 150 mm bay achieved this density after 5 passes of the roller.

5.7.1.4 Rut Formation and Testing During Trafficking

Figure 5.64 shows the average of the rut depths measured during trafficking (Table C5.29) in each of the three bays for both the inner and outer wheel paths. The magnitude of rutting generated over relatively few passes was large. After 14 passes of the lorry, ruts of 25 mm and 32 mm (inner and outer wheel path) were created in the 450 mm capping bay, while 26 mm and 37mm ruts formed in the 300 mm bay and 49mm and 66mm in the 150 mm capping bay. The outer wheel path showed a fast rate of rutting over the first two passes, which slowed slightly with increasing passes. The inner wheel paths showed a high rut formation over the first two passes which slowed slightly to a constant value for the remaining passes. Figure 5.65 shows the rut profiles at the end of the trafficking (data averaged along the length of the trial in each bay). This shows the formation of shoulders at the sides of the rut where material had been displaced.

The outer wheel path received twice as many wheel passes as the inner wheel path, albeit with different applied stresses, since both the single front wheel and the outer back wheel pass along the same path. Consequently the depth and rate of rutting were greater in the outer wheel path.

5.7.1.5 Sub-base Stiffness

Both the GDP and TFT show a considerable increase in stiffness with the addition of sub-base (Figures 5.59 to 5.61, Table C5.30). After one roller pass the GDP (Figure 5.59) showed a significant increase in sub-base stiffness across all bays, with the largest increase being in the thinnest bay and a lesser increase across the other bays. This change reduces with the thickness of the trial. After the second roller pass a further increase was seen across all bays. The thinnest bay showed a small increase, while the thicker bays
showed a much larger increase of approximately 5 MPa. After the final roller pass the stiffness reduced by a consistent amount across the trial, with the 150mm and 450mm bays reducing to a level similar to that seen after one pass. The stiffness of the middle bay reduced to a similar stiffness to that measured in the thickest bay.

The TFT (Figure 5.61) showed an improvement in sub-base stiffness over that measured on capping after one pass, of approximately 25 MPa. The middle bay showed a slightly larger improvement than the other bays. With the second pass there was a minimal change in stiffness in all bays, with the final roller pass a further improvement of 5MPa was measured in the thickest and thinnest bays with no improvement in the middle bay. The increase in stiffness with the addition of sub-base reduced the differences in stiffness seen between the different bays.

The FWD (Figure 5.60) only measured an improvement over capping stiffness with addition of sub-base in the thickest bay. It showed a slight reduction in the thinnest bay over the capping stiffness and a much lower stiffness than that previously measured in the middle 300mm bay. The FWD showed a reduction in measured stiffness after one pass of the roller in the thinnest two bays (more in the central bay), and a slight increase in the thickest bay, which resulted in a linear increase in sub-base stiffness with supporting capping thickness. The second roller pass caused a minimal change in stiffness in all of the bays. The final roller passes then caused small increases of 5MPa, giving a final stiffness in the thinnest and middle bay slightly below that previously measured on the capping. However, a higher stiffness than on capping was measured in the thickest bay. This resulted in a progressive increase in stiffness across the trial with increasing thickness.

Repeat measurements of stiffness eight months after completion showed a slight improvement in stiffness in the thinnest bay measured with the GDP and FWD of 5 and 13 MPa respectively, and a reduction of 3 MPa with the TFT. The 300mm bay showed a similar stiffness to that measured after construction with the GDP and FWD and again a reduction (of 8 MPa) with the TFT. The thickest bay showed no change when measured with the GDP and a slight reduction in stiffness when measured with the FWD and the TFT (12 MPa).
5.7.1.6 Sub-base Density

The density of sub-base showed an increase in density with the number of compaction passes (Figure 5.66 and Table C5.28). The magnitude of the density after one pass varied slightly with thickness of support, with the 450 mm capping bay showing the lowest sub-base density. All bays showed a similar increase in density between the first and second roller pass. With the final passes the 300 mm thick capping layer showed no further change in density, whereas the density on the 150 and 450 mm capping showed smaller increases such that a similar density in all three bay was achieved after five passes.

No significant change in density with change in stiffness was observed (Figure 5.67), although the measured stiffness was relatively consistent across all three bays. The standard deviation of the data was again large compared to the changes in density measured.

A sub-base density of 97% of laboratory maximum compacted density was achieved in all bays. The 150 mm and 300 mm bays achieved this density after two passes of the roller, whereas the 450 mm bay attained this density after five passes.

5.7.1.7 DCP

The subgrade initially showed penetration rates of approximately 30mm/blow across all bays (Table C5.31). With the addition of capping the subgrade penetration rates remain constant. The capping showed increasing penetration rates with reducing thickness, with the thickest bay showing a values of 17mm/blow and the thinnest bay 36 mm/blow which was only slightly more than the subgrade. The capping penetration resistance does not appear affected by the rain, and showed no significant change with the addition of sub-base. The subgrade showed no change in penetration to the time of sub-base construction except in the middle 300mm bay which showed a reduction in resistance of 10mm/blow, although this was probably due to one extremely high rate of penetration from one test after the rain.

In the long-term the subgrade showed no overall change in resistance from the values originally measured. The capping showed a large improvement in resistance to penetration after 8 months, a slight improvement was seen for the sub-base.
5.7.1.8 Stress-Dependency

The stress-dependency testing of the subgrade was undertaken on the limestone trial sections and is discussed in Section 5.7.2.8.

The change in measured stiffness with increasing applied stress for Granodiorite capping, showed that the measured stiffness increased slightly and progressively with increasing applied stress, with a similar pattern seen across all bays (Figure 5.68 and Table C5.32).

Similar to the capping the sub-base showed an increase in the measured stiffness with increasing applied stress (Figure 5.68). However the increase in stiffness was larger for sub-base than for capping, especially over the lower two applied stresses. This was followed by a limited change in stress-dependency between the middle and highest applied stresses.

The repeat stress-dependency data from tests performed after 8 months with both the TFT and the FWD (Figure 5.69) showed a similar pattern to that previously seen. However they showed no further increase in stiffness with increasing stress at the higher applied stresses used with the FWD (above that level already applied by the TFT), while the thinner two bays showed reduction which was similar for both bays.

5.7.2 Limestone Trial Results

5.7.2.1 Subgrade Stiffness Measurement

The stiffness measurements on the subgrade with the GDP (Figure 5.70 and Table C5.33) showed a slight reduction of 4 MPa along the trial giving an average subgrade stiffness of 10 MPa. The TFT (Figure 5.71 and Table C5.33) showed a larger increase of approximately 15MPa along the trial with an average of 18 MPa. The Limestone trial subgrade was slightly less stiff than the Granodiorite sections (7 to 12 MPa with the GDP). The TFT indicated the subgrade to have a stiffness of approximately 12 MPa in the thinnest bay to 22 MPa in the thickest. The standard deviations of the measured sub-grade were approximately 10 to 20% of the mean with the GDP, and approximately 30 to 45% with the TFT.
5.7.2.2 Limestone Capping Stiffness

Both the GDP and TFT showed a progressive increase in stiffness with additional layers of limestone (Figure 5.70 and 5.71). There was initially an increase in stiffness of approximately 10 to 15 MPa with the first layer measured by the GDP and 15 to 20 MPa with the TFT. The rate of increase reduced with the addition of the second layer of capping, however a further increase was seen. The magnitude of this varied between 5 and 20 MPa, the results showed large standard deviations due to a few high individual readings in each bay. The addition of the final layer in the 450mm bay showed a small increase in stiffness with a large standard deviation when measured with the GDP. The TFT showed an increase of 18 MPa to 60 MPa but again with a large standard deviation.

Figure 5.72 to 5.74 and Table C5.34 showed the change in top of capping stiffness with compaction of the final layer of capping measured with the various devices used. These showed that the thicker the supporting capping, the stiffer the completed layer for all three devices. The GDP (Figure 5.72) showed an increase in capping stiffness in the thickest bay over that seen on subgrade after 1 pass, followed by a small reduction in stiffness of 4 MPa with further compaction. The thinnest bay showed no change in stiffness with compaction pass over the level achieved after 1 pass. The central 300mm bay showed a similar pattern of reduction to the thickest bay.

The TFT showed an improvement of 20MPa in the thinnest (150mm) bay and 40MPa in the thickest (450mm) bay. The TFT similarly showed a large improvement in stiffness addition of capping after one pass, followed by a small improvement of 6 MPa in the thinnest bay to the end of compaction. The other thicker bays showed a small reduction with further compaction. The FWD (Figure 5.74) indicated a general increase in stiffness for all three capping thicknesses with compaction, although the trend is erratic. The thinnest bay showed a progressive improvement with compaction. The 300mm bay showed a similar stiffness to the 150mm bay after one pass, while after three passes the stiffness increased by 20 MPa and then showed no further change with the final passes. The thicker bays showed a very high stiffness, twice that seen in the other bays, followed by a slight reduction after three passes and a subsequent further increase.

The repeat GDP and TFT stiffness measurements taken on the reconstructed top layer of limestone capping prior to the installation of sub-base showed a large reduction in the measured stiffnesses compared to that seen previously (due to the affects of the rain
ingress). The GDP (Figure 5.72 and Table C5.34) showed a reduction in stiffness to a level marginally above that previously seen on the subgrade. The thinnest bay showed a slightly lower stiffness than that seen previously in the thinnest bay and a similarly low value in the other two bays indicating a large reduction in stiffness in the thickest two bays (10 to 20 MPa). Figure 5.73 shows the TFT to measure capping stiffness at a similar level to that measured upon the subgrade, with the 450mm bay being less stiff than the original subgrade. Both devices indicated that the reconstructed limestone has a similar stiffness regardless of the thickness of the capping, having shown a bigger reduction with increasing thickness. A reduction in stiffness in excess of 50% is observed.

5.7.2.3 Limestone Capping Density

The 150mm thick bay showed a progressive increase in capping density with compaction (Figures 5.75 and Table C5.35), the 300mm bay showed an increase with the first three passes followed by a small reduction in density with the final passes. The thickest bay showed a higher initial density than the other bays, followed by a minimal increase with the first set of passes and a larger increase up to the end of compaction. All three bays showed similar values of density at the end of compaction, however the 300mm bay shows a marginally lower value than the other bays.

Figure 5.76 shows the capping density to be independent of stiffness of the supporting capping since the compacted density remained approximately constant regardless of stiffness.

The density in all bays achieved a level above 95% of the maximum laboratory compacted density (Table C5.1) after three compaction passes on the thinnest two bays and after one pass on the thickest bay.

5.7.2.4 Trafficking and Ancillary Testing of Degraded Limestone

The Limestone capping had softened significantly due to wetting following heavy rain. When the lorry undertook its initial two passes of the material it immediately caused 60 mm of rutting. Water was displaced from the softened material as it compressed, filling the ruts formed with water. Profiles of the ruts formed after the two passes of the lorry are presented in Figure 5.77. Limited repeat GDP measurements on the wet Limestone gave values of approximately 10MPa, a substantial reduction from values previously seen.
Laboratory water contents measurements showed that the water content of the Limestone had almost doubled from 6.8% to 11.8%.

5.7.2.5 **Sub-base Stiffness on Limestone Capping**

The originally constructed bays of Limestone capping were greatly affected by water ingress following the heavy rain. The replacement 150 mm layer of capping material used to reconstruct the surface of the capping had a significantly lower stiffness than the originally compacted Limestone due to the reduction in strength and stiffness of the underlying materials. As a result of this degradation, no clearly discernible pattern of change in capping stiffness with supporting capping thickness was found, and this affected the stiffness results collected on the Sub-base. The change in stiffness with compaction of Sub-base, are shown in Figures 5.72 to 5.74 and Table C5.36.

The GDP (Figure 5.72) showed no significant increase in stiffness with addition of sub-base. The thinnest bay initially showed a small increase of 2 MPa followed by no change, and showed no significant improvement over the reconstructed Limestone. The middle bay initially showed an increase in stiffness after one pass of 5 MPa, and no further change after two passes followed by a reduction after five passes back to a similar level to the other bays and similar to the original capping stiffness. The thickest bay showed no change in stiffness with compaction of sub-base with the stiffness remained at a level equivalent to that of the underlying limestone. The final values of stiffness in all bays were similar to that on the limestone.

The TFT (Figure 5.73) showed an improvement of approximately 22 MPa over capping in the thinnest two bays during the initial pass, the second pass showed a further minimal increase followed by a small reduction (3 to 5 MPa) up to five passes. The middle (300mm) bay showed a smaller increase with the second pass than with the first, and a equivalent reduction with the final passes to give a finished stiffness similar to that seen after one pass, and similar to the value seen in the thinnest bay. The thickest bay showed a smaller improvement in stiffness over capping stiffness of about 10 MPa after one pass, followed by a further small increase with two passes. Finally the stiffness reduced with the final passes to a level only marginally above the original capping stiffness, and lower than the final stiffness of the other, thinner, bays by 8 MPa.
The FWD (Figure 5.74) showed a similar pattern, with a low stiffness being measured after one pass (15MPa) followed by an increase across all bays (approximately 20 MPa) with a further pass, followed by an equivalent reduction with the final passes back to the original stiffness.

The repeat stiffness measurements made eight months after completion all showed a considerable improvement in stiffness from that seen at the end of sub-base compaction. The GDP and TFT both showed an improvement in stiffness of approximately 15 MPa along the length of the sections, giving a trend of a slight reduction in stiffness with increasing supporting stiffness carried through from the previous stiffness measurements. The FWD showed a larger increase of 30 MPa in the thinnest bay, with slightly less improvement in the middle bay. The thickest bay showed an improvement of 20 MPa, giving a similar pattern of final stiffness to the other two devices. Only the thinnest bay showed final stiffness values above those measured on the original capping.

5.7.2.6 Sub-base Density on Limestone Capping

The thickest bay showed the highest initial density with compaction after one pass, followed by a small increase in density after two passes and slight reduction after five (Figures 5.78 and Table C5.35). The 300 mm bay had the lowest initial density, and showed a large increase in density between the first and second roller pass and a similar reduction to the thickest bay after five passes. The thinnest bay showed a progressive increase in density over the course of compaction (similar to capping) with a slightly greater increase over the first two passes than the final three, to yield the highest density of all three bays. A minimal change in density with change in stiffness was seen (Figure 5.79), although this had been affected by the variable nature of the stiffness results across the bays. The density measurements were all higher than 97% of the laboratory maximum compacted density (Table C5.1) after two compaction passes.

5.7.2.7 DCP Testing

The results from the DCP testing undertaken throughout the trial are presented in Table C5.37. This shows the average penetration rates averaged across each bay for material.

The subgrade initially showed penetration rates of approximately 30mm/blow across all bays. The DCP could not initially penetrate the Limestone, yet after the rain the material
could be penetrated readily, giving penetration rates through the limestone of 14 to 35mm/blow. The DCP results also indicated a softening of the surface of the subgrade and a loosening of the capping over the top 50 to 100 mm, although the average subgrade penetration rates remained similar to those originally measured.

With construction of sub-base the subgrade again showed similar penetration rates, the capping showed a further worsening of the capping material shown by an increase in penetration in the thickest bay. A slight improvement was seen in the other two bays, probably due to the affect of the replaced limestone. The sub-base itself showed high penetration rates with values being similar or marginally better than the underlying capping.

Eight months after completion the subgrade again showed no significant change, with the exception of the thickest bay which shows a doubling in resistance to penetration. The capping however did show a large increase in its penetration resistance, with penetration rates being approximately a fifth of the previous values. The sub-base also showed a improvement in penetration resistance, with the thickest bay showing an improvement from 33 to 21 mm/blow. The central bay showed a similar improvement, however the thinnest bay showed no change.

5.7.2.8 Stress-Dependency

The stress-dependency testing with the TFT for the Mountsorrel subgrade was undertaken on the subgrade for the limestone trial, and on an adjacent area of subgrade exposed eight months after the trial. The stiffness was found to reduce slightly with increasing applied stress, except the thinnest bay which shows limited stress-dependency, (Figure 5.80 and Table C5.38). The magnitude of the stiffnesses at the various applied stresses was similar for the thinner two bays and eight month values, the 450mm bay showed a higher stiffness than the other three sets of data, and a slightly larger reduction in stiffness with increasing applied stress.

The capping showed a small change in stress-dependency with the addition of capping such that the stiffness increased slightly with increasing applied stress (Figure 5.81 and Table C5.38). Similar patterns of capping stress-dependency were found across all bays.
Similar to the capping an increase in the measured stiffness with increasing applied stress was observed (Figure 5.81). However the increase in stiffness was larger with increasing applied stress for sub-base than for capping.

The repeat stress-dependency data performed after eight months with both the TFT and the FWD showed a similar pattern to that previously seen (Figure 5.82), but showed no further increase in stiffness with increasing stress at the higher applied stresses used with the FWD (above that level already applied by the TFT). In fact the thinner two bays showed a reduction which was similar for both bays.
Figure 5.1, Grading, 6F2 Cappings from the Field Trials

Figure 5.2, Grading, Fine Particle Size Materials from the Field Trials
Figure 5.3, Grading, Type 1 Sub-base, from the Field Trials

Figure 5.4. Relationship Between Dry Density and Water Content for Laboratory Compaction Tests on 6F2 Capping Materials from the Field Trials
Figure 5.5, Relationship Between Dry Density and Water Content for Laboratory Compaction Tests on Fine Particle Size Materials from the Field Trials

Figure 5.6, Relationship Between Dry Density and Water Content for Laboratory Compaction Tests on Type I Sub-base from the Field Trials
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Figure 5.12, Relationship between Repeated Deviator Stress and Resilient Modulus for the Nine Undisturbed Samples from Trial 1 Tested in the Repeated Load Triaxial Test.
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Figure 5.27, Comparison between Single Tests at a Point and Three Tests at a Point for Porphyritic Andesite 40 mm Down Quarry Waste, Trial 1

Figure 5.28, Comparison of GDP Stiffness Measurements on 6F2 Porphyritic Andesite both Before and After Trafficking and After 8 Months, Trial 1
Figure 5.29, Comparison of GDP Stiffness Measurements on 6F2 Limestone both Before and After Trafficking and After 8 Months, Trial 1

Figure 5.30, Comparison of GDP Stiffness Measurements on Porphyritic Andesite 40 mm Down Quarry Waste both Before and After Trafficking and After 8 Months Trial 1
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Figure 5.34, Rut Formation in the 6F2 Limestone Capping for Bays Subjected to Different levels of Compactive Effort, Trial 1
Figure 5.35, Rut Formation in the Porphyritic Andesite 40 mm Down Quarry Waste for Bays Subjected to Different levels of Compactive Effort, Trial 1

Figure 5.36, Relationship between Applied Stress and Stiffness measured with the TFT on the Subgrade, Trial 1
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Figure 5.38, Relationship Between Applied Stress and Stiffness Measured with the TFT on 6F2 Limestone, Trial 1
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Figure 5.42, Stiffness Measurements Made with the TFT During Construction of Trial 2
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Figure 5.52, Sub-base Rut Formation in the Outer Wheel Path for Trial Sections for Different Supporting Capping Thickness Bays, Trial 2
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Figure 5.54, Relationship Between Applied Stress and Stiffness Measured with The TFT on the Subgrade, Trial 2
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Figure 5.60, Relationship Between Subgrade, Capping and Sub-base Stiffness During Compaction and Capping Thickness for FWD Measurements Made on the Granodiorite Trial 3
Figure 5.61, Relationship Between Subgrade, Capping and Sub-base Stiffness During Compaction and Capping Thickness for TFT Measurements Made on the Granodiorite Trial 3

Figure 5.62, Relationship between NDG Dry Density and Number of Compaction Passes for Different Capping Thicknesses for the Granodiorite Trial 3
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Figure 5.64, Relationship between Wheel Path Rut Depth and Number of Lorry Passes for Trafficking of the Granodiorite Trial 3
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Figure 5.66, Relationship between Sub-base NDG Dry Density and Number of Compaction Passes for Different Supporting Capping Thicknesses for the Granodiorite Trial 3
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Figure 5.70, Stiffness Measurements Made with the GDP during construction on the Limestone Capping Trial, Trial 3
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Figure 5.72, Relationship Between Subgrade, Capping and Sub-base Stiffness During Compaction and Capping Thickness for GDP Measurements Made on the Limestone Trial 3
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6.0 DISCUSSION OF RESULTS

6.1 Introduction

Chapter five reports and presents the result from the various laboratory tests and field trials. This chapter discusses these results and the trends that were observed. Initially the laboratory classification tests are discussed, followed by the repeated load triaxial test data. The repeated load triaxial data section discusses the behaviour of the materials during testing, the effects of sample preparation, the test methodologies used and the apparatus. The field trials are then discussed commenting upon the measurements made, the changes in performance observed, the effects and suitability of the tests utilised and the performance under trafficking. The comparisons made between the laboratory and field tests are then commented upon, followed by consideration of the measured parameters and the effects they have demonstrated on the performance of pavement foundations. Finally the implications of these results on performance based design and field assessment are considered.

6.2 Material Characterisation

6.2.1 Granular Materials

6.2.1.1 Grading

The grading tests revealed the variability of materials supplied from the various sources. Both Limestone 6F2 cappings were from the same supplier, yet had different gradings and different performance was observed. The majority of the materials used in the field trials were ordered as standard graded materials. However, they frequently had gradings close to the standard limits, and four were marginally outside them. The quantity of materials required for the trials was small, and supplied from the quarry stockpiles, where segregation is possible, rather than being batched and graded for the order.
6.2.1.2 Compaction Tests

The parameters established from aggregate tests were strongly influenced by particle sizes and their distribution, water content and mineralogy of the materials under consideration. The compaction tests for the Bardon (Trial 2) capping showed the variability for a finer particle size material. The tests showed it had a relatively high maximum dry density. A repeat test was performed, which again showed a very variable water content-density relationship. The optimum water content of Hathern (Trial 1) Limestone (determined in the compaction test) seemed low at 2.2%. Similar Limestone used at Mountsorrel (Trial 3) showed the material to have an optimum value of 7%, implying that the difference in the material gradings affected the material performance.

Problems with particle size became apparent with the compaction tests. The compacted density was significantly affected by voids that resulted when two large particles were located next to each other during the test. The large particles in Limestone from Trials 1 and 3, could be seen to fracture during compaction, and this will have affected the maximum density recorded. It was easier to perform the tests upon the finer materials (<40mm particle size), yet variability of results was still found, as observed in the differences between the sub-base parameters and repeat testing of the fine capping materials (Section 5.2.1).

The maximum compacted density and optimum water content were calculated for each material using least squares linear regression. Due to the flat nature of the compaction curves errors will have resulted from the selection of the point of maximum density and optimum water content, due to accuracy of the fitted curve (indicated by the correlation coefficients). Therefore the establishment of repeatable and precise maximum values of density can be difficult, as found by Rolton et al, (1976).
Similar density values and variability in data to those found in the laboratory compaction tests were reproduced with the NDG box calibration tests (BS 1377, 1992 Part 9) where a larger sample is tested. This implies that some difficulty exists in evenly compacting granular materials in the laboratory to model field conditions. This is possibly due to some extent to the compaction methods used not replicating field compaction (notwithstanding the variability that will inevitably be found in the field).

Performing compaction tests using the vibrating hammer (BS5385, 1985) was difficult for coarse capping due to the particle size relative to the size of the compaction mould (125mm maximum particle size being compacted into a 150 mm diameter mould). A test that is capable of accurately assessing compaction properties of large particle size material in the laboratory is required.

6.2.1.3 Peak Shear Stress Ratio (PSSR) Tests

Generally the materials all had acceptable PSSR values. All the Type 1 materials had high PSSR values and behaved as would have been expected under trafficking. Both the cappings from Trial 3 (Granodiorite 40mm down quarry waste and Limestone 6F2) did not perform adequately under trafficking, showing considerable internal instability, although the PSSR values indicated that they should have performed adequately.

PSSR tests were performed to assess the granular materials' internal stability. Many of the tests performed did not show a characteristic peak in the values of shear force applied (which is normally seen for denser materials tested in the shear box), but showed a progressive increase in the applied shear stress up to the displacement limits of the box. However, Earland and Pike, (1984) suggested that the peak shear stress should occur at approximately 3 mm horizontal deformation. This suggests that the materials were not at their maximum density, however, the materials were compacted to above 96% of the laboratory maximum compacted density, as required by the test methodology. This implies that problems in the calculation of the materials maximum density (Section
6.2.1.2) affected the results, leading to inappropriate compaction of the materials. In addition the amount of material removed to achieve the required PSSR grading for the coarse materials (6F2) will have undoubtedly have affected their performance.

For coarse materials both the percentage removed and the grading of the remaining particles will have greatly influenced the overall performance of the material and the required compacted density. This is likely to vary significantly from the field compacted state of a capping material which will include the coarser particles. Therefore the validity of PSSR tests on coarser material is questionable.

Due to poor weather, the Limestone 6F2 capping (which had performed adequately during Trial 1), showed very poor performance in Trial 3 under trafficking, partly due to the apparent softening from water ingress (for further comparisons of PSSR to trafficking performance see Section 6.4.3). This suggests that a soaked PSSR type test may be advisable to assess the performance of materials after a period of rain during construction, by wetting and soaking the material in the shear box after it has been compacted.

The PSSR test is regarded as a crude indicator of a material’s inherent resistance to permanent deformation. These results confirm its suitability for testing sub-base (i.e. well-graded materials which can be tested without the need for large particles to be removed). However, the results for cappings imply that the PSSR was unsuitable for granular coarse cappings, as it did not predicted the poor behaviour of a material with a non standard grading. Ideally a new test needs to be developed that can predict the behaviour of capping materials, including testing those containing particles sizes up to 125 mm. Whilst the principle of the PSSR test and the shear box may be applicable for such a test, more research is clearly required.
6.2.2 Subgrade Classification Testing

6.2.2.1 CBR Tests

The predominantly cohesive samples tested exhibited little change in water content when soaked, and hence little change in CBR. However, due to the shallow sample depth it is anticipated that the soils sampled are already at their long-term water content (Section 6.3.3), which may account for the minimal changes observed. A comparison of the compacted density values indicated that the changes in CBR value were directly related to changes in sample density, and hence water content (Section 5.2). This implies that the compaction method may have resulted in slightly different sample properties, which led to the different CBR values measured.

6.2.2.2 Index Tests

Generally the results from the same site are reasonably consistent, particularly for the cohesive materials. Atterberg limit data for Trial 1 samples tend to show slightly more variability than samples from the other sites, and this is attributed to the mixed nature of the Trial 1 soil, which contained sand, silt and clay which varied in proportion along the length of the site. Care has to be taken when relying upon plasticity results from mixed soils as they are only relevant to material passing a 425μm sieve (further discussed in Section 6.3).

6.3 Repeated Load Triaxial Testing

6.3.1 Introduction

Six basic elements of the repeated load triaxial testing are commented upon in this section. These elements are the sampling of the soil, the prepared state, the method of
sample preparation used, the on-sample instrumentation, the apparatus used, and the
behaviour of the materials in the test.

6.3.2 Soil Sampling

Undisturbed (U100) samples for triaxial testing were taken at subgrade level from various
sites. Problems were experienced with the recovery of samples. At a number of sites few
usable samples could be recovered, since some samples contained voids, pockets of sand
or large gravel that caused the soil to collapse upon extrusion and to deform within the
tube during sampling.

The U100 samples were obtained by hand driving sample tubes using a drop weight and
anvil. It often proved difficult to drive the tubes vertically into the soil, since the sample
tube tended to tilt as it followed the path of least resistance into the soil. The impulse load
applied by the falling weight also affected the amount of disturbance to the soil, with
some of the samples recovered from the softer cohesive soils showing marking on the
side coinciding with the increments of driven depth. Despite the tubes being thoroughly
cleaned prior to use, friction of the sample on the walls of the tube during either driving
or extrusion resulted in some samples producing a curved column of soil rather than a
straight-sided sample. The curvature induced was so excessive in some samples that
suitably straight triaxial samples could not be recovered. It is not known whether such
problems would have occurred with conventional percussion shell sampling. These
features exacerbate the potential problems due to sample variability caused by sampling
in different locations within the same site (Section 6.3.4). The method of sample recovery
and disturbance caused will undoubtedly affect the sample performance.

6.3.3 Prediction of Equilibrium Water Content

The prediction of equilibrium water content is subject to a great many variables and
presents significant problems for mixed soils (Section 2.6) since it is based on the
measured plasticity. For soils that contain a granular fraction, which is removed for the plasticity tests, the calculation of equilibrium water content is distorted. Material variability also means that the plasticity data and in-situ water content can vary between each sample. The starting water content has a significant effect on the equilibrium water content predicted, consequently the index tests must be performed on material taken from, or immediately adjacent to, the U100 samples.

Because the majority of the samples collected in this research were taken near the surface, it is likely that the soils are approximately at their equilibrium water content values. Samples from a greater depth would be expected to be below their perceived equilibrium values due to removal of overburden and the consequent increase in negative pore water pressure, causing the samples to wet up with time as the suctions are dissipated.

The drawbacks of LR 889 (Black and Lister, 1979, as reported in Section 2.6) have been highlighted by a number of samples. The removal of the sand and gravel present in the soil for the index tests results in the predicted long-term equilibrium water content being lower than the in-situ water content, hence the limited amount of equilibrium testing undertaken. Prediction of suitable and repeatable equilibrium water content values remains a challenge for future research.

6.3.4 Resilient Deformation Behaviour (Stiffness)

The majority of soils tested in this research have been cohesive materials, usually silty clays, and have shown broadly similar patterns of behaviour. The materials regardless of their state, have all shown inverse stress-dependency of varying degrees. For stiffness versus repeated deviator stress a relationship showing a logarithmic curve (non-linear) tending towards a stiffness asymptote (linear) at higher deviator stress was found (i.e. a point was reached where the resilient deformation of the material increased proportionately to the repeated applied stress). This was consistent with the pattern predicted by the Kθ model (Section 2.5.5). The constants C₁ and C₂ were found by fitting
a curve to the triaxial data to give the values in Table 6.1. Hathem (Trial 1) subgrade samples had decay (power) constant \( C_2 \) in the range -1.06 to -0.09. The majority of samples tested have \( C_2 \) values in the range -0.2 to -0.6. These values correlate well with those found by other authors (Section 2.5.5).

The values of \( C_1 \) and \( C_2 \) show the variability of the performance of soil samples with different values being found between samples from the same site and between sites, showing the parameters to be soil specific. They must however be considered to incorporate the variability in strain measurement at low deviator stress (Section 6.3.11.1), and this will reduce the accuracy of the curves fitted and hence affect the constants.

The value of stiffness at the asymptote varied from sample to sample and from soil to soil. The variability between asymptotic values from samples from the same site was not as large as the comparable variability observed at low deviator stress (Figure 5.12). No pattern of stiffness with saturation was seen for the in-situ soil samples, as would be expected (Thompson and Robnett, 1979, Section 2.5.5). However, the samples tended to show a relationship with bulk density, such that the denser the sample the stiffer the sample, notwithstanding the fact that this effect was partially masked by the variability. This was not the case for the remoulded samples from the repeatability tests, which show a more consistent relationship with water content/ saturation (Section 5.3.4, Table C5.9). However for the other remoulded samples the performance improved with the compacted density of the materials, which was affected by the water content relative to the optimum water content value, as observed by Seed et al, 1983 and Hitcher, 1996.

For the more consistent soils from the same site (i.e. Mountsorrel, Trial 3), the higher the sample water content the poorer the performance (comparing remoulded and wetted samples). Samples beyond plastic limit showed significantly poorer performance than those below. The differences in the plasticity data between samples was indicative of the different response of samples from the same site, when considered relative to the sample
water content, and hence the consequent effects of suction on the stiffness measured was observed (Fredlund, 1977; Cheung, 1994; and Black and Lister, 1979).

Not withstanding the variability of the calculated soil stiffness for each sample at low applied stress (Section 6.3.11.1), the natural variability of the material properties between samples from the same site is an area of concern. Significant variability between stiffness data from samples from the same site and prepared in the same way was seen for all sites. This variability will have a significant bearing on the collection of analytical data for design, since the selection of suitable stiffness design parameters at appropriate applied stresses was shown to be difficult.

Assessment of the stiffness asymptote between the various samples (Table 6.1) shows that typically it was approached at an applied deviator stress similar to the value of undrained shear strength ($C_u$), i.e. one half of the value of deviator stress at failure ($0.5q_{\text{max}}$). This is further discussed in Section 6.3.10.

6.3.5 Permanent Deformation Behaviour

The permanent deformation behaviour of the tested samples generally shows very small changes over the first few deviator stress increments, after which the strain starts to increase rapidly with increasing stress (i.e. above the threshold stress, see Section 2.5.6), following an exponential curve. Beyond the threshold stress the curves tended to show large increases in permanent strain for small changes in applied stress. The higher the sample water content relative to the materials plastic limit, the worse the performance observed (similar to stiffness behaviour, Behzadi and Yandell, 1996). The stiffer samples generally show less permanent deformation at a given stress, as would be expected. The variability between the performance of samples from the same site was again observed, although the magnitude of the differences observed were lower for permanent deformation since the strains measured are much larger (Section 6.3.11). Because of the variability between samples from the same site the use of one sample to collect both
permanent and resilient strain data has been vindicated. It is doubtful that comparable resilient and permanent strain data can be collected from different undisturbed soil samples.

6.3.6 Effects of Remoulding on-sample Behaviour

An improvement in sample performance over that of the undisturbed samples was found with samples remoulded at in-situ water content. They typically exhibited higher bulk densities after compaction, higher shear strengths and generally higher stiffnesses than the equivalent undisturbed samples. This improvement in behaviour was attributed to any weak zones being redistributed throughout the samples, resulting in a significant improvement in overall sample strength and stiffness, and was particularly noticeable for the Hathern (Trial 1) samples (Figures 5.11 and 5.12) which contained a number of wet silty bands. Conversely the A1(M) sample (Figures 5.15 and 5.16), which had a more homogenous clay structure, when remoulded behaved similarly to the undisturbed samples, with marginally improved stiffness behaviour. In this case the remoulded samples exhibited similar permanent strain but lower resilient strain (thus higher resilient modulus) than the undisturbed samples.

The equilibrium water content remoulded samples sustained sample strains larger than equivalent in-situ water content remoulded samples, and had similar strain levels to those observed on the original undisturbed samples, despite typically having water contents 2 to 3% higher than the undisturbed soil. The equilibrium samples produced similar results for shear strength, stiffness and bulk density to the undisturbed samples, showing better performance than might have been expected.

An improvement in material behaviour is seen from the repeatability tests, whereby the middle water content samples show higher densities than the other samples. The redistribution of water or addition of water for equilibrium samples may change the performance of the material in the compaction process during sample manufacture, and
hence result in a better performance as well as the redistribution affects discussed above (Section 2.5.5, Elliot and Thornton, 1988). This implies that a sample (when prepared by remoulding) should perhaps be compacted to a target density rather than with specific energy.

Remoulding and compaction is a quick and effective method of preparing the disturbed samples, however it causes the destruction of the soil structure. The remoulded samples showed an improvement in performance contrary to that observed by Hitcher (1996) and Lee et al (1997) (Section 2.5.5). These findings call into question the use of remoulded and re-compacted samples of undisturbed soils for predicting worst case behaviour, due to redistribution of water and weaker bands of material.

6.3.7 Conditioning

Figure 5.17 indicates a slight improvement in the stiffness of the materials following conditioning over the values seen during the conditioning phase, which is contrary to the behaviour seen by Cheung (1994). The purpose of conditioning was to reduce the cross sample variability, by improving gauge connection. However, little reduction in the variability of the stiffness readings between the two on-sample gauges was observed at low stress after conditioning, (further discussed in Section 6.3.11.1). For stress controlled conditioning, this minimal improvement will partly be due to the material properties, since different materials will sustain different amounts of permanent strain for the same applied stress. Therefore the extent of the improvement in gauge connection will vary, and was limited for the small permanent strains induced by the low stresses applied for this conditioning, therefore stress controlled conditioning was inappropriate.

The Oxford Clay (Ox 1 etc.) samples used for assessment of strain controlled conditioning proved to be the stiffest tested, and therefore the stress required to generate 1% strain was high, almost to the limits of the apparatus. The stiffness of the samples post conditioning was notably greater at deviator stresses less than 40 kPa, whereafter the
conditioned sample behaved similarly to the unconditioned samples, (Figure 5.17). It is believed that this change in performance was due to the change in sample properties caused by the permanent deformation sustained, and possibly an improvement in the connection of the gauges. However, it is anticipated that the improvement in gauge connection would manifest itself throughout the load sequence (displacing the curve to the right). However, the samples all appear to return to the line of an extrapolated unconditioned stiffness curve (Figure 5.17), suggesting it was material properties that have resulted in the improvements observed.

The permanent deformation characteristics were changed by the conditioning, with strain hardening observed over the range of the conditioned stresses applied. Minimal permanent deformation was sustained by a sample post conditioning up to the level of the previously applied maximum conditioning stress (Figure 5.18). At stresses above those previously applied, the sample sustains a large deformation to a level marginally below that which would have been observed if the conditioning test is extrapolated. This shows that limited improvement in accuracy of measurement was achieved by conditioning, yet significant changes to the material properties are observed, as demonstrated by Monismith et al, 1975.

It could therefore be argued that if conditioning (permanent strain) is induced on site then as long as stresses are maintained below those previously applied, limited further deformation will be caused. This confirms the importance of proof rolling the formation with an appropriate level of stress during construction (but at a level insufficient to generate pore pressures). These data also show the significance of consideration of the load regime when assessing data and indicates the importance of material stress history (Brown et al 1975).
6.3.8 Behaviour during Cycling

The variability at low stiffness was initially thought to be partly due to generation of excess pore water pressures within the sample during the cyclic testing, although this would not be a problem with the majority of the samples tested since the soil is only partially saturated. If the pore water pressures could not be dissipated, this would cause a reduction in effective stress and consequent reduction in stiffness. A limited investigation of pore water pressure build up during testing was undertaken by assessing performance at various stages of the 1000 cycles at each load increment from some of the initial triaxial tests.

Figures 5.7 and 5.8 show that the permanent deformations were caused by increasing applied stress, and not by a reduction in sample strength caused by the generation of pore water pressures. This would be observed if the permanent deformation increased consistently with increasing cycle number, and if resilient deformation did not remain constant throughout the testing.

Further investigation of the behaviour of the samples during cycling could be used to refine the test methodology for practical application, but is outside the scope of this research. This may allow the number of cycles applied to a sample to be reduced, and hence reduce the time required to perform a test.

6.3.9 Shear Strength

Variations in the undrained shear strength (sometimes in excess of 100%) were found for materials prepared in the same state from the same site. Whilst some variability was expected, the effect on shear strength of the cyclic loading of the samples to approximately 5% permanent strain are unknown, however Brown, (1975) showed that cycling can increase shear strength. Samples have been shown to sustain cyclic loading at levels beyond maximum deviator stress ($q_{\text{max}}$) (O'Reilly et al, 1991), and this has been
observed on some samples during this testing, although sample permanent deformation behaviour was observed to be very unstable at these stresses (Figure 5.16). It is therefore likely that the strain hardening (as observed in permanent deformation behaviour seen in the conditioning testing, Section 6.3.7), will have resulted in some improvement in the undrained shear strength measured in all the unsaturated samples. The repeatability testing undertaken shows reasonably repeatable values of shear strength (Table C5.9), implying that material variability is the cause of the varying values of strength measured on samples from the same site.

6.3.10 Threshold Stress

Various methods to predict the point where the permanent strain behaviour changes, to become unstable (i.e. the threshold stress) have been assessed. The threshold stress has been defined at an arbitrary value of 1% permanent deformation (Cheung 1994), this has been assessed for all the materials tested (Table 6.1). The deviator stress value at 1% permanent strain occurred at different positions around the curve of stress against permanent strain for different soil samples from the same site (as shown on Figure 5.11) as would be expected because each sample has its own stress strain characteristics.

However the threshold stress perhaps should be defined as a mathematical function of the shape of the stress/strain curve, since it is unlikely that a unique value of strain will define the true threshold stress for all soils, due to their differing stress strain properties. To refine this point and to allow comparison with strength, and site measurements, two further approaches have been investigated, the deviator stress at the point of maximum curvature, and modified Sherby-Dorn Plots, (Section 2.10).

The point of maximum curvature of each for the permanent strain against deviator stress plots has been assessed from the figures and correlated against shear strength (Figure 6.1). The two values of threshold stress are very similar (Table 6.1), which confirms that the 1 % strain approach is a reasonable approximation. The data for both relationships
with shear strength exhibit significant scatter about the trend line, with both sets of data showing similar trend-lines and correlation coefficients. A further line plotting shear strength against $0.5q_{(\text{max})}$ is also shown. This shows a good correlation to the other two threshold trend lines, and implies that the permanent deformation behaviour is becoming unstable at this value of deviator stress, which is approximately the start of the stiffness asymptote, (discussed further in Sections 6.3.4 and 6.8.3).

Further limited assessment using modified Sherby-Dorn plots was performed to try to assess the point at which the permanent deformation became unstable (a typical curve is presented in Figure 6.2). The samples considered herein are all fine grained soils, previous research has used this method to find the threshold stress for granular materials (Lekarp and Dawson, 1997). Compared to testing on granular material, testing of fine grained soils uses much lower deviator stresses and a smaller number of cycles. This consequently yielded two potential problems for Modified Sherby-Dorn plots. The samples assessed in this way do not sustain a sufficiently high level of permanent strain, within the 1000 cycles applied at low deviator stress to reach the turning points of either the tangent curve or the logarithmic parts of the curve (i.e. the threshold or shakedown limit, Section 2.10). Therefore it cannot be determined whether the soil is behaving in a manner below shakedown or above shakedown, (Figure 2.31). The permanent strain induced by a single cycle of load is extremely small at low deviator stress, to calculate the increase in permanent strain/cycle the average of the total change in strain over the cycles, to date during the application of a particular deviator stress was used. This further masks the patterns observed, since the increase in permanent strain is non-linear throughout the 1000 cycles of each set of deviator stress applications, (Figure 5.8 and Section 6.3.7). Therefore the strain per cycle varies throughout each period of cycling, unlike granular materials.

This initial limited assessment of Modified Sherby-Dorn plots reveals them to be unacceptable for the fine grained materials tested to date. This approach may be of more use on granular soils, although it is likely that the number of cycles employed and the
level of permanent strain induced will be insufficient within the methodologies adopted for this research.

6.3.11 Test Methodologies and Apparatus

6.3.11.1 Variability of Sample Strain Measurements

Close investigation of the data collected from the tests showed that the level of resilient strain generated in a sample at low deviator stress was typically in the region of 2 to 3 $\mu$e (approximately 0.01 mm deflection for a 200mm sample). This is divided by the applied stress to calculate the resilient stiffness. Therefore any data processing errors or movements of the on-sample gauges resulted in an error in the strain reading. This error was magnified by the calculation of the stiffness. Small errors in the strain measurement on stiffer samples, in which strains at low stress were much smaller, will consequently be magnified by a larger amount. Hence the larger discrepancies between the stiffness calculated from the two on-sample gauges for the stiffer samples (Figure 5.9) at low stress.

The stiffness results presented in Figure 5.9 and Tables C5.3 to C5.8 show the typical variability of the calculated stiffness at low deviator stress. For deviator stress applications typically above 30 kPa the typical differences between the two on-sample strain measuring devices are small. However the magnitude of this deviator stress varies from soil to soil according to its inherent stiffness (the stiffer the soil the greater the magnitude of stress at which convergence occurs). The main reason for this variability was believed to be the difficulty of accurately measuring small strain at low deviator stress. At higher stresses the magnitude of the resilient deformation was greater, and consequently the effect of a small error in strain reading on the calculation of stiffness diminishes.
The curve for resilient modulus determined from the total sample strain (LVDT) typically lies below those of the on-sample strain gauges, indicating that the strains are greater. Conventional thinking would suggest the reverse, i.e. that the strain in the middle portion of the sample would be greatest due to the lack of restraint from the load platens (Section 2.7.1.2). However, at low stress the deformation caused to the sample is very small and is extremely sensitive to areas of lower stiffness. These could occur due to disturbance at the ends of the sample during sample preparation. Sample compaction effects may also play a role for re-compacted samples, as the Proctor compaction method employed may not produce the same soil fabric throughout the sample (Mohammad et al, 1997). However, apparatus precision also plays a role.

There is no pattern to the cross-sample variability in stiffness measured by the on-sample strain gauges between samples from the same site or those manufactured by different methods. Samples that contained gravel and voids showed cross sample variability. The majority of subgrade soils tested were partially saturated, at low strains it is possible that the resilient stiffness of the samples will be reduced by compression of air in voids, or increases locally due to the effects of gravel in the sample structural matrix.

Towards the ends of the tests, at higher deviator stress, after high permanent strain, the moduli calculated from the two on-sample gauges converge to form a stiffness asymptote (Section 6.3.4). This convergence suggests that when higher strains were induced the accuracy of the data collected increases. Material cross sample variability should be independent of strain level and therefore present throughout the testing. This again implies that material performance is not the sole cause of the variability found at low stress/strain.

It should be noted that the studs used to connect the strain gauges to the sample were pushed in, which results in some local disturbance. This may result in a poor connection between the sample and the strain gauge loop. As the samples sustained permanent deformation the soil compresses around the studs and improves the fixing of the strain
gauges (Section 6.3.7). Samples remoulded at below their Plastic Limit tended to produce small voids at the edges, the shape of the discrete particles that were compacted to make the sample could clearly be seen. These small voids and particulate structure may have resulted in problems of poor strain gauge connection at low stress, at very low strains it is likely that gauge connection plays a significant role.

The repeat triaxial tests and dummy sample demonstrated that the precision of the apparatus plays a significant role in the variability at low deviator stress since very variable resilient strain data are collected (Figure 5.10), with more variation being observed on the stiffer rubber sample. The tables of precision data (Table C5.3) based on the analogue to digital converter confirm this to be the case. This may also partly explain the observation of low initial LVDT measured stiffness, as the calibration range of the device is large and therefore a small strain will be measured as one digital bit. This will correspond to a potentially larger strain than that actually found. Similar effects were also found for the on-sample gauges.

At very low strains the background 'noise' from the transducers may also play a role in the strain measurement. Figure 6.3 shows the noise from a strain gauge with cycling at 5kPa, the pattern of the cycle can not be seen. Figure 6.4 shows the same gauge during cycling at 10kPa, and clearly shows the load cycle. These problems question the data produced at deviator stresses below 30kPa for the calibration ranges used, not only in this research but in the work undertaken to produce the apparatus (Cheung 1994). Cheung used a calibration range of 2.5mm, but this still leads to potential errors in excess of 50% at low deviator stress. In addition, these issues may affect the data collected during other research using whole sample measurement or on-sample strain measurement at low strain using LVDT or strain gauge loop measurement (Section 2.7.1.2).

The appropriateness of the range of stresses applied by the triaxial test equipment and the method of strain measurement is also a further area of concern. For the construction condition the back-analysis (Section 6.8) suggests that the subgrade is loaded to deviator
stresses above 20 kPa during trafficking of a thick capping. This is at the limit of the lower range of stress of acceptably measured strains as defined above. In service the applied stress beneath a full pavement will be much lower than during the construction condition and therefore the performance measured at lower strains will be open to question, partly due to instrumentation precision as described above. However, the appropriateness of the method of strain measurement should be considered. At a measured stiffness in excess of 200MPa at deviator stress of 10 kPa and below, the strain range is approaching the very small strain range (<0.001%, Atkinson, 2000) where wave methods of strain measurement (Bender Elements) are deemed more appropriate.

The permanent deformation measurements were recorded as a cumulative total throughout the cycles and consequently the strains measured were larger. In addition, analysis of permanent deformation was not subject to the sensitivity of further calculation, hence the difference between on-sample readings for permanent strain at low stress are not as large as seen for stiffness.

Permanent strain values from the two on-sample gauges tend to show reasonable correlation until the rate of permanent deformation starts to increase at an exponential rate (Figure 5.8), at which point they start to diverge. This divergence occurs just beyond the threshold stress value. This was thought to reflect the true behaviour of the samples as the stress was increasing towards the soils shear strength, with the possible formation of shear bands near shear strength, and this may affect the deformations observed.

6.3.11.2 Radial Strains and Area Correction

To calculate the applied stress to a sample for calculating the resilient stiffness the area of the sample when compressed under the applied load is required. Therefore the radial strain needs to be assessed, and is measured using non-contact proximity transducers in the apparatus used. During the course of the tests, these radial transducers became a further source of error.
Two radial strain transducers were fitted within the triaxial cell and foil strips were glued to the surface of the soil to act as targets to measure the change in sample diameter. The range of the transducers was such that the transducers had to be adjusted during the set up procedures to read a specific initial value of proximity. When the cell pressure was applied to the sample in the test rig the foil was forced against the surface of the soil, (even though the foil was carefully fitted). This changed the initial strain reading and in some cases pushed the transducers out of scale. On some occasions, either one or both of the transducers ceased to work during the course of the tests. To compensate for the temperamental nature of radial strain measurements, assessment of the change in sample area was made using the standard fixed volume area correction (BS1377, Part 7, 1990). Comparison between the values of stiffness calculated with the two methods has been made for a number of samples at a range of deviator stresses (summarised in Table 6.2). It shows only a slight difference in the stiffness values calculated (typically less than 2MPa), which when allowing for other likely errors in the proximity transducer data, (such as those considered for the on-sample gauges) is not considered significant. Therefore, for the later tests performed the stiffness was calculated using the standard volume correction based on-sample permanent deformation alone, as the measurement of radial strains does not appear to be essential for routine testing.

6.3.11.3 Repeatability

The repeatability tests (Figure 5.19 and 5.20) show that the apparatus produces reasonably repeatable results at deviator stresses above those influenced by the apparatus precision. The soils all tend to similar values of stiffness asymptote for each group of water contents (Table C5.9) although the slight differences can be attributed to the slightly different sample water contents and the different density values (possibly due to compaction).
6.4 Field Trials

6.4.1 Introduction

In this section the construction, results and testing of the field trials are discussed and the anomalies, trends, and patterns seen from the data collected from the sites are discussed. Typically the data are grouped and discussed for the type of testing undertaken, although where specific elements relating to one site have been observed these are discussed on a trial specific basis. Initially the stiffness measurements made are discussed followed by a discussion of the devices used. The density measurements, trafficking and additional site tests are then discussed.

6.4.2 Stiffness Measurements

6.4.2.1 Subgrade Stiffness Measurements

Generally the stiffnesses measured in each bay are relatively low and show significant variability, with some readings within a bay being twice the value taken at an adjacent point. Evidence was found of a change in stiffness with slightly differing visual appearance of the subgrade on site, such that the more sandy areas of the subgrade at Trial 1 produced higher stiffness than the siltier bays. Similarly, Trial 3 subgrade contained areas that were obviously wetter than others, and hence lower stiffness was measured in these. Trial 2 subgrade contained a significant amount of gravel and showed the most variable behaviour as the interlock between adjacent gravel particles increased the measured stiffness and thus the individual results depend on the amount of gravel locally, (Huston et al 1994). The subgrade of the thicker Trial 2 bays was observed to contain more gravel, and hence this is reflected in their higher measured stiffnesses and larger standard deviation to the mean than the other bays. The variability of the subgrade readings makes it difficult to define absolute values of subgrade stiffness for any site since it shows similar variability in behaviour to that observed in the laboratory testing.
(Section 6.3.4). This suggests that the stiffness values should perhaps only be used as comparative data to judge performance on adjacent areas of subgrade.

6.4.2.2 Stiffness Measurements on Capping

The well graded cappings tended to show better stiffness and lower variability of readings due to better particle interlock (Rada and Witzac, 1981),(Section 6.4.3). No significant difference in stiffness was seen with materials of different particle size. However, differences were observed between similar materials with different shaped grading curves, (for example the Limestones used at Trials 1 and 3), implying that material grading is important, as has been observed in triaxial tests on granular materials (Rada and Witzac, 1981). However improvements in material behaviour, and stronger trends are seen in the better confined and instrumented triaxial cell compared to field measurements and hence the limited trends seen on site. This suggest that similar factors that affect granular material performance in the triaxial test affect performance on site, although the magnitude of the effect will be different.

Comparison between the Trial 2 top of capping stiffness for all data collected on various thickness layers during construction, and that from each bay on the final equivalent layer thickness of capping for Trial 2 (Figures 5.41 to 5.44), shows no significant difference in stiffness. This implies that the higher subgrade stiffness in the thicker bays has not significantly affected the top of capping stiffness. Therefore, capping stiffness may control the measured stiffness in a thicker layer due to the internal matrix of the capping, which is also observed at Trial 1 after 8 months (Section 6.4.2.4).

The thinner bays tended to show a slight reduction in stiffness from the subgrade stiffness after installation of capping. This is attributed to the effects of compaction on the subgrade, generating positive increments of pore water pressure that reduces its stiffness. Increases in composite stiffness above subgrade values were only seen with thicker capping, suggesting that devices measuring on adequately compacted thin layers are
possibly more influenced by the layers beneath, which is related to plate size and stress
distribution (Section 6.4.3). Therefore no significant changes in stiffness are witnessed
until a 300mm thick material was provided, (Figure 5.42 and 5.70), when the capping
begins to have more influence on the measured performance.

The Granodiorite capping used on Trial 3 had a low fines content. The fines interlock
between particles and therefore play an important role in stiffness (Rada and Witzac
1981), thus low capping stiffness accounts for the low stiffnesses measured,
(subsequently confirmed by back analysis, Section 6.8). No significant change in stiffness
of top of capping with increasing thickness was observed for the Granodiorite at Trial 3.
This suggests that the capping stiffness itself, rather than the effects of the sub grade
beneath, are controlling the measured stiffnesses. This would appear to confirm the
effects described above, although near the surface confinement of the material will be
lower and this may also affect the deflection measured, (used to calculate stiffness). If the
sub-base is considered as an additional layer of capping, then the maximum thickness of
granular material used in Trial 3 is 600 mm. There is no significant increase in stiffness
between the 300 mm (+ 150 mm sub-base) and the 450 mm (+ 150 mm sub-base) bay
since the limited improvement in stiffness occurs with the provision of a thin layer on top
of a low stiffness formation (Figure 5.41 and 5.42).

With additional compaction (Figure 5.59), the Trial 3 Granodiorite stiffness was lower
than after the initial five compaction passes. An element of over compaction was thus
observed. This is seen to affect the thinner bays more than the thicker (i.e. the influence
of the subgrade and hence the affects of over compaction on the composite measured
stiffness reduce with increasing thickness). This may also account for effects previously
described for the thinner bays of Trial 2, (Figure 5.41 and 5.42).

The Trial 3 Limestone capping initially exhibited much better particle interlock and
significantly increased stiffness with increasing capping thickness than any other material
assessed, (Figure 5.70 and 5.71). The replacement Limestone layer showed very low
stiffness (Figure 5.72) which can perhaps be linked to the performance of the poor (remaining Limestone) supporting material, and to the limited influence of the top replaced layer after the poor weather. Again the Limestone demonstrated that the measured stiffness achieved was not sensitive to the amount of compaction applied to each bay, but to the thickness of the material provided (Figure 5.72 and Section 6.5).

6.4.2.3 Stiffness Measurements on Sub-Base

The measurement of stiffness on the sub-base compacted upon the cappings at Trials 2 and 3 shows no significant increase in stiffness with the number of passes of the compaction plant similar to capping (Figure 5.46 and 5.59). However, a strong trend of an increase in stiffness with thickness of supporting capping is found, (Figure 5.46). The stiffness on top of sub-base generally shows an improvement over the value measured on the top of capping, except for the thinner bays which is again perhaps related to the thin layer effects previously described. The sub-base shows a greater increase in stiffness on the thicker and stiffer materials, implying either that the subgrade stiffness still influences the measurement, or that more of the supporting capping contributes to the measured stiffness. Nevertheless, the increase can be attributed to the reducing effect of the subgrade improving the composite stiffness measured.

For Trial 2 there is little contrast in the improvement in the composite stiffness provided by the addition of sub-base compared to that for the addition of capping of similar thickness, (e.g. comparing a 350 mm (interpolated) capping stiffness with 200 mm capping plus sub-base value, Figures 5.43 and 5.46). This implies that either the sub-base has a similar stiffness to the capping, or that the thin layer effect is again observed and the sub-base has minimal effect other than allowing more capping to influence the measurements. The grading curves of the capping and sub-base at Trial 2 are very similar, which may account for this similarity in stiffness, however this does not manifest itself in the back analysed stiffness, (Section 6.8). This implies that the capping still controls the stiffness, and that the affect of the subgrade has been further diminished.
The small difference between the completed stiffnesses on the Granodiorite sections at Trial 3 across all of the bays can possibly be attributed to the stiffness of the Granodiorite capping controlling the composite stiffness. The material was very loose therefore the majority of the deformation induced during the loading will be contained within the granular materials (Figure 5.39). The small changes in stiffness can be attributed to the limited effect that the subgrade has on the stiffness measurement, together with the limited effect of the sub-base in the longer-term.

The sub-base compacted upon the replacement Trial 3 Limestone materials showed no significant change in stiffness with thickness or compaction, with the thickest bay showing a slight reduction (Figure 5.70). The minimal changes and low stiffnesses are directly attributable to the magnitude of the capping stiffness after the rain and subsequent reconstruction, again little improvement in composite stiffness is found with an addition of a thin layer. The apparent reduction in stiffness in the 450 mm bay might be attributed to the generation of positive pore water pressures within the saturated capping.

6.4.2.4 Stiffness Measurements after Trafficking and Long-term Monitoring

The majority of points at Trial 1 showed an increase in stiffness following trafficking (Figures 5.28 and 5.30). This may be partly due to an increase in subgrade and capping stiffness due to drying. However it is more likely to be due to densification of the capping during trafficking (Section 6.4.7), indicated by the bays that received least compaction showing the greatest improvement in stiffness following trafficking (Figures 5.28 to 5.30). This in turn can be related back to the density of the materials, which is generally lower in the bays that received less compaction, especially the Porphyritic Andesite bays, (Figure 5.25).
A short period of wet weather occurred during the trafficking phase, and this may have assisted in the densification and hence improvement in stiffness. The substantial increase in the stiffness of the Trial 1 Limestone bays was influenced by cementing of the Limestone particles over time, (Figure 5.29) which again was probably aided by this ingress of water.

The further increase in stiffness of Trial 1 up to 8 months after completion, above the readings taken after trafficking, is surprising since the poor weather during the winter was anticipated to result in a reduction in the subgrade stiffness, and consequently in the composite trial stiffness. An element of aggregation of particles was seen to occur, thus lower stress will be transmitted to the subgrade via a stiffer capping, therefore the subgrade stiffness is higher, (as observed in the triaxial tests at low stress), thus an increase in the composite stiffness is possibly measured.

There is a slight increase in variability in the stiffness measured after trafficking. This increase is especially large with the TFT (although the results still show a significant increase in stiffness). This change in variability may be due to two reasons. Firstly, the contact of the plate with the ground is an important influence on the magnitude of the dynamic stiffness measurements, and a loss of fines may influence near surface effects (Section 6.4.3) and was observed to have occurred on all the materials at Trial 1. Secondly, the Limestone exhibited the greatest variability over the long-term, and this may be due to the varying degree of cementation between particles that occurred.

Significant improvement in stiffness only a few days after completion of construction of the sub-base is seen on Trial 2, and indeed in some bays a 100 % increase is observed (Figures 5.43 and 5.44). This may be due to the dissipation of the effects of compaction. For instance the Trial 2 bays received 11 passes of the vibrating roller over the last two days of construction, during compaction of the final capping layer and sub-base. This may have generated excess pore water pressures in the subgrade due to the high compaction stresses, which then dissipated over the course of the rest period. The dissipation of pore
water pressures in the surface of the Trial 2 clay subgrade is likely to be relatively quick due to the gravel content of the clay. However, the improvements (only measured with the GDP) over the first four days seem to occur quickly (Figure 5.43) and are not seen with the FWD data compared to the TFT, allowing for their differences, (Section 6.4.4).

The Trial 2 stiffness increases further up to the time of the trafficking phase, (Figure 5.44) but the increase at the trafficking phase over the values taken a few days after construction is not large. An element of aggregation of the particles may have contributed to the observed improvements in stiffness in the long-term, as well as possible thixotropic effects in the subgrade improving response in the thinner bays (Seed et al, 1962).

Aggregation is evidenced by the fact that the compacted Trial 2 materials could initially be dug with a garden fork, yet at the time of excavation following trafficking the trial bays could only be excavated with a pick axe, this corresponds to the findings at Trial 1. Further changes up to 8 months after construction are again seen, but are smaller than those observed up to the time of trafficking, suggesting that the winter weather may have played a role. The subgrade may have deteriorated during the winter months, resulting in slightly poorer performance, but the improved capping performance negated these effects resulting in the further slight increase found.

Trial 3 shows the most significant changes over time with the limestone showing very poor performance in the short-term followed by large improvements in the long-term (Figures 5.72 to 5.74). The Limestone capping shows a great improvement over the values measured on the replacement material in the long-term, which can only be attributed to particle aggregation (Figure 5.73). The limestone removed (to allow its replacement) was dumped on site was of the consistency of toothpaste when removed, yet it had cemented overtime to form a brittle yet stiff matrix. The Granodiorite trial showed limited improvement in the longer-term and this is again attributed to subgrade thixotropic effects, as no aggregation of the materials was observed on site.
6.4.2.5 Weather Effects on Measurements

The dry crust, formed quickly on the subgrade of Trial 1 during the warm weather after the subgrade was exposed. The effects of the crust alone do not appear to have effected the subgrade stiffness measurements, which showed the subgrade stiffness to be broadly similar along the trial. This is to be expected, since stresses applied by the dynamic stiffness measuring devices directly to the crust will be transferred via the stiff, fractured and cracked surface, through to the soft materials beneath, rather than acting as a homogenous layer. This again shows the thin layer effects previously described for capping. Once capping is placed, the dry subgrade surface may have more of an effect since the stress applied to the subgrade will be greatly reduced, and the stress-dependency of the subgrade and near surface effects from the crust may contribute more to the composite stiffness measured. However, the crust itself should have little effect on the overall strength (rutting) performance of the subgrade under construction traffic.

Conversely, the effects of poor weather rendered the Limestone at Trial three useless due to its water absorption. It is unlikely that provision of drainage would have prevented this damage as the water was held within the fines of the material.

6.4.2.6 Effects of Single and Three Stiffness Tests at a Point

Taking more than one stiffness reading at each point with the dynamic stiffness measuring devices was expected to cause an increase in stiffness, since the material under test will have received slightly more conditioning (Section 6.3.7). This may induce an element of permanent deformation (and densification) that would change the material stiffness. It was anticipated the low contact stress applied to a stiff material would have a lesser effect than on a material of lower stiffness. Thus it would be expected that the data for three tests at the same point would be stiffer than that for the single tests at adjacent points, and that this effect would be more evident in the bays receiving fewer compaction passes.
Neither the FWD nor the GDP show any difference between the results for single and three readings on subgrade. This again is to be expected since the clay will not undergo densification in the same way that a compacted granular material would due to packing and compaction effects.

Due to the variability of the readings taken with the devices no statistically significant differences between data at multiple test points was seen over the low applied stresses used to measure stiffness on site (Section 6.3). Repeat FWD measurements on Trials 2 and 3 (where the higher applied stresses were used followed by a repeat test at the lowest applied stress) showed no changes due to the previously applied higher stresses. This concurs with the findings from triaxial tests (Section 2.5.5), which showed no changes in stiffness of granular materials with stress history (Brown and Hyde, 1975). This is somewhat surprising since the devices measure both resilient and permanent strain on loading, which if densification occurs will manifest itself in a lower measured stiffness. This suggests that the deformations are all recoverable, which may be due to the rate and magnitude of the loading being well below the threshold strength, (Selig, 1987, and Boyce, 1978).

6.4.3 Analysis of Stiffness Measurements and Variability Due to Dynamic Plate Devices

6.4.3.1 Contact Effects

The dynamic stiffness tests rely upon a good contact between the plate, the transducer and the ground, in order for an accurate reading to be obtained. Errors may therefore be incorporated if good contact is not achieved. In addition, the particle size of the material being tested may influence the result, particularly in the cases of the TFT and FWD, since they measure deflection directly via a small foot that bears onto the ground, unlike the GDP which measures plate deflection. This was demonstrated at Trial 2, with the influence of the gravel in the subgrade being apparent (Section 6.4.2.1).
Testing on bays that received limited compaction showed greater variability in individual readings, which may be due to looser particle packing and particle size effects at the surface causing the surface to vibrate under test. This would account for the reduced variability with greater levels of compaction and smaller particle size (Section 6.4.2). Generally the variability is seen to reduce with increasing thickness (at construction stage). This suggests that the more regular and well graded the material, the better the interlock, the better the particle contacts and the better the plate contact, which results in more repeatable results.

6.4.3.2 Plate size

Plate size has been shown to play an important role in the assessment of shallow stiffness such that a plate only influences the ground from 0.5 diameters to 3 plate diameters (Fleming, 2000). This may account for the low change in stiffness with the addition of a thin layer (Section 6.4.2.3).

6.4.3.3 Scatter

The standard deviations of the stiffness measurements from the data in each bay show the GDP to exhibit generally less scatter between readings than the TFT and FWD. For both the capping and sub-base the GDP shows approximately half the standard deviation observed with the TFT. The standard deviation with the GDP is typically in the range 10 to 15% of the measured mean stiffness, (Coefficient of Variance, COV) although values of up to 30% are seen. The TFT and FWD, have COV ranges from approximately 10 to 20%, with peak values up to 50% being observed. The materials are known to be variable, and the degree of scatter in the readings should not therefore be interpreted as a problem that is solely attributable to the measuring devices.
The devices all utilise simple static elastic theory to interpret an elastic stiffness modulus from the measured (or assumed) values for contact stress and indirectly measured deflection. Dynamic (inertia) effects are therefore not taken into account. Since the measurements are made either on the plate, or on the ground directly beneath the plate, the response of the underlying (interacting) layers is thus superimposed to produce a single surface deflection value. The stiffness is therefore a composite elastic stiffness modulus. The surface deflection is however a complex function of the stress and hence strain distributions within the layers (Section 2.9). Their interaction is further complicated by the stress-dependent behaviour of the materials. Variations in the applied stress and load pulse duration will also contribute to the differences in the results.

Significantly the GDP assumes a constant contact stress of 100kPa. This is thought to be a source of the error observed, since the applied stress is dependent on the stiffness of the material under test. However the GDP signal processing (i.e. smoothing, digitising and interpretation) is believed to be responsible for the lower scatter seen in comparison to the other devices.

6.4.3.4 Rubber Tests

The different response measured by the FWD on the rubber (Figure 5.84) may be due to the large static pre-load that it applies. When the trailer is positioned, the hydraulic rams lower the bearing plate and, to ensure good contact and stability during testing, partially lifts the whole trailer assembly (which weighs several hundred kilograms), thus statically loading the material under test. Therefore the rubber is being pre-stressed by perhaps 50kPa or more before the additional transient stress is applied. This is thought to have contributed significantly to the high rubber stiffnesses recorded by the FWD, and may also influence its response on less stiff materials.

The TFT stiffnesses for two and three layers of rubber are much closer to the GDP stiffness and show no stress-dependency, as expected. The single layer results show that
an increase in stress from 40 to 120 kPa cause an increase in stiffness from 40 to 120 MPa, which suggests that the deflection remained constant. This may also have been caused by the interaction of the rubber and the velocity transducer foot, which is free to move within the protective housing, without restraint (i.e. it can bounce). This may also explain the slightly larger variability seen with TFT data when compared to the other devices. The FWD transducers are restrained by springs to ensure that they remain in contact with the material under test.

The limited scatter observed by the devices on the two and three rubber sheet tests show that the tests are repeatable (Section 5.4), and it is material variability and contact effects that play the most significant role in the variability seen.

6.4.4 Correlation between Stiffness Measuring Devices

Stiffness data are typically plotted against those measured by the FWD, since this is the most widely accepted device for measuring stiffness. Scatter graphs of the TFT and GDP stiffness data at 100kPa contact stress have been plotted against FWD data. Where FWD data is not available the TFT and GDP have been compared. Figures 6.5 and 6.6 show typical correlation between devices on subgrade capping and sub-base. For Trial 1, coefficients are compared for single and multiple tests at a point and combined data. Due to the small sample sizes, all the data from the three cappings have been combined to give an average value. The lines of best fit to the data are forced through the origin and the correlation coefficient for the data are calculated using least squares linear regression. The $R^2$ values and correlation coefficients are presented in Table 6.3.

The correlations observed were generally not strong ($R^2$ of $\pm 1$ being a good correlation) for any of the trials. The GDP shows better correlation coefficients than the TFT, due to the higher scatter of results observed with the TFT. There was no discernible pattern to the correlations, with both good and bad correlations observed for all materials at single and three test points for Trial 1, and for combined data from the other sites. It is likely
that the low stiffnesses measured on the subgrade and the greater variability of subgrade stiffness contribute significantly to the poor correlations, and hence the differing coefficients that were found between sites.

The correlation coefficients on subgrade for both the TFT and the GDP in Trials 1 and 3 are lower than those observed by others. However, the capping and the combined capping and subgrade combined correlations are similar to those seen by Shahid et al., (1997) (see Section 2.9.1).

When the data for subgrade and capping are combined the correlations improve. This may be partly due to the increase in sample size, and range of stiffness assessed. Forcing the lines of best fit through the origin distorts the correlation quite substantially on some plots, especially those on subgrade where more data was grouped at lower stiffness (Figure 6.7).

The GDP measurements were concentrated about a value of approximately 10 MPa for subgrade, indicating a possible lack of discernment in the GDP measurement. In addition, it should be noted that the lower limit of accuracy for the GDP is quoted in the handbook as 12 MPa.

The GDP gave correlation coefficients in the range 0.43 to 0.748 with the FWD, with the majority in a band from 0.48 to 0.6. The TFT was found to correlate more closely to the FWD, with a range of values from 0.78 to 1.40, with the majority lying between 0.89 and 1.06.

Since the capping correlations include the measurements taken as the capping layers were constructed upon the subgrade, a number of the readings will be significantly influenced by the subgrade stiffness beneath. The correlation coefficients tend to be better with increasing thickness of construction. This partially explains the greater scatter observed for the capping readings when compared to the sub-base readings especially at low

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stiffness due to the influence of subgrade. However, a significant degree of scatter was found, generally at higher stiffnesses (where a small strain is measured) due to the magnification of small errors in measured deflection in stiffness calculation caused by contact effects.

Factors such as the thickness of materials being assessed, the nature of the materials being tested and the nature of the devices themselves, (influenced by factors such as variations in the applied stress and load pulse shape, the deflection measurement transducer type and position), will affect the correlations observed. For example, the equivalent correlation coefficients (GDP to TFT) on the subgrade and Limestone 6F2 capping at Mountsorrel (Trial 3) were 0.567 ($R^2=0.38$) and 0.673 ($R^2=0.87$) respectively in comparison to 0.551 and 0.602 for Hathern (Trial 1).

It is clear therefore that no unique correlation factors will exist between devices, and that the correlations will be significantly site specific. Generally similar patterns and trends of stiffness are seen between readings with different devices on the same sites. However, although the correlation coefficients are useful for comparing sets of data, it is suggested (for the reasons described above), that applying them directly at individual points is inadvisable.

6.4.5 Density Measurement

6.4.5.1 Capping Density Measurement

In general the finer cappings shows much smaller variation in NDG density as the particles can pack closer together with more consistency. All data show small scatter in terms of the overall density measured. However, if the scatter is considered as a percentage of the change in density measured, between bays or series of tests then large variability is seen, which implies limited significance in the changes in density observed.
Due to the hot, dry weather during construction the Trial 1 aggregate materials dried. As a consequence the cappings were placed and compacted at below optimum water content (Table C5.1). This will have reduced the maximum density that could be achieved on site, and hence contributed to the greater variability in density measured (Figure 5.32). The Trial 3 Granodiorite capping had a low maximum dry density due to the low fines content, which prevented the material from packing closely and this accounts for its scatter.

For this research density targets have been of limited use in predicting rutting since the majority of materials have achieved target density values (based on the TRL values in Section 2.9.2, Nunn et al, 1997) and have nonetheless rutted (i.e. the Trial 3 materials). The fine graded material (6F1) at Trial 1 did not achieve target density but it did not rut significantly. This demonstrates the limits of using density measurement as a construction control for granular materials. However, no other simply measured parameter that can assess adequate construction is currently available.

For the fully compacted cappings there was no significant change in density with increasing capping thickness or stiffness, (Figure 5.45). This implies that the thickness and stiffness of the founding layer does not significantly affect the density of the subsequently compacted material (Figure 6.8). These results imply that the density in situ is controlled solely by the properties of the material being compacted and the compaction it receives, and not by the properties of the substrate upon which the materials are being compacted. Essentially, the results show that the more compaction applied, the denser the material becomes, until the maximum achievable density for the type of compactive effort applied for the material has been achieved. This contradicts Thom (1988), who measured the density of materials compacted on to substrates providing various levels of support (Section 2.5.7, and further discussed in Section 6.5).
6.4.5.2 Sub-base Density Measurement

The sub-base showed a similar pattern of change of density to the capping, i.e. the density increased with increasing compaction pass, and not with increasing stiffness/support from the underlying layer, (Figure 5.48). More variation in density was observed on the completed sub-base than was observed on capping, but there was no regular pattern to this along the trials, and again it is small in comparison to the values measured but large relative to the changes seen, similar to capping.

Typically, the stiffer the initial base upon which the sub-base was compacted the smaller the increase in density with additional compaction. High density was achieved over the initial passes. The thinner bays (regardless of the capping substrate) show a more progressive change in density over the initial passes. After completion of compaction, all the bays on their respective trials had similar densities, regardless of the rate at which these densities were achieved. This again suggests (similar to capping), that there is a maximum value of compacted sub-base density that can be achieved on site with the given type of compaction, and that this density can be achieved as long as sufficient compaction is applied, confirming the current method specification approach (Section 2.4.1).

Since density is the only measure currently available to assess adequate compaction, if target density can be achieved regardless of the stiffness but by sufficient compaction, then the requirement of a necessary stiffness to allow adequate compaction of the subsequent unbound foundation layer is called into question. However, a target may still be required for satisfactory compaction of the subsequent structural layers. (This is further discussed in Sections 6.6. and 6.7).
6.4.6 Deformations during Compaction

Trials 1 and 2 showed minimal deformation during the compaction, little trimming of the surface was required to maintain level tolerance, and no problems were encountered during construction.

During compaction of the Trial 3 sections, the relative mobility of the Granodiorite capping became apparent. As the roller passed over the material it created a large “bow wave” in front of the roller, which resulted in ripples in the surface of the capping along its entire length in all bays regardless of thickness. The effects of the “bow wave” were particularly apparent on the initial layer of capping, where the subgrade was more affected by the compaction (Dawson, 1997, Section 2.5.7). Although the material was trimmed and the surface re-profiled at interim stages during compaction, the maintenance of acceptable thickness tolerance could only just be achieved. Similar effects were exhibited by the Trial 3 Limestone capping during its compaction in the initial 150mm layer, indicating the influence of the subgrade. Once the second layer of capping was placed, no further “bow wave” was apparent during compaction. When the replacement Limestone was placed, a ‘bow wave’ could be seen all along the trial, caused by the affects of the poor material beneath, and its poor support.

When the sub-base was compacted upon the Granodiorite capping no bow wave or surface deformation could be seen. This is attributed to the additional confinement of the Granodiorite provided by the sub-base, improving its performance. A significant bow wave could be seen during the compaction of sub-base upon the replaced Limestone. This resulted in a significant unevenness in the surface being produced, and a step forming at the overlap of the roller runs (Figure 6.9), caused by permanent deformation generated at the edges of the roller, directly attributed to the poor support provided by the underlying layer.
Although the material was adequately compacted, sufficient resistance to deformation (both resilient and permanent) is required of a material under the high compaction stresses applied, to ensure that the materials can be constructed within the level tolerances required, even though the target compaction parameters may be achieved.

6.4.7 Trafficking and Rutting

The initial rut formation was seen to be variable for all trials and is attributed to densification and ‘tightening’ of the surface of the capping materials as larger ruts formed in the materials that had received the least number of roller passes (and had lower densities) (Section 2.5.7). It can be clearly seen from the trials that the initial rut generation occurs quite readily, (Figure 5.49) slowing with increasing number of lorry passes.

This implies that ensuring adequate density could be a means of minimising the potential rutting within a material as long as that material has sufficient internal stability when compacted. Additional compaction may also result in better particle orientation that will be able to better resist applied rotating stresses (wheel loads).

Some of the variations in rut depth during early trafficking were possibly caused by material movements at the surface, induced by shear stresses applied by the passage of the lorry wheels, due to lack of surface confinement. The inaccuracy of monitoring very small changes in level (using a staff and level) will also have contributed to these apparent inconsistencies. The materials which showed the most movement of this type were the two 6F2 coarse cappings at Trial 1 (Figures 5.33 and 5.34). This is probably due to the size of the particles and the ease with which they can rotate within the confinement provided by the adjacent materials. The larger particles in the 6F2 capping provided a more open texture in the surface of the road, and movements of larger particles in the 6F2 cappings resulted in a larger level change. The well-graded fine materials at Trial 1 and 2 (Figures 5.35 and 5.52) showed little surface movement of the particles, since the higher
fines content of the material appeared to close and fill the gaps around the coarser fraction and thus restrict their movement at the surface.

The magnitude of rutting measured on Trial 1 was lower than was originally expected. Firstly, there was an increase in the stiffness of the subgrade surface during the warm weather. It is believed that initiation of rutting at the base of a (competent) granular layer occurs at the interface between the granular materials and the underlying subgrade, as usually the subgrade is weak and will undergo local shearing. Surface desiccation and strengthening of the clay may have inhibited such movements taking place (Section 6.4.2.5). All the rutting occurring on Trial 1 formed basic depressions within the surface of the material, with no shoulders forming, implying that the ruts were of Mode 0 (Dawson, 1997, Section 2.5.7), suggesting the rut to be confined within the granular material.

For Trial 2, the magnitude and nature of rutting observed in the thinner bays suggests that the subgrade was controlling the rutting (Figure 5.52) (Mode 2, Figure 2.19). The middle thickness bays initially showed Mode 0 rutting whereby the rut formed without shoulders but then as rutting progressed shoulders formed, (Mode 1, Figure 2.18). The capping then thinned as the subgrade started to control rut formation (Mode 2). It is likely that the high subgrade deformations under load resulted in a reduction of capping stability (Fleming and Rogers, 1995) and hence the subsequent formation of the subgrade rutting. In the thicker bays the limited early rutting is attributed to the internal stability of the capping (as seen in the PSSR section 6.2.1.2) and a lack of the influence of the subgrade with Mode 0 ruts forming, propagating to Mode 1. The pattern of rutting implies that a combined thickness of capping and sub-base of approximately 500 to 550 mm will sustain the trafficking adequately, whereas 450 mm was adequate for 100 to 150 lorry passes only.

The level of rut transfer to subgrade on Bardon (Trial 2) varied from bay to bay and was difficult to evaluate successfully, since small ruts/wide depressions in the subgrade could
not easily be identified. The effect of compaction of granular materials upon the subgrade caused some of the material from the lower layer of capping to penetrate the subgrade and this, combined with the natural gravel content in the subgrade, obscured the subgrade rut transferred, i.e. reduced accuracy of measurement. Approximately 50% of the surface rutting on Trial 2 in the sub-base transferred through to the surface of the subgrade.

The behaviour of the material under trafficking at Trial 3 was adversely affected by the weather. It was anticipated that the Granodiorite capping would rut after very few passes of a laden vehicle, since the surface of the material behaved in a very mobile fashion and moved easily underfoot due to its lack of fines, showing the influence of grading. The ruts were formed by displacing material from the wheel path to form shoulders adjacent to the wheel paths (Mode 1) in all bays (Figure 5.65). It is believed that for the thicker bays the ruts were confined within the capping material. However due to the magnitude of rutting it was anticipated that in the thinner bays the subgrade would have sustained damage, although this was not quantified (Figure 6.10).

It was anticipated that the Trial 3 Limestone capping would not undergo significant rutting when trafficked, since Trial 1 had shown very little rutting on similar Limestone 6F2 capping. However trafficking proved impossible, due to the weather induced degradation (Figure 5.77).

Relating the rutting performance to the PSSR results (Sections 5.2.1.2 and 6.2.1.3) shows that the materials should all have performed adequately, if sufficient thickness was provided. The materials at Trials 1 and 2 did perform adequately, however, this proved not to be the case for the capping materials at Trial 3 due to the poor grading of the Granodiorite and the influence of the weather on the Limestone. This again suggests the drawback of the PSSR tests for assessing capping materials, however the assessment of sub-base does still appear suitable. Therefore in the absence of a suitable laboratory tests or a predictive method to assess the rutability of materials it is suggested that a field trial
utilising trafficking with a suitably laden lorry is required to assess likely rutting performance of materials.

Currently the control for design purposes for permanent deformation (Powell et al., 1984) is to limit subgrade rut, by limiting the surface rutting and hence its transference, yet this transference has proved difficult to quantify. Each material (subgrade and capping) will behave differently under trafficking due to its mineralogy, grading, load sustained, and composite behaviour, as shown in the previous sections. Since different materials show different rut susceptibility, the level of rut transfer to the subgrade will be different for different capping-subgrade combinations. Therefore it is difficult to set a precise rut limit at the surface. No model currently exists to predict rut formation, so it is clear that further research is needed into the control of rutting and formation of sub-surface rut. If direct measurement of sub-surface rutting can be developed then the monitoring of sub-surface rut formation by direct measurement could be used to limit the amount of trafficking in order to control the rut until predictive methods are available.

6.4.8 DCP Tests for Strength Assessment

There was a slight variation in resistance to penetration between the series of DCP readings taken upon completion of construction of the capping and those taken in the longer term (which usually include the sub-base layer). For Trial 2 this change was initially an improvement probably due to the additional confinement provided to the capping by the sub-base (Figures 5.49 and 5.50), and possibly contributed to by the additional compaction of the underlying materials during the placement of the sub-base. In the long-term a reduction in the cappings resistance to penetration was seen which was probably due to the water ingress that had occurred but this did not manifest itself in the stiffness measurements made (Figure 5.51). The sub-base and subgrade showed no change in penetration rate over time and the sub-base typically showed a lower rate of penetration than the capping, probably due to the thin layer and the lack of surface confinement. Equating DCP to CBR of the capping at the end of construction shows CBR
values below the 15% suggested as acceptable by Powell et al, 1984. However this is achieved after confinement following construction of sub-base. The sub-base at no stage during the trial had a CBR of 30% (an acceptable value also proposed by Powell et al, 1984).

The Trial 2 subgrade shows variable DCP results attributed to the gravel in the subgrade, although the surface of the subgrade shows a slightly higher penetration (poorer performance) in the long-term at its interface with the capping, which can be attributed to slight softening caused by water ingress.

For the Trial 3 materials the Limestone shows the most significant changes over time. The material could not initially be penetrated due to particle size etc. However, after the rain it could easily be penetrated (Table C5.37), and a further slight reduction in its resistance was then seen with the addition of sub-base following re-construction of the top layer, again attributed to water ingress and the affects of the water remaining within the lower capping layers. With time the material gave very low rates of penetration, which is again indicative of the cementation of particles. The sub-base follows a similar pattern, with a small improvement in its performance in the long-term again possibly attributable to cementation. The subgrade shows little change in penetration resistance over time indicating that the capping alone was affected by the weather, and little softening of the subgrade occurred. The capping target CBR of 15% (Powell et al, 1984) was only reached in the long-term and the sub-base never attained the 30% CBR value.

The Trial 3 Granodiorite shows similar patterns to the limestone although initially it could be penetrated due to the low fines and smaller particle size, which resulted in a matrix that could freely allow particles to move, hence enabling the DCP to penetrate. In the longer term it showed little change during construction and with the extra confinement provided by sub-base, further indicating the materials internal instability. A slight improvement was observed in the long term (possibly due to aggregation), which resulted in the capping CBR being above the 15% value. The subgrade shows a low
penetration resistance and showed no changes with time. Similar behaviour was observed for the sub-base implying that the lack of confinement and its thin layer will be insufficient to allow aggregation improvement to occur as the particle are always free to move within the sub-base matrix. Again the sub-base target of 30% CBR is not achieved at any stage.

Across the interface between the different materials a progressive change in penetration gradient can be seen. The length of this change is a function of the relative resistance of the materials being tested. It typically occurred over 100mm which may also explain the poor performance of the thinner layer of sub-base, since the material is only starting to mobilise its full resistance towards the base of its layer (due to lack of surface confinement) which leads to the lower penetration. This will also explain the lower capping resistance at the subgrade capping interface, where the two materials are noticeably different.

Generally the rate of penetration for the capping was similar across all bays of the respective trials, regardless of the stiffness and thickness of the supporting material. This concurs with the density results, where all bays show similar values of compacted density regardless of thickness and stiffness of support.

From the above the DCP (whilst being a good indicative test) can clearly be seen to be layer specific. Its inability to assess composite performance is a severe limitation. The test was observed to have severe limitations for assessing granular materials. Isolated large particles have a significant affect on the DCP measurements within the finer materials tested and the problems are more apparent for the coarse materials, as large particles must be fractured (where they can be penetrated) to enable further driving, significantly effecting the results.
6.5 Relationship between Stiffness Density and Compaction

The literature suggested that adequate stiffness of support is required to achieve adequate compaction of materials (Section 2.4). In this research, adequate density of the overlying layers of compacted materials could be achieved regardless of the level of compaction applied or the stiffness of support for the materials for the methods used (Section 6.4.5). However, Trial 1 did show slight reduction in density with less compaction, although this was limited, and was probably due to compaction effects as a light roller and hence thin layer were used with a large particle size material.

If compaction changes volume, then it will change density. If density does not significantly affect stiffness (Section 2.5.3), then no change in the composite stiffness of the material should occur with compaction, assuming adequate density was achieved after initial compaction. However, the stiffness of the trials has been shown to be influenced by the addition of more capping since the effects of the lower layers are reduced (Section 6.4.2).

The lack of a change in stiffness with improving compaction confirms the findings of Selig and Brown, (1991), who showed that the stiffness of laboratory compacted samples were not significantly affected by changes in the materials density. Young, (1992, as Reviewed by Lekarp et al (a) 2000), showed little change in material stiffness at compacted densities above optimum, indicating that resilient modulus is not particularly density sensitive. The thin layer effects discussed in Section 6.4.5 may contribute to this implying that the top layer has limited affect on stiffness. It therefore follows that limited change in composite properties are measured with improved compaction of that layer.

In addition in well compacted materials the number of particle contacts will be greater within a system (Lekarp et al (a), 2000). This results in a better transfer of stress at a lower magnitude between the particles. This will have two effects. The resilient deformation of the materials should be less, hence a higher stiffness will be seen.
However, the better the load spreading, the lower the applied particle stresses within the layers, the lower the measured stiffness of the capping materials due their stress dependant nature. These two effects may to some extent cancel each other out. Adequate density should therefore ensure that the materials are adequately compacted, and that no further particle movement are able to occur under load, which could result in a high transient deformation or further densification. However, a well-compacted angular material will show a better ‘knitted’ matrix, which will restrict internal particle movement/reorientation under rotation of principal stresses. This knitted behaviour will not manifest itself as a change in volume and hence produce improved density or resilient performance, for the reasons described above. However, it may lead to improved strength/permanent deformation related response. This may account for the maximum values of density achieved on site and the limited further changes with compaction, but may also contribute to the different rutting performance seen in less well compacted materials, although low confinement at the surface will also play a role. This is confirmed by Semmelink (1995), who showed improved CBR with improved density, but not stiffness, perhaps due to an improvement in the strength performance, but not resilient response of materials.

The limited differences in compacted density found with stiffness/strength of support contradicts the findings of Thom (1988), and Valeux and Morell (1980) all of whom showed improved density on subgrade with higher CBR. This could be due to a number of reasons, Thom (1988) measured on site density using the NDG back-scatter method. This is less accurate than the direct transmission mode (Section 2.8.4) used in this research, and therefore more likely to be influenced by near surface affects and lack of surface confinement. Both studies referred to used thicker placed layers in their trials (range 200 to 300mm), and this will therefore influence the response to compaction, due to the reduction in applied stress in the materials at depth. Thom used a sub-base layer of 240mm and 6 passes of a Bomag BW213 roller. The MCHW (Table 8.1) requires 8 passes with this roller on a maximum of a 225mm layer of sub-base, although 250mm of capping material can be compacted in accordance with the MCHW (Table 6.4) with 10.
passes of this roller. Therefore, the influence of a stiffer substrate on the density achieved lower in a thick layer may be greater for a partially compacted material, as observed at Trial 2 during compaction (Section 5.5.4). The materials used in this research have typically been compacted with methods that correspond to the compaction requirements of MCHW (1994). The effects of the higher compaction stresses may have a greater influence at the base of a thin layer and potentially the layer below, (as evidenced by the roller bow waves, Section 6.4.6). Therefore the influence of support on the full compaction of a thin layer appears limited, yet the influence of greater material thickness more significant. It is suggested that on less stiff materials the effects of compaction lower in the layer will be reduced in the thin layers being compacted, therefore leading to the more progressive changes in density observed.

No target stiffness value for top of capping can be established from this research (for the granular materials assessed) which would insure that the next layer of granular material can be adequately compacted, (Section 6.4.5). However, it is likely that the construction of the subsequent bound structural layers above will be influenced by the stiffness of the layers beneath and target stiffness values for achieving adequate compaction should therefore be set from the top layer down.

6.6 Correlation of Laboratory to Field Data

6.6.1 Correlation of the Triaxial Stiffness to Stiffness Measured with the GDP

In order to assess a correlation between stiffness in the laboratory and in the field, GDP stiffness measurements at the U100 locations prior to sampling have been compared with the triaxial stiffness at a comparable applied stress (q) of 100 kPa. Insufficient data from the other two devices used in the research (the FWD and TFT) are available to produce satisfactory single point correlations. The GDP measures the acceleration of the plate and from this infers the total strain upon loading. Stiffness is calculated using Boussinesq half space elastic analysis (incorporating permanent as well as resilient deformation, Section 251.
6.4.3), however, the triaxial stiffness is calculated directly from the resilient deformation upon unloading and therefore the two values will show some discrepancy as they are not measuring precisely the same thing.

The correlation of GDP stiffness and triaxial stiffness are presented in Figure 6.11, and Table C5.4. Where the triaxial sample reached the test limiting strain (5%) prior to a deviator stress of 100 kPa the stiffness value reported has been extrapolated to 100 kPa. As expected, the GDP reads slightly lower values of stiffness than the triaxial data. The GDP readings on subgrade have typically been in the region of 15Mpa, consequently, limited data of higher in-situ stiffness measurements are available. Figure 6.11 shows a poor correlation between the two sets of data, with the GDP reading approximately 33% of the triaxial stiffness. This poor correlation is attributed to the line of best fit being forced through the origin, inaccuracy due to the assumed contact stress of the GDP as well as the small element of permanent strain induced when testing with the plate devices, reducing the measured stiffness. If allowances are made for the correlation between the TFT and FWD (Section 6.4.4), with consideration given to the variability observed with the triaxial data and the different confining conditions between laboratory and field tests these data imply that the field measured values are in a similar range to the laboratory measured data.

6.6.2 Comparison of Stress-Dependency from Laboratory Tests to Field Data

Figures 6.12 and 6.13 show the triaxial stiffness measurements for the undisturbed subgrade samples from Trial 1 and Trial 3 together with the stress-dependency data for the subgrade in the same bays. The subgrade stress-dependency measured on site in Trials 1 and 3 show (as expected) reduced stiffness and smaller changes in stiffness with increasing applied stress, such that a stiffness asymptote is measured (Figures 5.36 and 5.80). The Trial 2 subgrade showed slightly different behaviour, due to the gravel, showing an increase in stiffness with applied stress (Figure 5.54), similar to the granular materials.
Figures 6.12 and 6.13 show that the site stress-dependency data are all within the range of the triaxial stiffnesses. The patterns of stress-dependency measured in the field and laboratory are similar, but due to the variability of readings both in the laboratory and on site no specific correlation can be seen between data from similar points.

However, these data confirm similar stiffnesses and patterns are observed in the laboratory and in the field. This indicates that typical trends from laboratory data and measurements made on site are comparable. Therefore, the use of appropriate worst case laboratory design data with appropriate factors will enable prediction, with statistical certainty, of acceptable field target data, therefore designs can be verified by site testing.

With the addition of capping the stress-dependency of the materials can be seen to change. For Trials 1 and 3 the composite stress-dependency of the capping and the subgrade changes to become similar to the stress-dependency of the capping alone (Figure 5.82). This trend becomes more pronounced with increasing applied stress and increased thickness, demonstrating the reduced influence of the subgrade.

The Trial 2 stress-dependency also shows a progressive change with thicker capping. At higher applied stress (FWD applied stresses) the stress-dependency data on the composite foundations are shown to flatten (Figure 5.57). This may indicate that a maximum value of trial stiffness was observed, whereby the increased applied stress resulted in a limited increase in stiffness of capping, and a similar reduction in subgrade stiffness. Alternatively, the sub-base could show a large increase in stiffness, as found in the back analysis, (however, the sub-base stiffness values calculated must be treated with caution, Section 6.8). This could result in a lower applied stress to the capping and a reduced capping stiffness, which therefore gives a constant value.

The variability of the stiffness measurements on subgrade again play a role in the stress-dependency data collected, and again it is difficult to predict precise stiffness/stress...
relationships for the site, although the trends can clearly be seen. Therefore, each bay of
each of the trials will have its own stress dependant behaviour owing to the composite
performance of the different bays, due to the different subgrade and capping performance.

6.6.3 Threshold Stress Parameters

In Sections 6.3.4 and 6.3.10 the triaxial threshold subgrade stress parameter has been
investigated at points of maximum curvature and at the point of 1 % permanent strain.
The lines of best fit through the threshold stress values have been shown on Figure 6.1 to
correlate with a deviator stress equivalent to the value of undrained shear strength.
Although the correlations are poor this indicates a relationship between shear strength and
the point at which the permanent strain starts to become unstable. This point further
correlates with values of stiffness at or near the stiffness asymptote during the last sets of
load applied in the triaxial test, compared to stiffness at the point when the permanent
strain starts to become unstable (based on stiffness at 0.5$q_{max}$ (Section 6.3.10)) (Figure
6.14).

To develop a site measurable threshold stress parameter, it is proposed to link the
threshold stress to the undrained shear strength and then correlate this result to DCP
subgrade penetration rates from testing on site. The correlation between the DCP
penetration and undrained shear strength from the undisturbed triaxial samples and with
threshold stress is presented in Figure 6.15. This shows a considerable scatter and poor
correlations to the lines of best fit, but a basic trend of reducing strength with DCP is seen
as well as a comparable result for threshold to DCP. The DCP results from the majority
of the sites tested showed little variation in penetration across the particular subgrades,
and therefore it is considered that the main reason for the scatter is the more accurate
measurement of strength in the triaxial cell, although the scatter observed is not
unexpected, (Van-Deussen et al 1994). If a suitable shear strength test for site can be
defined that correlates well with laboratory values then the threshold stress approach to
field compliance testing is worthy of further consideration.
The back-analysed capping stiffnesses and the data from the FWD tests undertaken at higher applied stress appear to suggest that the capping on site becomes less stress dependent at higher stresses (similar to the subgrade) (Section 6.6.2). However the magnitude of the capping field stiffness asymptote will vary from material to material, similar to subgrade, as has been shown by the back-analysis for the various materials.

6.7 Back Analyzed Capping and Sub-base Stiffness

The back-analysed stiffness of the cappings from the measured stress-dependency of the composite trial performance and subgrade testing reveal a number of issues on their stress-dependency. Each analysis has been undertaken by fitting a curve to the stress-dependency data and using its equation to predict values of stiffness based on back-analysed applied stresses (Section 4.6). Due to the nature of the analysis based on the composite stress-dependency measured with the TFT and the variability of the data from the devices (Section 6.4.3) the back-analysed values of capping stiffness should perhaps only be regarded as indicative values.

The cappings have typically shown improving stiffness with increasing applied stress Figures 6.16 to 6.20. The back analysed capping stiffness values were significantly lower than values that have been measured for granular materials in the laboratory (Section 2.5.3), with stiffnesses typically being in the range 60 to 150Mpa. The stress-dependency typically follows the K9 model (Section 2.5.3). Table 6.4 gives the values of the constants k1 and k2 derived for the curves. The values of k2 are in the range 0.0001 to 1.739 with the majority of values being in the range 0.3 to 0.65, which match values of the constant seen in other research (Section 2.5.3). The higher and lower values of k2 being attributable to problems with the analysis due to variability of the data. The varying confinement between site and laboratory and changes in applied stress with depth, which will result in changes in the granular material stiffness also play a role in this variability. These effects are not reflected in the linear elastic modelling used (Section 2.11) or in the
triaxial cell, (Section 2.5.3). This questions the suitability of capping stiffness measured in the triaxial test for routine design purposes.

Only the thickest layers have been assessed for Trials 1 and 3 Granodiorite, based on an average triaxial test subgrade stress-dependency, and TFT measured top of capping stress-dependency (Figure 6.16). For Trial 2 and the originally constructed limestone at Trial 3 each bay has been considered separately, based on TFT subgrade stress-dependency for Trial 2 and triaxial stress-dependency for Trial 3. The top of capping and sub-base composite performance has been based on TFT data.

The Trial 2 capping stiffness shows a slight reduction in stiffness with increasing stress in the thickest 400mm bay (Figure 6.17). This is due to the high stiffness and stress-dependency of the subgrade in this bay (Figure 5.54) and the effects of the composite behaviour from which the capping stiffness was analysed. Some of the bays show better capping stiffness in thinner bays at Trial 2.

This is due to two reasons, in a thick layer the stress reduces though that layer, therefore the response of the lower layers is less and the stiffness of a thick layer will consequently appear lower due to the influence of the material lower within the layer. This effect will reduce the applied stress to the subgrade, which increases the stiffness of the inversely stress dependant materials beneath. In a thinner layer the affects of reducing stiffness with depth is minimal, therefore a large change in stiffness is required to give a small change in composite stiffness, and a high stiffness is thus found. However, these observed trends are more functions of the type of analysis than true material behaviour.

Generally for all the back-analysed capping stiffnesses the stress-dependency shows a minimal change in stiffness with increasing applied stress. The finer cappings and more well graded materials tend to show a better stiffness than the coarser materials and slightly lower stress-dependency (Figure 6.16). The influence of the rain can clearly be seen on the Limestone stiffness when comparing the back analysed values both wet and
dry from Trial 3 (Figure 6.16). The dry limestone shows a clear pattern of increasing stiffness with increasing applied stress, and the bays show improved stiffness with thickness (Figure 6.18). Both the Trial 1 and Trial 3 Limestone initially show similar patterns of stress-dependency, with the differences probably being due to the different material gradings, although these will be influenced by the stress-dependency of the subgrade on the back-analysed values. The stiffness of the Trial 3 Granodiorite capping is low as, expected (20 to 40 MPa), with limited stress-dependency (Figure 6.16).

The sub-base stiffnesses are back analysed as being very high, this is due to the thin layer, with the back analysis being based on the composite performance measured on the capping prior to sub-base placement (Figure 6.19). The capping when confined will probably show improved behaviour, and thus a better composite performance as evidenced by its site performance during compaction of sub-base. It can be seen from the analysis that the change in stiffness of sub-base required to have an influence on the stiffness of the composite needs to be very large. Thus the very large changes in sub-base stiffness with change in stress indicate that a thin layer has a minimal effect on stiffness unless its stiffness is very high, as described above. This is again observed in the combined sub-base and capping analysed stiffnesses assessed on Trial 2 (Figure 6.20).

These data suggest (not unexpectedly), that the behaviour of a foundation is a function of the granular materials used, the subgrade and the thickness of the materials used. Therefore, unique relationships of composite performance will occur for each material and design thickness.

6.8 Back-Analysed Stiffness in Relation to Permanent Deformation

Utilising the stress-dependency for the capping and sub-base derived from the back analysis the applied stress and deformation at subgrade level have been calculated. The threshold stress values derived from the laboratory testing then compared to the back-analysed applied subgrade stress and observed rutting performance.
For Trial 1 the back-analysed-applied subgrade stress under wheel loads are well below the threshold and therefore no significant rutting was seen. Assuming similar stiffness/stress relationships for thinner bays than those back analysed for the 450mm Trial 1 cappings and a design subgrade threshold stress (at 1% strain) of 40 kPa, (averaged from the data in Table 6.1), the thicknesses required to limit applied subgrade stress to below 40kPa are 315mm for the Limestone, 376mm for the 6F1, 40mm down and 395mm for the 6F2, Porphyritic Andesite. These design thicknesses rank the materials for their observed rutting performance. i.e. the Limestone was most over designed and therefore showed least rutting, the 6F2 was least over designed and hence showed most rutting, and showed marginally more rutting than the 6F1, which was slightly more over designed than the 6F2, assuming the cappings remain internally stable.

For the Granodiorite capping at Trial 3 a sub grade threshold stress of 70 kPa has been defined from the triaxial test data. This threshold leads to a back-analysed capping design thickness of 300mm to limit applied stress. This thickness corresponds with the change in the observed rutting modes seen in the limited trafficking undertaken (Section 6.4.7), although the capping instability still would have discounted this material for trafficking.

For Trial 2, the rutting was acceptable within the 400mm and 150 sub-base bay (Bay 1) and on the limit in the 300mm plus 150mm sub-base bay (bay 2). The back analysed applied subgrade stress in Bay 1 was 25kPa back analysed, and 38kPa in bay 2. By correlation of PI and DCP to strength and assuming Cu is equal to $q_{thres}$ then a threshold stress of approximately 40kPa is observed which links to the rutting behaviour observed. Similar to Trial 1 the bays are ranked according to the rutting observed by the value of back analysed subgrade stress, although the differences are smaller, according to the back analysis.

The rutting data and the threshold stress approach suggested here are not directly linked, since the field-measured data will include the rotation of principal stress. The above
classification approach is based on resilient behaviour, (rotation of principal stresses has minimal effect on stiffness, Section 2.4.4) influenced by resilient strain of the subgrade and its effects on horizontal strain at the base of the granular layer (Section 2.5.7) increasing applied subgrade stress. This suggests that resilient properties have a significant influence upon rut formation as well as the permanent strain/shearing of the granular materials. Therefore limiting applied stress to subgrade to control rutting may be appropriate as long as the values of stress at which stiffness is measured are comparable both in terms of method of measurement and magnitude.

6.9 Implications for Performance Based Design, Specification and Construction Compliance Testing

6.9.1 Suitability of Dynamic Plate Bearing Tests for Field Assessment

The applied stresses of the TFT is seen to cover the range of applied subgrade stresses back analysed beneath the constructed granular materials, when testing on the subgrade alone (<100kPa). However stress-dependency testing at stresses less than 40kPa is suggested. The range of applied stresses from the TFT appears too low for the complete granular pavement where a larger stress equivalent to a wheel load is required (500 kPa) (Section 2.3.1). For many of the materials assessed in this research the capping asymptote is approached at the highest of the TFT applied stresses (Figure 6.16). This suggests that a TFT stiffness at its highest applied stress may be acceptable as it will give sufficient data to forward predict the stiffness at much higher stresses. For the design approach suggested below it is important that the composite properties are measured at representative higher stresses similar to those found in service as well as over the range of stresses assessed in this research, which potentially rules out the GDP as an acceptable device. The TFT could be used with a smaller plate to assess higher stresses (comparable to those from a wheel load) however the effect of particle size and plate diameter, particularly for coarse cappings, may then become significant. The FWD can apply these higher stresses, but can not assess at the lower range required for subgrade, but the
correlation between the devices suggests that site testing using the two devices will give data acceptable for design.

6.9.2 Assessment of Density for Compliance

Given the variability and problems in establishing acceptable laboratory compaction properties (Section 6.2.1.3) of large particle size granular materials, setting acceptable target density values for site compliance is difficult due to problems with laboratory testing. Therefore, a target value for site compliance set from a field compaction trial could be used, based on the maximum density that can be achieved during a field trial, which may also be required to assess the rutability.

6.9.3 Target Stiffness Values for Compliance Testing

Many different top of foundation target stiffness values have been proposed (Section 2.9.2) to define a required value of top of foundation stiffness needed either to allow satisfactory compaction of overlying materials, or above which adequate trafficking of the foundation has been observed. These values for trafficking have typically been high, in the region 80 to 120 MPa (for a 300mm plate, see Section 2.9.2). These stiffness values have only been achieved in a few bays in any of the trials and these bays have typically shown acceptable performance under trafficking. However, these targets seem high for potentially lower bound values which are at the limits of acceptable performance.

Trial 1 showed the best performance under trafficking and showed least rutting. The coarse material bays typically had stiffnesses in excess of 50Mpa when measured with the FWD and TFT, and above 30MPa when measured with the GDP. This finer 6F1 capping failed to reach these stiffness values yet still showed acceptable trafficability. The values measured are below the target stiffness values suggested for acceptable trafficability (Section 2.9).
Trial 2 showed stiffnesses above the suggested TRL target values (Section 2.9) when measured with the TFT in the three thickest bays (including sub-base), and slightly below the target values in a fourth. However, acceptable performance was only seen in the thickest bay, which has a stiffness of 82Mpa, which is considerably higher than the next stiffest bay, although large scatter to the data is seen. Similarly, the GDP stiffnesses are again all above TRL target in four out of five bays.

The variability of performance of the trials in comparison to the rutting targets set by various authors (Section 2.9) is not surprising. A high composite stiffness is normally observed with materials with high strength and hence transfer of applied stress though the materials. The resilient properties of the materials will play a role in rutting performance (Fleming and Rogers 1995), hence the stiffness will have some bearing on rutability. However, this will vary from material to material and from site to site, due to the materials' internal stress, strain and strength performance and the composite capping/subgrade performance, Therefore stiffness to rutting targets values should be site specific.

The variability of the performance of in-situ subgrade has been found to be significant within the small areas assessed as part of the trial, defining values of composite stiffness for assessment in situ that will be applicable across a complete site is difficult. However the behaviour of the capping can be derived from back-analysis during a trial to give a likely indication of the capping performance that could then be used with a worst case capping stiffness value for design. However, the current inability to assess capping stiffness in the laboratory to give stiffness parameters for design is a significant limitation.

6.9.4 Selection of Stiffness values for Design from Laboratory Data

For design purposes conservative values of stiffness at high deviator stress (asymptotic values) and hence low stiffness are currently used from remoulded samples and these
values are probably significantly below the actual stiffness of the materials at the lower applied stresses experienced during service in a pavement.

6.9.5 Threshold Stress Based Design

The back-analysed rutting data (Section 6.8) are significant as they indicate a possible trend that can be used for design. If a back-analysed capping stiffness and stress-dependency can be identified from a field trial and a subgrade stiffness stress-dependency defined, the threshold stress can be evaluated (from the relationship with shear strength Section 6.6.3). The capping thickness that will limit the applied subgrade stress can then be defined (as in Section 6.8). The stiffness at threshold stress is shown to be approaching stiffness asymptote (Section 6.6.3). The asymptotic value may be an acceptable value for the long-term design, not only for threshold stiffness as described above, but also for target stiffness for construction of upper layers.

In this approach the stress-dependency of the subgrade could be defined from the triaxial test. Although long-term water content effects must be considered, consequently a worst case value of stiffness based on an asymptote may be acceptable. The stiffness asymptote has been shown to be a worst case with similar values on the respective sites. This can then be checked against a threshold value, leading to a design thickness based on a measured stiffness value.
Table 6.1, Comparison of Triaxial Data for Threshold Stress and Shear Strength and Stiffness

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<tr>
<th>Sample</th>
<th>Shear Stress @ 1% Strain (kPa)</th>
<th>Threshold Stress (kPa)</th>
<th>Max Curve Stiffness (MPa)</th>
<th>Asymptote at q=(\gamma) (MPa)</th>
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### Table 6.1, Continued, Comparison of Triaxial Data for Threshold Stress and Shear Strength and Stiffness

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Table 6.2, Comparison of Triaxial Stiffness Calculated using Fixed Volume and Proximity Based area Correction

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<td>Proximity Transducers</td>
<td>-12.23 -12.00 -11.55</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 6.3, Correlation of FWD to GDP and TFT for the Various Field Trials

<table>
<thead>
<tr>
<th>Site</th>
<th>Materials</th>
<th>TFT Vs FWD</th>
<th>GDP Vs FWD</th>
<th>R²</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Trial 1</strong></td>
<td><strong>Subgrade (1 Test at a Point)</strong></td>
<td>0.743</td>
<td>0.423</td>
<td>-0.36</td>
<td>-3.75</td>
</tr>
<tr>
<td></td>
<td><strong>Subgrade (2 Tests at a Point)</strong></td>
<td>0.784</td>
<td>0.482</td>
<td>-0.139</td>
<td>-0.91</td>
</tr>
<tr>
<td></td>
<td><strong>Capping (1 Test at a Point)</strong></td>
<td>0.92</td>
<td>0.543</td>
<td>-0.232</td>
<td>0.388</td>
</tr>
<tr>
<td></td>
<td><strong>Capping (2 Tests at a Point)</strong></td>
<td>0.899</td>
<td>0.603</td>
<td>-0.899</td>
<td>-0.295</td>
</tr>
<tr>
<td></td>
<td><strong>Combined (1 Test at a Point)</strong></td>
<td>0.904</td>
<td>0.532</td>
<td>0.707</td>
<td>0.799</td>
</tr>
<tr>
<td></td>
<td><strong>Combined (2 Tests at a point)</strong></td>
<td>0.899</td>
<td>0.593</td>
<td>0.63</td>
<td>0.821</td>
</tr>
</tbody>
</table>

| Trial 3 | Subgrade                  | 0.911      | 0.643      | 2.217 | 2.31 |
|         | Granodiorite              | *          | 0.456      | *     | 0.278 |
|         | Limestone                 | 1.057      | 0.589      | 0.307 | 0.278 |
|         | Sub-base on Granodiorite  | 1.335      | 0.709      | 0.29  | 0.037 |
|         | Sub-base on Limestone     | 1.404      | 0.748      | 0.481 | 0.359 |

<table>
<thead>
<tr>
<th>Site</th>
<th>Materials</th>
<th>TFT Vs GDP</th>
<th>R²</th>
<th>GDP Vs FWD</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Trial 2</strong></td>
<td><strong>Subgrade</strong></td>
<td>1.815</td>
<td>0.296</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Capping</strong></td>
<td>1.622</td>
<td>0.687</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Sub-base</strong></td>
<td>1.816</td>
<td>0.856</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Combined</strong></td>
<td>1.749</td>
<td>0.741</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Trial 3 | Subgrade                  | 0.567      | 0.378 |
|         | Limestone                 | 1.485      | 0.872 |
Table 6.4, Backanalysed Granular Material Stiffness Constants

<table>
<thead>
<tr>
<th>Trial 1</th>
<th>Thickness</th>
<th>Granular material</th>
<th>k1</th>
<th>k2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>450mm</td>
<td>6F2 Limestone</td>
<td>16.716</td>
<td>0.4172</td>
</tr>
<tr>
<td></td>
<td>450mm</td>
<td>6F2 Porphyritic Andesite</td>
<td>78.238</td>
<td>0.0509</td>
</tr>
<tr>
<td></td>
<td>450mm</td>
<td>6F1 Porphyritic Andesite</td>
<td>125</td>
<td>0.0001</td>
</tr>
<tr>
<td>Trial 3</td>
<td>450mm</td>
<td>Granodiorite</td>
<td>3.829</td>
<td>0.483</td>
</tr>
<tr>
<td></td>
<td>450mm (Wet)</td>
<td>Limestone</td>
<td>9.929</td>
<td>0.557</td>
</tr>
<tr>
<td></td>
<td>450mm (Dry)</td>
<td>Limestone</td>
<td>4.137</td>
<td>0.314</td>
</tr>
<tr>
<td>Trial 3</td>
<td>450mm Bay (Average)</td>
<td>Limestone</td>
<td>14.831</td>
<td>0.449</td>
</tr>
<tr>
<td></td>
<td>300mm Bay (Average)</td>
<td>Limestone</td>
<td>9.130</td>
<td>0.511</td>
</tr>
<tr>
<td></td>
<td>150mm Bay</td>
<td>Limestone</td>
<td>4.010</td>
<td>0.590</td>
</tr>
<tr>
<td>Trial 2</td>
<td>300mm Bay 2</td>
<td>Porphyritic Andesite</td>
<td>3.378</td>
<td>0.634</td>
</tr>
<tr>
<td></td>
<td>200mm Bay 2</td>
<td>Porphyritic Andesite</td>
<td>2.316</td>
<td>0.520</td>
</tr>
<tr>
<td></td>
<td>100mm Bay 2</td>
<td>Porphyritic Andesite</td>
<td>8.415</td>
<td>0.174</td>
</tr>
<tr>
<td></td>
<td>200mm Bay 3</td>
<td>Porphyritic Andesite</td>
<td>8.399</td>
<td>0.568</td>
</tr>
<tr>
<td></td>
<td>100mm Bay 3</td>
<td>Porphyritic Andesite</td>
<td>0.079</td>
<td>1.279</td>
</tr>
<tr>
<td></td>
<td>100mm Bay 4</td>
<td>Porphyritic Andesite</td>
<td>0.015</td>
<td>1.739</td>
</tr>
<tr>
<td></td>
<td>400mm+150mm Sub-base Bay 1</td>
<td>Porphyritic Andesite</td>
<td>135.170</td>
<td>0.060</td>
</tr>
<tr>
<td></td>
<td>300mm+150mm Sub-base Bay 2</td>
<td>Porphyritic Andesite</td>
<td>24.101</td>
<td>0.319</td>
</tr>
<tr>
<td></td>
<td>200mm+150mm Sub-base Bay 3</td>
<td>Porphyritic Andesite</td>
<td>125.880</td>
<td>0.101</td>
</tr>
<tr>
<td></td>
<td>100mm+150mm Sub-base Bay 4</td>
<td>Porphyritic Andesite</td>
<td>3.487</td>
<td>0.545</td>
</tr>
</tbody>
</table>
Figure 6.1, Threshold Stress Vs Shear Strength for All Triaxial Samples

\begin{align*}
\text{Threshold Stress} &= \text{Cu} \\
\text{Threshold Stress at Max. Curvature} \\
\text{Threshold Stress at 1\% Strain}
\end{align*}

\begin{align*}
y &= 1.0597x \\
R^2 &= 0.6407
\end{align*}

Figure 6.2, Sherby-Dorn Plot for Bardon Sample A, (each curve is plotted over 1000 cycles at each deviator stress)

\begin{align*}
\text{permanent strain \%} & \quad 0.19 \quad 0.17 \quad 0.23 \quad 0.73 \quad 0.87 \quad 1.52 \quad 2.1 \quad 2.45 \quad 3.34 \quad 3.58 \quad 4.45 \quad 5.17 \quad 5.79 \\
\text{1} & \quad 0.1 \quad 0.1 \quad 0.01 \quad 0.01 \quad 0.001 \quad 0.001 \quad 0.0001 \quad 0.0001
\end{align*}
Figure 6.3, On-Sample Strain Gauge Reading Against Time for Cycling at 5kPa Deviator Stress

Figure 6.4, On-Sample Strain Gauge Reading Against Time for Cycling at 10kPa Deviator Stress
Figure 6.5, Correlation between both TFT and GDP Stiffnesses and Stiffness Measured by the FWD on the Subgrade and Capping for Three Tests at a Single Point at Trial 1

Figure 6.6, Correlation Between the Stiffness measured using the TFT and the GDP on the Subgrade, Capping and Sub-base Combined Trial 2
Figure 6.7, Correlation Between the Stiffness measured using the TFT and the GDP on the Subgrade at Trial 2

Figure 6.8, Relationship Between Sub-base Stiffness and NDG Dry Density Granular Material Thickness, Trial 2
Figure 6.9, Deformation Caused by the Roller during Compaction

Figure 6.10, Rutting of the Thinner Bays of the Trial 3 Granodiorite
Figure 6.11, Relationship between GDP Stiffness and Triaxial Test Stiffness for a Deviator stress of 100 kPa

\[ y = 0.3319x \]
\[ R^2 = -0.7522 \]

Figure 6.12, Relationship Between Repeated deviator Stress and Stiffness for Undisturbed Samples from Trial 1 Tested in the Repeated Load Triaxial Test and with the TFT

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Figure 6.13, Relationship Between Repeated deviator Stress and Stiffness for Undisturbed Samples from Trial 3 Tested in the Repeated Load Triaxial Test and with the TFT.

Figure 6.14, Comparison of Stiffness Asymptote Values and Stiffness at 0.5qmax from the Triaxial Test.
Figure 6.15, Relationship between DCP Penetration Rate and Undrained Shear Strength and Threshold Stress for Undisturbed Subgrade Samples Tested in the Triaxial Test

\[ y = -0.2452x + 56.313 \quad R^2 = 0.1872 \]

\[ y = -0.2887x + 57.385 \quad R^2 = 0.3055 \]

Figure 6.16. Back-analysed Capping Stiffness for 450mm Bays Trials 1, 2 and 3

\[ y = -0.2502x + 57.486 \quad R^2 = 0.2812 \]
Figure 6.17, Trial 2, Back-analysed Capping Stiffness for each Layer in Each Bay

Figure 6.18, Trial 3, Limestone Trial Back Analysed Stiffness for Each Layer in Each Bay
Figure 6.19, Back-analysed Sub-base Stiffness based on Composite Capping Stiffness

Figure 6.20, Trial 2, Capping and Sub-base (Combined) Back-analysed Stiffness
7.0 CONCLUSIONS and FUTURE RESEARCH

7.1 Conclusions

A performance based specification for road foundation layers would provide greater assurance of adequate construction and long-term performance and would allow the use of more marginal capping materials as their in-service performance could be assured. However its success requires accurate measurement of the elastic stiffness of subgrade, capping and sub-base materials, as well as provision to restrict rutting under construction traffic loading. This research has investigated all of these key elements and demonstrates that such an approach is an achievable goal. However further research to continue development of laboratory testing techniques (especially granular materials) and analytical pavement foundation modelling is required.

The Repeated Load Triaxial Test developed by Cheung (1994) is the most appropriate laboratory method of measurement of the resilient modulus of the subgrade at anticipated conditions during construction. However, the testing programme reported herein revealed a number of significant problems that must be addressed prior to this apparatus being used for routine testing, (see Section 7.2). The small strains measured at low applied deviator stresses showed significant variability across samples and between samples from the same site for a number of reasons. The calculation of stiffness modulus at low deviator stress (<30 kPa) was highly sensitive to small differences in strain, and this lead to large variation in the test data. This was caused by errors related to apparatus precision and the method of connection of on-sample strain gauges.

Soil variability made it difficult to obtain representative laboratory derived values of soil properties for any given site, not only between test samples, but also across any one sample. Significant variability was found in the resilient and permanent deformation properties of samples of soil collected from small areas of the same site.
Conditioning of samples in the triaxial test prior to assessment of their performance properties was shown to reduce significantly the permanent deformation sustained below the conditioning stress previously applied and results in a slight increase in the measured stiffness. Therefore the importance of assessing permanent deformation and resilient properties from the same sample of subgrade has been demonstrated.

Remoulding and re-compaction of soil for the manufacture of samples for laboratory testing caused a destruction of soil structure. This was found to redistribute weak or strong materials and water within a sample, which affected compaction and subsequent test behaviour. These changes in performance will affect samples prepared to assess the long-term properties of soil samples i.e. those prepared at predicted long-term water contents, and therefore appropriate methods of sample manufacture must be used to mimic site conditions if accurate results are to be obtained. This would mean producing samples wetted from undisturbed samples which has been shown to be impractical, therefore this change in behaviour should be considered when assessing performance in manufactured samples.

Threshold stress at 1% permanent strain for soil samples tested in the repeated load triaxial test showed an approximate correlation with the point of maximum curvature of the plots of stress against permanent strain. Adoption of a threshold stress approach therefore appears suitable for assessing the point beyond which the permanent strain of the soil starts to become unstable. These threshold stress values were also shown to correlate to the permanent strain at a deviator stress of 0.5$q_{\text{max}}$ (i.e. a deviator stress equivalent to the undrained shear strength). Stiffness at a deviator stress of 0.5$q_{\text{max}}$ was similar to, or slightly above, the value of stiffness at the stiffness asymptote. This indicated that large changes in the stress-dependency of fine grained soils occurred below the deviator stress at which the permanent deformation behaviour of the soil became unstable. Conversely when permanent strain becomes unstable the stiffness tends to a stiffness asymptote.
This research has shown the TFT to be suitable to assess pavement foundations during construction and in combination with the FWD can apply stresses across the full range experienced in a pavement. The assumed single value of contact stress (100kPa) adopted by the GDP in place of load measurement suggests this is an inappropriate device for use other than for comparative testing.

As observed in the triaxial testing, the natural variability of the subgrade resulted in significant variability in the measurements of subgrade stiffness made with the dynamic plate devices. The field trials showed that significant scatter of the data was produced by all the dynamic stiffness measuring devices, with typical coefficients of variance in the range 20 to 50%. More scatter was observed on the subgrade (mainly attributed its variability), than on thick layers of granular capping and sub-base materials, when the influence of the subgrade on the test was reduced. Results from different dynamic testing devices at adjacent points on subgrade were sometimes dramatically different. This difference was attributed to a combination of different loading pulse shapes, the function and location of the measurement transducers, and the way in which the measurements are converted into displacements.

More generally material type, particle size and contact effects were shown to influence the results. Regardless of the materials tested the GDP consistently measured a lower stiffness (by approximately 50%) than the FWD on both subgrade and capping, whereas the TFT typically measured slightly lower stiffnesses (approximately 90%) of the FWD values. In addition, site/material specific device correlations were observed from the trials, and thus the correlations between dynamic plate stiffness measuring devices are considered acceptable for assessing behaviour with different devices at the same site.

The adequate compaction of thin layers of granular materials, as assessed by density, was achieved in the field trial regardless of the supporting stiffness. No target stiffnesses that will ensure the adequate compaction of an overlying layer, if assessed by density, can therefore be set from this research. (For the structural pavement layers such targets may
exist, but should be assessed from the top layer of the pavement down. It is anticipated that such targets will be site and material specific.) The density of the materials achieved during compaction was found to be related primarily to the compactive effort applied, rather than to the efficacy of support. A maximum value of density to which the materials could be compacted on site was apparent from the trials. However, the fieldwork suggested that better compaction (although not manifested by an increase in higher density), could be achieved in terms of an improved compacted (or structural) matrix, and this may improve the materials’ resistance to permanent deformation.

The measured stiffness of the compacted layers was more dependant on the total thickness of the supporting granular layers than the compactive effort applied when thin layers of granular materials were utilised, but did increase with the amount of compactive effort expended. The addition of a thin layer caused only a limited increase in stiffness of a low stiffness material, but caused a significantly greater increase in stiffness when laid upon a stiffer material. However, this finding is believed to be partly related to the fact that the devices used for assessment are most strongly influenced by the materials close to the surface. Improved stiffness of the compacted materials was observed to be greater a few days after completion of construction, than immediately after construction. This was attributed to the dissipation of the effects of the higher applied stresses during compaction (probably pore water pressure effects). Further improvements in stiffness were observed in the long-term due to material aggregation and subgrade thixotropy.

Since the granular materials could be adequately compacted (as evaluated by density) regardless of the stiffness and thickness of support, adequate design to limit permanent deformation is a more critical load case for the construction condition. Permanent deformation readily occurred where the granular materials were thinner and where they possessed low shear strength. Thinner bays exhibited more rutting, and rutting occurred more quickly, demonstrating the importance of adequate stiffness in dissipating applied stresses to the subgrade.
Where the applied sub-grade stress was beyond the threshold value, the strength of the materials (especially subgrade) appeared to control the extent of rutting. Granular materials with low inherent strength rutted readily, demonstrating the importance of ensuring that the granular materials possess sufficient internal stability.

The fieldwork indicted that there may be a link between resilient deformation of subgrade and rutting, although limited data are available. This suggests that a target stiffness exists above which rutting will not occur (i.e. a stiffness that limits applied subgrade stress to below a threshold value). However this will be site specific.

Similar patterns and values of stiffness and stress-dependency of the subgrade were observed between laboratory and field tests, although correlations between discrete tests have not been possible due to the natural variability of the subgrade and the variability in results between testing devices. However, the values of stiffness and trends of stress dependency of the subgrades assessed in the field were all within the boundaries of the loci of test data collected from the similar subgrade samples tested in the repeated load triaxial test, showing that the trends and values are comparable. Poor correlations were found between the measurements of subgrade shear strength in the laboratory and comparable field strength measurements determined using the DCP. However, a correlation between DCP and laboratory threshold stress was found, although this finding was based on limited data and further observations are required to support this conclusion.

The influence of the subgrade on the performance of a composite road foundation was found to be site specific and related to the functional properties of the subgrade and the imported materials used to construct the foundation as well as their combined composite behaviour and foundation thickness. The precise influence of the subgrade can only be identified if the precise performance properties of all the materials used can be assessed at appropriate conditions.
The influence of weather conditions on the constructed properties of materials can be considerable, and the susceptibility of the materials to these effects, particularly the influence of increases in water content, should be assessed prior to construction. Site conditions can improve material performance of both subgrade and granular materials in the short term.

7.2 Suggested Future Research

The repeated load triaxial test developed by Cheung requires further modification to be used as a routine piece of apparatus to assess subgrade properties. Although the loading system has proved adequate, the strain measurement needs to be refined so that low resilient strain can be precisely measured over a large permanent strain range. This could be achieved either by a series of differently calibrated on-sample strain gauges, or via non-contact optical methods. Non-contact methods would also have the advantage of eliminating the gauge connection errors that were observed during this research. The test methodologies could also be refined to reduce the time required to perform a test.

The modelling of subgrade samples in the long term is a significant challenge. A suitable method of varying the water content of soil to represent dissipation of suctions for in-service pavement subgrades is required. The method chosen must be such that it does not cause an (unrealistic) improvement in properties, in mixed or fissured soils. Current methods of prediction of long-term water contents rely heavily on the soil plasticity data and predicted soil suctions. Soil suction is difficult to measure routinely, while relying on plasticity data to predict suction is liable to error in mixed soils. A suitable method of predicting change in water content over time is required in order to allow assessment of worst case soil properties.

The laboratory testing of aggregates containing large particle size is a further area that requires research. The tests that are currently widely used for assessing compaction properties of granular materials are unsuitable for particle sizes above 40mm. A
laboratory test (similar to the PSSR test for sub-base) is required in order to assess the internal stability of coarse aggregates subject to trafficking. The current inability to measure, in a laboratory environment, the stiffness of granular materials with large particle size at appropriate (and variable) levels of confinement, similar to those experienced in the field, is a significant limitation for the analytical assessment of pavement foundations performance. It is therefore clear that such a test also needs to be developed.

Target values of stiffness for ensuring adequate construction of pavement foundations and to control the formation of rutting have been shown to be site specific, further research is therefore required to assess such targets for typical materials.

The mechanism of the development of rutting clearly requires further research. This should include studies of the initiation of rutting, the mechanism by which it occurs, and the particle movements, and associated material/layer interactions. A rutting model then needs to be developed which would enable accurate simulation of rutting. A further series of rutting trials is required in order to confirm the role of resilient deformation in the onset of rutting, from this an enhanced analytical design based on threshold stress could be established. These trials should be performed utilising a similar capping on a variety of subgrades, so that the influence of subgrade threshold stress can be found (possibly using the correlation with strength defined in Section 7.2). The stress-dependency of the subgrade and granular materials should be determined, and the granular material stiffness fully evaluated, in order to give repeatable and consistent values of capping stiffness that can then be used to design and evaluate rutting performance observed in the trials.

7.3 Implication of this Research on the Development of a Performance Specification Approach to Pavement Foundation Design

A number of important issues for the development of a performance specification for road foundations, for the construction condition, have been addressed in this research and
concluded upon in Sections 7.1 and 7.2. This research has demonstrated that site compliance tests for a performance specification are currently available, and importantly, that similar trends in data between laboratory and field test have been observed for the specific conditions assessed. The research thus indicates that achievable site-specific compliance values can be set. However, such values arguably need to incorporate a level of safety to provide confidence that adequate performance will occur. The analysis undertaken in this research suggests that 20% variability in field data is typical. However further data collection and a thorough statistical analysis are considered necessary to enable suitable factors of safety to be derived. Given the site-specific nature of the data collected in this research, such safety factors may prove to be site specific.

Other fundamental issues exist that need to be investigated and resolved prior to the development of a true performance specification approach to pavement foundations. The assessment of the long-term performance of materials, particularly the durability of secondary or recycled aggregates, is required ideally at the initial design stage and certainly before construction commences. The development and validation of suitable analytical design tools, incorporating appropriate material models, is also required with robust and routine field and laboratory testing techniques.

It is therefore suggested that, in the short-term, the introduction of site compliance performance testing using currently available techniques is realistic, and would be enhanced by the aid of a pre-construction field trial. The field trial would give greater confidence for the setting of appropriate compliance targets for acceptable construction and performance and could also incorporate the site specific influences to a large degree. However, it is also clear that the implementation of a performance specification requires changes in current contract procedures and responsibilities. Therefore the move to a fully analytical approach to pavement foundation design, utilising appropriate laboratory and field compliance testing to replace the current design approach and ‘recipe’ specification remains an area where further research is clearly required.
8.0 REFERENCES


APPENDICES
APPENDIX A

Description of the Repeated Load Triaxial Apparatus and Sample set-up
Procedures (Summarised from Cheung 1994)

1.0 Loading System
The loading frame is made of rectangular hollow steel section. The triaxial cell is set on a platen which can be raised to position the cell loading piston on the load actuator.

Axial load is supplied by connecting the hydraulically actuated load ram to the piston rod of the triaxial cell. The load platen, with a central cone recess to receive the end of the triaxial piston, is made of aluminium to reduce the dead weight acting on the specimen. A computer commanded electro-hydraulic regulator is used to regulate pressures into the load actuator with a feed-back control loop from the triaxial load cell (Section 7.0). Confining stress is provided by a manually regulated air pressure supply. A maximum cell pressure of 200 kPa is permitted.

To contain the sample a triaxial cell similar to a conventional cell for testing 100mm diameter specimens is used. It comprises three main parts

(1) A cell top made of aluminium,
(2) A cell wall/sleeve of 'Perspex' material, and
(3) A base of epoxy resin.

'O' rings are fitted at both ends of the cell wall to ensure air-tightness between the wall and the top and base. The sleeve is held by an aluminium plate which is fixed by three screws to an epoxy resin base. The base is provided with sealed sockets into which the instrumentation cables are connected within the cell.

2.0 Sealing System
Latex membranes are commonly employed in triaxial testing to provide the enclosure for specimens when the confining fluid is water or oil. However, latex is susceptible to air penetration. The permeability of a 1 mm thick sheet at an air pressure
difference of 1 kPa is $11.49 \times 10^{-9}$ cm$^3$/s/cm$^2$. This is approximately equivalent to 10 cm$^2$ of air penetration per hour at a confining stress of 100 kPa. Since the confining stress to the specimen is provided by pressurised air, membranes made of neoprene, which are impermeable to air, are chosen to eliminate air penetration. However, a dry neoprene membrane may absorb water from the soil specimen. Membranes are thus immersed in de-aired water for 24 hours before being used. The sealing system is completed by the usual method in which two pairs of rubber 'O' rings are used to seal up the ends of the membrane at the top and the bottom platens.

3.0 Instrumentation
A total of seven transducers are employed for the testing:
(1) A pair of strain loops for measuring axial movement up to 2 mm (3% strain),
(2) A pair of proximity transducers for horizontal movement up to 2 mm (4% strain),
(3) A LVDT for vertical movement up to 50 mm (25% strain),
(4) A pressure transducer for monitoring cell pressures (0-700 kPa), and
(5) A load cell for axial stresses (0-1000 kPa).

4.0 Axial Strain
To provide accurate, but simple "on-sample" measurement of small resilient and permanent axial deformations, free from the end effects of the platens, a new measuring device was designed. There were three major considerations in designing the on-sample instrument for measuring the vertical displacements-

(a) Weight of the device,
(b) Measuring range,
(c) Limited space between the specimen and the cell wall.

It is important to minimise the weight of the "on-sample" measuring unit to avoid creating undesirable local distortion of the specimen, especially when soft clays are tested. Two light-weight loops of 12g each were made (Figure 1).

Each loop consists of two locating studs, a pair of rubber bands and an open ended loop. The studs are made of aluminium, and are 3g in weight. The studs connect the
loop to the specimen. At one end of the locating stud, a cross-headed pin provides an anchorage into the soil. At the other end, a square block with a sharp recess at the centre provides a precise fixing point to support the loop. Connection between the square block and the loop is achieved by providing a pin-tailed location screw which fits into the recess of the block. A rubber band is used to provide the fixation. Since the screw is adjustable, setting of an initial reading can be made after the loop has been installed. Once the initial setting is finished, the screw is locked in its final position with the loop by a nut to prevent relative movement during testing.

For the loop, 5 mm diameter hollow brass tube sections of length 46 mm are used at both ends in order to reduce the weight of the instrument. A 5 mm wide by 0.56 mm thick phosphor bronze strip mounted with strain gauges forms the active centre part of the loop. The use of the thin phosphor bronze strip prevents the development of any significant spring force which may be induced by the loop to the specimen. The device records axial movements over the middle third of the specimen. Because of electronic restrictions, the precision value for the present arrangement is limited to 28 microstrain. The device fits easily in the limited space between the specimen and the wall of the triaxial cell. Two loops are provided on each side across the diameter of the specimen.

Although the adjustable screws of the loop can allow re-setting to prevent over-ranging whenever necessary, the measuring range of the loops is not sufficient for triaxial compressive strength testing. The upper limit recommended by BS 1377 (1990) for testing soil strength is 20% axial strain. Hence, a second axial displacement measuring device is provided. It is a large LVDT clamped onto the load ram outside the triaxial cell. This transducer is able to provide a linear measurement of axial deformation up to a range of 50 mm, which is equivalent to 24% of the height of the specimen. The precision is limited to 53 microstrain (again due to an electronics rather than a sensor restriction) over a gauge length of 206 mm.

5.0 Radial Strain

For the new repeated load triaxial apparatus two proximity transducers (part no. 2UB) supplied by Kaman are used to monitor the radial strain of the soil specimen.
The proximity transducers are placed at the mid-height of the specimen and at right angles to the strain loops across the diameter. The measuring device consists of three main parts (Figure 2):

(1) Conductive target,
(2) Sensor, and
(3) Signal conditioning unit.

When the target moves away from the specimen, it causes an impedance change in the coil of the sensor. The signal conditioning unit then converts this change to a voltage. The voltage is proportional to the displacement of the target. A holding frame, supports the proximity transducers, hence, the instrument exerts no force onto the tested specimen. They comprise an external case and an adjustable component to allow the sensor to travel up to 10 mm. To prevent interference to the magnetic field in front of the sensor, the holding materials within the sensing zone of the transducer are made of plastic. The target of the transducer is aluminium foil, 0.07 mm thick and 30 mm square. It is stuck to the surface of the specimen and enclosed between the membrane and the sample. The maximum measuring range of the proximity transducer is 2 mm, which is equivalent to 4% strain across the radius of the specimen, the transducer has a precision of 10 microstrain.

6.0 Axial Load

Axial load applied to the specimen is able to be measured upto 8 kN. The load cell is made of two pairs of strain gauges which are attached to the surface of the triaxial cell piston, where its cross sectional area has been reduced close to the load platen, and is therefore within the triaxial cell. This ensures that friction between the load ram and the top of the triaxial cell does not affect the measurement of applied loads. The precision of the load cell, is 0.4 kPa.

7.0 Confining Stress

A pressure transducer with a measuring range from 0 to 700 kPa is used to monitor the confining stress. Since the transducer is installed inside the triaxial cell, errors
arising from attenuation in the piped system are eliminated. The precision of the cell pressure transducer is 0.4 kPa.

8.0 Control and Data Acquisition Systems

A computer is used as a multi-purpose central unit to issue command signals, collect data and process results. Communication between the computer and other units, including all transducers and the load regulator, is made by a CIO-AD16 Board which is capable of converting analogue to digital signals and vice versa for up to 16 channels. Electronic units from Peter Goss Associates provide the main signal conditioning for all signals except for those from the proximity transducers which possess their own integrated conditioning electronics. The hardware comprises:-

(1) A five channel transducer signal conditioning unit and,
(2) A command signal conditioning unit capable of handling eight control channels.

With the associated DCS (1990) system software, the system is able to generate signals to control the frequency and magnitude of the required load pulses. At the same time, electronic signals from all transducers and from the command channels can be viewed on the computer monitor and stored in the hard disk. The control software runs under the Microsoft Windows environment and is very 'user-friendly' (the software has been specifically written for the repeated load testing of pavement and geotechnical materials).

9.0 Routine Set-up Procedure

The specimens are extruded from the cylinder and trimmed to length. Load spreaders which were made by cutting a tube of an internal diameter equivalent to the size of the soil sample, into three equal longitudinal sections are used to handle soft specimens. After the soil is transferred to the base of the triaxial cell, the exact locations of the fixing points on the specimen for the "on-sample" instrument are marked using a rig and a needle. The main features are two sets of diametrically opposite guide rods fixed between top and bottom rings, and two sliding blocks.
To mark the samples the blocks are allowed to slide to a pre-set height, a needle is then pushed through the hole at the centre of the block. When the marking procedure is finished, the vained studs for supporting the strain loops are inserted by hand into the specimen at the marked locations.

The two pieces of aluminium foil, used as the target for the proximity sensors, are prepared. They are put at the mid-height of the specimen and are placed, in plan, 90 degrees away from the studs.

A standard membrane stretcher is used to place the neoprene membrane, over the specimen. When the membrane is on, the top platen is positioned. To seal the sample, two pairs of rubber 0-rings are rolled into grooves on the top and bottom platens.

A hot soldering iron is used to puncture the membrane over the studs. The blocks, which support the strain loops, are then screwed through the membrane into the studs and the loop attached. Between the membrane and the block, a small "0" ring is used as a seal. To fix the loops onto the blocks, rubber bands are employed. Finally, the frame fitted with the two proximity transducers is bolted onto the base of the triaxial cell. The transducers are all adjusted to read within their specified ranges and the triaxial cell fitted and the cell pressure applied. The cell is then positioned in the load frame and tested.
Figure 1. Strain Measuring Loop

Figure 2. Proximity Transducer
APPENDIX B
Field Trial Site Investigations and Design

1.0 TRIAL 1. HATHERN

1.1 Site Investigation

The site investigation included the excavation of five trial pits. Bulk bag samples and five U100 samples for repeated load triaxial testing were taken from the trial pits. Dynamic stiffness measurements at assumed formation level within the trial pits were performed, at the locations from which U100 samples were taken, using the German Dynamic Plate Bearing Tester (GDP). Dynamic Cone Penetrometer (DCP) tests were carried out in three rows running along the length of the proposed road.

The DCP tests produced typical penetration rates of approximately 69 mm/blow. Hand shear vane tests undertaken in the sides of the trial pit show the soil to have typical strengths of 60 to 70 kPa in the clay i.e. firm consistency. The underlying silty sand, was observed to be loose.

The site was found to be consistent in terms of the soils present along the length of the trial sections, with the ground becoming slightly softer at the northern end. Generally the ground conditions were found to consist of 0.2m of topsoil overlying 1m of a very soft sandy silty clay with occasional gravel, overlying a wet loose single sized silty sand with occasional gravel. The groundwater table was 1.2m below ground level.

1.2 Design of the Trial Road

Data from the DCP tests were correlated with CBR along the site. The following design thicknesses using (Table 2.2, HD25/94 DMRB Vol. 7) were established from the various individual (typically averaged) site investigation design parameters:
DCP equated to CBR (averaged) 465 mm Capping
Plasticity Data (PI = 16%) 250 mm Capping

The design thicknesses based on the full range of site investigation measurements were as follows:

DCP equated to CBR (range 2.0% to 4.0%) 600 mm to 300 mm Capping
Plasticity Data (PI = 11-17%) 250 mm to 300 mm Capping

It was therefore decided to construct the trial sections using a capping design below that required for a worst case value of 2% CBR, whilst still ensuring a sufficient thickness of capping to allow satisfactory trafficking. A design thickness of 450 mm was chosen.

2.0 TRIAL 3. MOUNTSORREL

2.1 Site Investigation

A site investigation was undertaken in September 1997. It included the digging of 4 trial pits (two of which were immediately adjacent to the final location of the trials). A series of Dynamic Cone Penetrometer (DCP) tests was carried out. Bulk samples of the underlying soil and one undisturbed (U100) sample were taken from each pit for laboratory testing.

The trial pits showed the ground to consist of two strata across the site, essentially 300 mm of topsoil overlying a stiff red brown silty CLAY (weathered Mercia Mudstone), containing occasional Granodiorite gravel, the gravel becoming more prevalent with depth. Occasional Granodiorite cobbles were found at 1.5 m and below.

Hand shear vane tests indicated the clay to have an undrained shear strength of approximately 100 kPa. The DCP readings along the site showed the clay to have a penetration resistance of approximately 42 mm/blow, to a depth of 700 mm, reducing to
20 mm/blow at greater depths. These readings equate to CBR values of 3.5 % and 9 % respectively.

Laboratory CBR tests (BS1377 Part 4 1990) were performed on remoulded samples of the clay from the bulk soil samples taken adjacent to the trial road site. CBR values of 2.4 %, 4%, 5% and 12% were measured on samples taken from 1.0m and 1.5m below ground level. Plasticity tests indicated the clay to have Atterberg limits of PL=19.1%, and LL=40.4%, which indicate the material to be a clay of low plasticity, BS 5930, 1981. The specific gravity was determined as 2.6 Mg/m$^3$ (BS1377 Part 2).

Repeated load triaxial tests were performed on the U100 samples. The test show the material to have an undrained shear strength range of 55 to 120 kPa, indicating the material to be a firm to stiff clay.

### 2.2 Design of the Trial Road Foundations

The design thickness of the foundation was based upon the standard design charts (HD25/94 DMRB Vol. 7), and an analytical approach using linear elastic analysis. The triaxial stiffness data was used to design a capping thickness for a target top of foundation stiffness of 50MPa measured with the TFT at 100kPa applied stress (assuming a capping stiffness of 100 MPa for both capping materials). The design were also assessed using field measure data from the GDP.

DCP (42 mm/blow),
(50 MPa at 70 kPa, deviator Stress),
Average remoulded laboratory CBR (4%),

300 mm Capping
100 mm Capping
300 mm Capping (HD25/94 DMRB Vol. 7),

In addition, tests on the subgrade at the site investigation stage using the GDP and DCP and from laboratory tests gave the following additional range of design values.

GDP Stiffness (range 11 to 15 MPa),
600 to 500 mm Capping,
DCP equated to CBR (range 2.5% to 7%), 500 mm to 150 mm Capping
Plasticity Index equated to CBR (4%), 300 mm Capping
Remoulded laboratory CBR range (2.4 to 12%), 600 mm to 0 mm Capping

From these data a design thickness of 300 mm was chosen for both the trial sections.

The justification for this thickness was based on the observation that the average values tended to indicate that 300 mm would be adequate, the only strong indicator of the thicker capping being the dynamic plate stiffness (GDP). The triaxial data showed significant variability across the site at low deviator stress assessed in the design analysis so a conservative design was adopted with variation about the design thicknesses assessed on site.
APPENDIX C
APPENDIX C

Presentation of Data Chapter 5 Tables

Table C5.1, Summary of Results from Laboratory Tests on Granular Materials Used in the Field Trials

Table C5.2, Results from Laboratory CBR Tests on Subgrade Samples from Field Trial Sites

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Table C5.4, Summary Table of Repeated Load Triaxial Test Data on Undisturbed Samples

Table C5.5, Summary Table of Repeated Load Triaxial Test Data on Remoulded Samples

Table C5.6, Summary Table of Repeated Load Triaxial Test Data on Remoulded Equilibrium Samples

Table C5.7, Summary Table of Repeated Load Triaxial Test Data on Undisturbed Samples (Conditioned)

Table C5.8, Summary Table of Repeated Load Triaxial Test Data on Remoulded Samples (Conditioned)

Table C5.9, Summary Table of Repeated Load Triaxial Test Data, Repeatability Tests

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Table C5.19 Stiffness Measured on each Layer of Capping During Construction of Capping at Trial Two (Bardon)

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Table C5.21 Stiffness Measured with the GDP, TFT and FWD on Sub-base During Compaction Trial Two (Bardon)

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Table C5.26, Top of Granodiorite Capping Stiffness Measured with the GDP and FWD during compaction of the final layer (Trial Three)

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Table C5.28, Corrected Capping and Sub-base, NDG Dry Density on the Granodiorite Capping Trial (Trial Three)

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Table C5.31, Rate of DCP Penetration Through Various Layers of Materials on the Granodiorite Capping Trial (Trial Three)

Table C5.32, Stiffness Measured at Various Applied Stresses During the Granodiorite Trial (Trial Three)

Table C5.33, Stiffness Measured on Each Layer of Capping During Construction of the Limestone Capping

Table C5.34, Top of Limestone Capping Stiffness Measured with the GDP, TFT and FWD During Compaction of the Final Layer of Capping, and Reconstructed Limestone
Table C5.35, Corrected Capping and Sub-base, NDG Dry Density on the Limestone Capping Trial (Trial Three)

Table C5.36, Sub-base Stiffness Measurements Made with the GDP, TFT and FWD During Compaction and Eight Months After Completion of the Limestone Trial

Table C5.37, Rate of DCP Penetration Through Various Layers of Materials on the Limestone Capping Trial (Trial Three)

Table C5.38, Stiffness Measured at Various Applied Stresses During the Limestone Trial at Trial Three
Notes for Appendix C Chapter Five Tables

Notes for Repeated Load Traxial Test Data

<table>
<thead>
<tr>
<th>Sample</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bardon</td>
<td>Sample of weather Mercia Mudstone collected from Bardon Hill (Not from the trial site)</td>
</tr>
<tr>
<td>AIM</td>
<td>Sample from the AIM road widening project</td>
</tr>
<tr>
<td>Boston</td>
<td>Sample of a clayey silt</td>
</tr>
<tr>
<td>Hat SI</td>
<td>Sample from the site investigation at Hathern (Trial 1)</td>
</tr>
<tr>
<td>Hat Bay</td>
<td>Sample collected from Bay X during the Hathern Trial</td>
</tr>
<tr>
<td>Ox</td>
<td>Sample of Oxford Clay collected from a brick pit</td>
</tr>
<tr>
<td>m/s</td>
<td>Sample collected from Mountsorrel (Trial 3) during the trial</td>
</tr>
<tr>
<td>m/s U</td>
<td>Sample collected from Mountsorrel during the site investigation</td>
</tr>
<tr>
<td>Lon</td>
<td>Sample of London Clay collected from a brick pit</td>
</tr>
<tr>
<td>+</td>
<td>Two similar samples mixed</td>
</tr>
<tr>
<td>w</td>
<td>Sample with water added to provide a further material to assess</td>
</tr>
<tr>
<td>Rpt</td>
<td>Manufactured soil sample mixed from various samples for repeatability testing</td>
</tr>
</tbody>
</table>

Stiffness at 100 kPa  Stiffness measured at 100kpa deviator stress
* Estimated value
Range at 10kPa Comparison of the data from the two on-sample strain gauges at 10kpa deviator stress
1 value Only one on-sample gauge functioning

Notes for Field measured Stiffness Data Tables

All data reported as average values of stiffness or density measured within each bay at any given stage during construction.

FWD (Single) FWD stiffness measured at a point where only one set of tests performed
FWD (Three) FWD stiffness measured at a point where all three devices used
Std. Dev Standard deviation of the value reported

All rutting data given in mm unless stated otherwise
Table C5.1. Summary of Results from Laboratory Tests on Granular Materials Used in the Field Trials

<table>
<thead>
<tr>
<th>Sample</th>
<th>Optimum Water Content</th>
<th>Maximum Compacted Density</th>
<th>% Material Removed for PSSR Test</th>
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<tr>
<td><strong>Hather</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6F2 Porphyritic Andesite</td>
<td>3.8%</td>
<td>2.062 Mg/m³</td>
<td>3.3%</td>
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<tr>
<td>40mm Down Porphyritic Andesite</td>
<td>5.2%</td>
<td>2.202 Mg/m³</td>
<td>3.2%</td>
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<tr>
<td>6F2 Oolitic Limestone</td>
<td>2.2%</td>
<td>1.829 Mg/m³</td>
<td>2.6%</td>
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<tr>
<td><strong>Bardon</strong></td>
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<td></td>
</tr>
<tr>
<td>6F1 Porphyritic Andesite</td>
<td>5.5%</td>
<td>2.481 Mg/m³</td>
<td>3.0%</td>
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<tr>
<td>Type 1 Porphyritic Andesite</td>
<td>5.0%</td>
<td>2.077 Mg/m³</td>
<td>3.1%</td>
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<tr>
<td><strong>Montsorrel</strong></td>
<td></td>
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<td></td>
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<tr>
<td>40mm Down Grano-Diorite</td>
<td>2.37%</td>
<td>1.694 Mg/m³</td>
<td>4.0%</td>
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<tr>
<td>6F2 Oolitic Limestone</td>
<td>6.99%</td>
<td>1.896 Mg/m³</td>
<td>3.9%</td>
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<td>Type 1 Grano-Diorite</td>
<td>5.67%</td>
<td>2.004 Mg/m³</td>
<td>6.6%</td>
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Table C5.2, Results from Laboratory CBR Tests on Subgrade Samples from Field Trial Sites

**Hathern**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Remoulded CBR %</th>
<th>Water Content %</th>
<th>Dry Density (Mg/m³)</th>
<th>Bulk Density (Mg/m³)</th>
<th>Plastic Limit %</th>
<th>Liquid Limit %</th>
<th>Plasticity Index %</th>
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**Bardon**

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<thead>
<tr>
<th>Sample</th>
<th>State</th>
<th>Remoulded CBR %</th>
<th>Water Content %</th>
<th>Dry Density (Mg/m³)</th>
<th>Bulk Density (Mg/m³)</th>
<th>Specific Gravity Gₛ</th>
<th>Saturation Sₛ</th>
<th>Plastic Limit %</th>
<th>Liquid Limit %</th>
<th>Plasticity Index %</th>
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<tr>
<td>1</td>
<td>Remoulded</td>
<td>1.1</td>
<td>17.0</td>
<td>1.839</td>
<td>2.196</td>
<td>1.209</td>
<td>16.9</td>
<td>27.1</td>
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<td>Soaked</td>
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<td>17.0</td>
<td>1.841</td>
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<td>2</td>
<td>Remoulded</td>
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<td>Soaked</td>
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<td>Remoulded</td>
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**Mountsorrel**

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<tr>
<th>Sample</th>
<th>Remoulded CBR %</th>
<th>Water Content %</th>
<th>Dry Density (Mg/m³)</th>
<th>Bulk Density (Mg/m³)</th>
<th>Specific Gravity Gₛ</th>
<th>Saturation Sₛ</th>
<th>Plastic Limit %</th>
<th>Liquid Limit %</th>
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<td>2</td>
<td>9.5</td>
<td>19.9</td>
<td>1.654</td>
<td>1.984</td>
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<td>18.9</td>
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<td>1.622</td>
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<td>19.1</td>
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<td>19.1</td>
<td>40.4</td>
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Table C5.3 Percentage Error Due to Instrumentation Precision for the Strain Gauges used in the Repeated Load Triaxial Test

<table>
<thead>
<tr>
<th>Stress</th>
<th>300 Mpa</th>
<th>250 Mpa</th>
<th>200 Mpa</th>
<th>150 Mpa</th>
<th>125 Mpa</th>
<th>100 Mpa</th>
<th>75 Mpa</th>
<th>50 Mpa</th>
<th>40 Mpa</th>
<th>30 Mpa</th>
<th>20 Mpa</th>
<th>10 Mpa</th>
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<tbody>
<tr>
<td>5 kPa</td>
<td>104.66</td>
<td>87.21</td>
<td>69.77</td>
<td>52.33</td>
<td>43.61</td>
<td>34.89</td>
<td>26.16</td>
<td>17.44</td>
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<td>10.47</td>
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<td>10 kPa</td>
<td>52.33</td>
<td>43.61</td>
<td>34.89</td>
<td>26.16</td>
<td>21.80</td>
<td>17.44</td>
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<td>8.72</td>
<td>5.61</td>
<td>4.65</td>
<td>3.49</td>
<td>2.33</td>
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<tr>
<td>15 kPa</td>
<td>34.89</td>
<td>29.07</td>
<td>23.26</td>
<td>17.44</td>
<td>14.54</td>
<td>11.63</td>
<td>8.72</td>
<td>5.61</td>
<td>4.65</td>
<td>3.49</td>
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<td>1.16</td>
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<td>20 kPa</td>
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<td>17.44</td>
<td>13.08</td>
<td>10.90</td>
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Table C5.4, Summary Table of Repeated Load Triaxial Test Data on Undisturbed Samples

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(*=Estimated)
### Table C5.5, Summary Table of Repeated Load Triaxial Test Data on Remoulded Samples

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</tr>
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### Table C5.6, Summary Table of Repeated Load Triaxial Test Data on Remoulded Equilibrium Samples

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<th>Shear Strength (Cu) kPa</th>
<th>Threshold Stress kPa</th>
<th>Water Content %</th>
<th>Bulk Density Mg/m3</th>
<th>Plasticity Data PL %</th>
<th>LL %</th>
<th>PI %</th>
<th>Stiffness at 100 kPa MPa</th>
<th>Stiffness Range at 10 kPa MPa</th>
<th>Specific Gravity Comment</th>
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<td>27</td>
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<td>28</td>
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Table C5.7, Summary Table of Repeated Load Triaxial Test Data on Undisturbed Samples (Conditioned)

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<th>Shear Strength (Cu) kPa</th>
<th>Threshold Stress @ 1% Strain kPa</th>
<th>Max Curve kPa</th>
<th>Water Content %</th>
<th>Bulk Density Mg/m3</th>
<th>Plasticity Data PL %</th>
<th>LL %</th>
<th>PI %</th>
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<th>Stiffness Range at 10 kPa MPa</th>
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<th>Comment</th>
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<td>150</td>
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<td>54.9</td>
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<td>After Conditioning</td>
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Table C5.8, Summary Table of Repeated Load Triaxial Test Data On Remoulded Samples (Conditioned)

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<th>Water Content %</th>
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<th>Plasticity Data PL %</th>
<th>LL %</th>
<th>PI %</th>
<th>Stiffness at 100 kPa MPa</th>
<th>Stiffness Range at 10 kPa MPa</th>
<th>Specific Gravity at 10 kPa</th>
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<td>148.0</td>
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<td>28.0</td>
<td>1.815</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>96</td>
<td>74-118</td>
<td>-</td>
<td>London 2 wetted</td>
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<td>Lon2/7</td>
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<td>50.4</td>
<td>65</td>
<td>32.5</td>
<td>1.844</td>
<td>-</td>
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<td>London 2+7 mixed</td>
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<td>M/s 3/6</td>
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<td>1.998</td>
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Table C5.9, Summary Table of Repeated Load Triaxial Test Data, Repeatability Tests

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<th>Threshold Stress @ 1% Strain kPa</th>
<th>Max Curve kPa</th>
<th>Water Content %</th>
<th>Bulk Density Mg/m³</th>
<th>Plasticity Data PL %</th>
<th>LL %</th>
<th>PI %</th>
<th>Stiffness at 100 kPa MPa</th>
<th>Stiffness at 10 kPa MPa</th>
<th>Range at 10 kPa MPa</th>
<th>Specific Gravity</th>
<th>Comment</th>
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Table C5.10 Repeatability Tests with the Dynamic Plate Stiffness Measuring Devices on Rubber Sheets

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<td>FWD (Single) (MPa)</td>
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Table C5.12, Subgrade Dynamic Cone Penetrometer Tests, Trial 1 (Hathern Trial)

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<th>Row C mm/Blow</th>
<th>Average mm/Blow</th>
<th>Error mm/Blow</th>
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Table C5.13, Capping Stiffness Measurements at Trial One (Hathern), Single at Three Measurements at a Point

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| 6F2 Limestone             |
|--------------------------|--------|--------------|---------|--------------|---------|--------------|---------|--------------|---------|--------------|---------|
|                          | 4      | 62.15        | 17.56   | 68.13        | 9.13    | 67.43        | 8.53    | 29.78        | 3.06    | 36.49        | 8.68    |
|                          | 8      | 56.95        | 9.32    | 59.86        | 9.34    | 62.25        | 12.81   | 36.67        | 1.99    | 40.09        | 5.17    |
|                          | 16     | 61.87        | 5.65    | 62.38        | 4.39    | 67.64        | 6.74    | 41.80        | 2.18    | 53.91        | 0.54    |

| 6F1 Porphyritic Andesite |
|--------------------------|--------|--------------|---------|--------------|---------|--------------|---------|--------------|---------|--------------|---------|
|                          | 4      | 48.00        | 13.07   | 34.76        | 12.89   | 44.24        | 7.40    | 22.39        | 2.42    | 25.21        | 2.27    |
|                          | 8      | 56.52        | 11.80   | 56.82        | 3.18    | 54.03        | 10.38   | 24.63        | 1.92    | 29.68        | 3.83    |
|                          | 16     | 67.60        | 10.04   | 57.45        | 4.29    | 49.10        | 8.16    | 31.53        | 2.25    | 34.49        | 4.92    |
Table C5.14, Repeat and Long-term Capping Stiffness Measurement Trial 1 (Hathern Trial) (all data combined)

### 6F2 Porphyritic Andesite

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Table C5.16 Monitoring of the Formation of Rutting on Trial One (Hathern Trial)

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### Table C5.19 Stiffness Measured on each Layer of Capping During Construction of Capping at Trial Two (Bardon)

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### Table C5.20, Corrected Capping and Sub-base, NDG Dry Density Trial Two (Bardon)

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Table C5.22, Rate of DCP Penetration Through Various Layers of Materials on the
Trial Two (Bardon)

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Table C5.23 Measurements of Rut Formation During Trafficking of Trial Two (Bardon)

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Table C5.24, Stiffness Measured at Various Applied Stresses on Trial Two (Bardon)

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Average: 40.31 27.20

Summary: 63.10 29.40

Average: 98.36 31.50
Table C5.25, Stiffness Measured on Each Layer of Capping During Construction of the Granodiorite Capping (Trial Three)

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Table C5.26 Top of Granodiorite Capping Stiffness Measured with the GDP and FWD during Compaction of the Final Layer (Trial Three)

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Table C5.27, Repeat Stiffness Measurements made with the GDP and TFT on The Granodiorite Capping During Extra Compaction (Trial Three)

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<th>Repeat Thickness (MM)</th>
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<td>11.33983</td>
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<td>3.95</td>
<td>12.82</td>
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<th>Prior</th>
<th>Sub-base (MPa)</th>
<th>Std Dev (MPa)</th>
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Table C5.28, Corrected Capping and Sub-base, NDG Dry Density on the Granodiorite Capping Trial (Trial Three)

<table>
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<tr>
<th>Capping</th>
<th>Density 1 Pass</th>
<th>Std Dev</th>
<th>Density 2 Passes</th>
<th>Std Dev</th>
<th>Density 5 Passes</th>
<th>Std Dev</th>
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<td>Mg/m³</td>
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<td>Mg/m³</td>
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<td>0.053</td>
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<td>1.603</td>
<td>0.041</td>
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</table>

**Sub-base**

| Bay 1   | 450            | 1.878   | 0.023            |         | 1.922            | 0.015   |
| Bay 2   | 300            | 1.914   | 0.076            |         | 1.981            | 0.041   |
| Bay 3   | 150            | 1.906   | 0.018            |         | 1.951            | 0.006   |

Table C5.29, Measurements of the Ruts on the Granodiorite Capping During Trafficking (Trial Three)

<table>
<thead>
<tr>
<th>Capping</th>
<th>Outer Thickness 2 Passes</th>
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<th>14 passes</th>
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<td>(mm) (mm) (mm)</td>
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<td>-16</td>
<td>-27</td>
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<tr>
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<td>-31</td>
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<tr>
<td>Bay 3</td>
<td>150</td>
<td>-18</td>
<td>-51</td>
</tr>
</tbody>
</table>

**Inner**

| Bay 1   | 450 |-12 | -16 | -25 |
| Bay 2   | 300 |-9  | -18 | -26 |
| Bay 3   | 150 |-12 | -27 | -49 |
Table C5.30 Sub-base Stiffness Measurements Made with the GDP, TFT and FWD during Compaction and Eight Months After Completion Granodiorite Trial (Trial Three)

<table>
<thead>
<tr>
<th>Capping</th>
<th>Thickness (mm)</th>
<th>1 Pass (MPa)</th>
<th>Std Dev (MPa)</th>
<th>2 Passes (MPa)</th>
<th>Std Dev (MPa)</th>
<th>5 Passes (MPa)</th>
<th>Std Dev (MPa)</th>
<th>8 Months (MPa)</th>
<th>Std Dev (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
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Table C5.31, Rate of DCP Penetration Through Various Layers of Materials on the
Granodiorite Capping Trial (Trial Three)

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<th>Capping</th>
<th>Std Dev</th>
<th>Sub-base</th>
<th>Std Dev</th>
<th>8 Months</th>
<th>Std Dev</th>
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Table C5.32, Stiffness Measured at Various Applied Stresses During the Granodiorite Trial (Trial Three)

<table>
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<tr>
<th>Bay</th>
<th>Thickness (mm)</th>
<th>Capping Stress (kPa)</th>
<th>Capping Stiffness (Mpa)</th>
<th>Sub-base Stress (kPa)</th>
<th>Sub-base Stiffness (Mpa)</th>
<th>8 Months Stress (kPa)</th>
<th>8 Months Stiffness (Mpa)</th>
<th>FWD Stress (kPa)</th>
<th>FWD Stiffness (Mpa)</th>
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Table C5.33, Stiffness Measured on Each Layer of Capping During Construction of the Limestone Capping (Trial Three)

<table>
<thead>
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<th>Capping Thickness</th>
<th>Subgrade Std (MPa)</th>
<th>150 mm Std (MPa)</th>
<th>300 mm Std (MPa)</th>
<th>450 mm Std (MPa)</th>
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Table C5.34 Top of Limestone Capping Stiffness Measured with the GDP, TFT and FWD During Compaction of the Final Layer of Capping, and Reconstructed Limestone (Trial Three)

<table>
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<tr>
<th>Capping Thickness</th>
<th>1 Pass Std (MPa)</th>
<th>2 Passes Std (MPa)</th>
<th>5 Passes Std (MPa)</th>
<th>Recon LS Std (MPa)</th>
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<td>19.92</td>
<td>15.59</td>
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<td>TFT</td>
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<td>FWD</td>
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Table C5.35, Corrected Capping and Sub-base, NDG Dry Density on the Limestone Capping Trial (Trial Three)

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<th>Capping</th>
<th>Density</th>
<th>1 Pass</th>
<th>Std Dev</th>
<th>Density</th>
<th>3 Passes</th>
<th>Std Dev</th>
<th>Density</th>
<th>5 Passes</th>
<th>Std Dev</th>
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<tr>
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<td>Mg/m³</td>
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<td>Mg/m³</td>
<td></td>
<td></td>
<td>Mg/m³</td>
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</tr>
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<th>Std Dev</th>
<th>Density</th>
<th>2 Passes</th>
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<td>Mg/m³</td>
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Table C5.36, Sub-base Stiffness Measurements Made with the GDP, TFT and FWD During Compaction and Eight Months After Completion of the Limestone Trial (Trial Three)

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<th>Std Dev (MPa)</th>
<th>2 Passes (MPa)</th>
<th>Std Dev (MPa)</th>
<th>5 Passes (MPa)</th>
<th>Std Dev (MPa)</th>
<th>8 Months (MPa)</th>
<th>Std Dev (MPa)</th>
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<tr>
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Table C5.37, Rate of DCP Penetration Through Various Layers of Materials on the Limestone Capping Trial (Trial Three)

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Table C5.38, Stiffness Measured at Various Applied Stresses During the Limestone Trial at Trial Three

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