ROUGHTON INTERNATIONAL

In association with
THE UNIVERSITY OF BIRMINGHAM
and
THE UNIVERSITY OF NOTTINGHAM

Assessing Materials for Low Cost Roads

DFID Department for International Development

February 2000

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Report III
Investigation of Material Assessment Apparatus

Appropriate and Efficient Maintenance of Low Cost Rural Roads

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February 2000

Department for International Development
Knowledge and Research (KaR) Programme

Project Title:	Appropriate and Efficient Maintenance of Low Cost Rural Roads
DFID Project Reference:	R6852
Subsector:	*Transport*
Theme:	T2
Element C:	Assessing Materials for Low Cost Roads
Date:	*February 2000*

This document is an output from a project funded by the UK Department for International Development (DFID) for the benefit of developing countries. The views expressed are not necessarily those of DFID

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ACKNOWLEDGEMENTS

This report was prepared by Mr A. Dawson, Dr I. Alobaidi and Ms K. Adams. of the University of Nottingham.

A great many people have made significant contributions to the research for this report. Their co-operation is much appreciated and gratefully acknowledged: Dr Baden Clegg (Australia), Jim Crandell (Australia), Matthew Frost (UK), Matthew Brough (UK), Angela Kabiru, (ILO ASIST), David Stiedl (UK), Dr Harold Bofinger, (South Africa), Mike Carr, (UK), Simon Gillett, (UK), Simon Ovens (UK), Jason Redgers, (UK), Eng Samuel Bagonza, (Uganda), David McNaughton, (Fiji), Dhamendra Prasad (Fiji), JoneTbakala (Fiji). Mr Nick Thom (UK).

EXECUTIVE SUMMARY

This element of the project was concerned with the evaluation and development of a pavement and material assessment apparatus in the context of low cost rural feeder roads in developing countries. The aim was to develop a simple method and procedure for the assessment of the quality of those roads. It was proposed that a simple apparatus be adopted which could be made locally at an affordable cost rather than using imported specialist apparatus, thus placing it within the reach and repair capability of poor rural district maintenance authorities.

It was firstly necessary to select an appropriate piece of equipment and then modify it. After initial investigation work, the Dynamic Cone Penetrometer (DCP) was chosen as the equipment to be modified for assessment of roads *in-situ* and of aggregate materials at source.

It was judged that an ideal assessment procedure for the assessment of low cost rural roads should, as far as practical, comprise a method of assessing strength, stiffness (resilient modulus) and permanent deformation characteristics of the road, in-situ. The same, or similar, procedures should be applicable as a predictive tool, to assess aggregate materials at source (from borrow pits). By this means a simple evaluation technique of the basic engineering properties of the road construction material in the present works and future construction was planned.

Laboratory and field investigations in the UK allowed the modification of the apparatus to be undertaken. The principle development was the use of the ‘flat’ tips of various diameters on the DCP apparatus. This was then followed by a number of visits to developing countries to assess the equipment on different soils and establish suitable procedures.

Analysis of the data collected (both from the UK and overseas) yielded some useful and applicable results. It was possible to establish a measure of stiffness by using a modified version of the dynamic cone penetrometer in-situ. Additionally, the version of the equipment developed for testing materials at source yielded some promising results, once an appropriate specimen preparation procedure was developed.

The research demonstrated good applicability of the new techniques in certain circumstances. On crushed rock materials and in certain lateritic soils it could be implemented directly. However, the evidence for its applicability in a generic manner was not conclusive, suggesting that there may be merit in further research work in the area. Unfortunately a significant finding was the problem of introducing new ideas and equipment in developing countries due to institutional issues. Such issues are discussed towards the end of the report.

To facilitate use of the new equipment and its take-up for the assessment of material in borrow pits, a guideline document has been produced for people wishing to trial the equipment.

REPORT III - INVESTIGATION OF MATERIALS ASSESSMENT APPARATUS

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INVESTIGATION OF MATERIAL ASSESSMENT APPARATUS

1. AIM AND PURPOSE

The aim of this element was to develop a simple method and procedure for the assessment of gravel rural roads. It was expected that an in-situ aggregate assessment apparatus (which could be used at borrow pits to check sources and on aggregate and soil layers in the road to assess existing or post-maintained condition), could be modified from some existing equipment. It was proposed that the simple apparatus be made locally at an affordable cost rather than comprise an imported specialist apparatus, thus placing it within reach and repair capability of poor rural district maintenance authorities. The first aim was therefore the selection and adoption of an available piece of *in-situ* testing apparatus, while the second aim was the modification of the selected equipment. The Dynamic Cone Penetrometer (DCP) was chosen as the equipment to be modified for assessment of roads *in-situ* and aggregate materials at source, the reasons are discussed in Section 4.2.4.1.

The purpose of assessing roads in-situ was, as far as practical, to develop a method of assessing strength, stiffness (resilient modulus) and permanent deformation characteristics of the road. It was also intended to use the apparatus as a predictive tool, to assess aggregate materials from borrow pits.

2. INTRODUCTION

Road maintenance is an issue that has been looked at by many researchers, organisations and donors over the last 30 years at least. In more recent years, road maintenance in developing countries has featured more and more in proposals funded by donors. The issues tackled may be technical, institutional or external, (as in the 'Brooks pyramid' – OSRN 15 1998[1]), but they are generally intrinsically linked. When a project tries to tackle only one aspect, while endeavouring to ignore the other issues, such as approaching a technical issue while ignoring the institutional ones, problems can arise. For example in this case, it was intended to develop a piece of apparatus that would allow easy assessment of materials used for unpaved roads to be undertaken at a local level. However, no matter how good the piece of equipment, nor how valid this rationale, the equipment would not necessarily be used due to constraints within organisations. This work has concentrated on the development and research surrounding the technical ideas and concepts of the apparatus, while the institutional and external issues which might ultimately hinder the equipment from being used have been addressed separately in Report I to the institutional issues which may affect implementation and these are discussed later in the report.

[1] OSRN 15, Guidelines for the design and operation of road management systems, 1998, p10, TRL

3. BACKGROUND TO THE WORK – ROAD MAINTENANCE

3.1 Unpaved and Gravel Roads

Unpaved roads include those which are constructed from ‘earth’ and are not engineered, but often develop through use, starting off as well trodden tracks and, as various forms of vehicles start to use them, they encroach on the surrounding vegetation until they resemble a road. The number of vehicles using such roads is often very low, yet they do often provide important access to facilities for local communities. Unfortunately they can often become impassable in the wet. The Draft TRH 20:1990[3] defines that an earth road as such if no imported gravel is used, instead the natural material is compacted once the vegetation has been cleared, either by design (an engineered earth road) or by practice and use. In contrast gravel roads do have an imported layer. The road is normally of a specific width which will depend upon the local advice in the country. The width, camber, drainage and materials will no doubt depend on the class of the road, and be country dependent. It should also be noted that the terminology used in road classification is country dependent, but generally it could be assumed that a main road would join the main towns or cities, and the feeder roads would feed traffic from the villages to the nearest main route into the town.

It is not intended to discuss the classification of roads, but simply to make the reader aware that unpaved and gravel roads may be crucial links to places of importance, or they may be crucial links between a community and, for instance, the local school. As a result the specifications for the road will, in general, reflect the perceived importance of the route and the level of traffic it is likely to take. Additionally the specification for the type of material to be used when building the road will reflect the importance of the route. Whether or not the road is situated in an urban or rural setting will also be of significance.

‘Gravel roads have a designed layer of imported material which is typically constructed to a specified standard and width and provides an all-weather surface. The vertical and horizontal alignment is generally upgraded to appropriate standards. Maintenance of gravel roads is carried out on a more regular and systematic basis and a higher level of service is obtained, although the roughness varies considerably with time and depends significantly on the maintenance activity’ – (Draft TRH:20 1990)[4].

‘Roads which have less than 25mm thickness of granular surfacing which has been transported onto the site are often referred to as ‘natural’ surfaced roads’ (Ferry 1986)[5].

For the purpose of the current investigation, gravel roads in a rural setting are the main focus of interest. The level of traffic on gravel roads will obviously depend on the class of the road and its setting. It may be trafficked by 100 ox carts per day or 300 cars and trucks per day. Generally a gravel road is likely to have a traffic level of under 300 vehicles per day (Draft TRH:20 1990)[6], or the average traffic level may be regarded as between 20 and 200 vehicles per day (ARRB 1993)[7].

With regard to the thickness of the unpaved road pavement, it will depend on the climate – in particular whether or not the country is dry or wet and the soil strength.

[3] DRAFT TRH 20:1990, The structural design, construction and maintenance of unpaved roads, Committee of state road authorities, 1990.
[4] Draft TRH:20 1990, The structural design, construction and maintenance of unpaved roads., Committee of State Road Authorities, p 2.
[5] Ferry, A G., Unsealed Roads – A manual of repair and maintenance for pavements, RRU Technical Recommendation TR8, National Roads Board, New Zealand, 1986, p1-2.
[6] The structural design, construction and maintenance of unpaved roads. Draft TRH:20 1990, Committee of State Road Authorities, p 2.
[7] Unsealed Roads Manual – Guidelines to good practice, ARRB, 1993, p3.2.

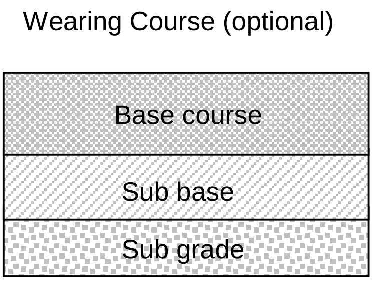
3.1.1 What Constitutes a Good Gravel Road?

According to the Unsealed Roads Manual (1986): *'A pavement on a well-designed unsealed road would have the following:*

- Sufficient total thickness and strength in order to spread the loading safely onto the subgrade without overstress;
- Be composed of a locally available materials in such a way as to minimise overall costs;
- A waterproof surface and camber capable of shedding most rainwater;
- A dense graduation of the materials including an amount of cohesive binder (clay) which is slow to respond to changes in weather, whether wet or dry;
- A uniform non-skid surface for traffic safety;
- Relative freedom from dust;
- Low maintenance and material placement cost;
- A smooth surface producing economical vehicle operating costs;
- Reasonable durability;
- Suited to local environment and traffic requirements' (Ferry 1986)[8].

The Unsealed Roads Manual gives the following advice with regard to the thickness of the gravel pavement: *'There are two approaches which have been adopted in deciding the first stage of pavement thickness: provide a nominal single course, (e.g.: 100 to 150mm of pavement) or provide a designed pavement thickness 50 to 100mm less than that required for a sealed road, with a minimum thickness of 100 to 150mm'*[9]. The thickness will generally be a function of the quality of the subgrade (usually expressed in terms of the index value, CBR – see section 3.4.2) and the expected traffic level.

Figure 3-1 - An Unsealed Road



The approach used will be dependent on the climate, amount of rain, traffic, number of heavy vehicles and materials available. It is not intended to discuss in detail the thickness design of gravel pavements, for which the reader is directed to the following publications: Unsealed Roads Manual – Guidelines to good practice (1993)[10], Draft TRH 20 (1990)[11], Unsealed Roads – A manual of repair and maintenance for pavements (1986)[12]. Generally an unsealed road will consist of the subgrade, possibly a subbase (depending on the strength of the subgrade) and a base course. See Figure 3-1 for a basic schematic of a pavement structure. A running course (wearing course) may also be added, or may develop naturally (a thin approximately 12 mm layer of broken stone or gravel)[13]. The compacted thickness of the 'top' layer which can be laid directly onto the prepared subgrade can vary

[8] Ferry, A G., Unsealed Roads – A manual of repair and maintenance for pavements, RRU Technical Recommendation TR8, National Roads Board, New Zealand, 1986, p1-5.
[9] Unsealed Roads Manual – Guidelines to good practice, ARRB, 1993, p3.3.
[10] Unsealed Roads Manual – Guidelines to good practice, ARRB, 1993.
[11] Draft TRH:20 1990, The structural design, construction and maintenance of unpaved roads., Committee of State Road Authorities.
[12] Unsealed Roads manual (RRU Technical Recommendations TR8) – A manual of repair and maintenance for Pavements, A.G. Ferry, 1986.
[13] Unsealed Roads manual (RRU Technical Recommendations TR8) – A manual of repair and maintenance for Pavements, A.G. Ferry, 1986, p4-13.

from 100 to 150 mm (where a top layer may be taken to mean all the layers which are not sub grade).

It should not be forgotten that in many countries, unsealed roads have developed through use, having started as a well trodden track, the vegetation may have slowly been cut back to allow animal drawn vehicles to use the route and eventually motorised vehicles. These roads start off as earth, but can only be used in relatively dry conditions, as they usually become impassable in bad weather. If they are sufficiently important the local engineer may apply a gravel surface to improve access. The obvious problem with such roads is that they have not been designed and therefore they often do not have a designed camber, or any drainage.

3.2 Maintenance of Gravelled Rural Roads

In theory the assessment of rural roads in developing countries is carried out by two methods. The first one, which is the most common, is visual inspection in the field[14]. The second method is to quantify the apparent defects in the road such as measurements of rutting, change in camber and loss in gravel thickness etc. Overseas Road Note 2 gives details about how to perform these measurements[15]. A condition assessment which relies on physical measurement is unlikely to be undertaken on low cost roads, simply because the money is not generally available in developing countries. Assessments of maintenance needs, at the low cost end, usually rely on the experience of the local staff – and the organisation's policy regarding the frequency with which the activities should be undertaken. Unfortunately, maintenance is not always carried out to the specified frequencies, due to lack of money, availability of staff, equipment, materials and a whole mix of institutional issues, this is explained in much greater depth in Report I.

Guidance on the frequency of maintenance operations is provided in many documents which are widely available, and also in documents such as maintenance manuals which are country specific and not so widely available. It should be remembered, of course, that just because the maintenance manual states the frequency of routine and periodic maintenance, it is unlikely that an 'ideal' scenario exists and it is more likely that the maintenance engineer has to use his cunning, judgement and planning to allow him to maintain the roads according to his budget.

The reader is advised to consult the following publications for examples of the frequency of maintenance operations for unsealed roads: Overseas Road Note 1, TRL (1985)[16], Overseas Road Note 2, TRL (1987)[17], Draft TRH 20 (1990)[18], RRU Technical Recommendation TR8 (1986)[19] and many others besides (each country normally has its own manual containing such information). It should obviously be noted that the frequency will be dependent on, subgrade, climate, terrain, equipment available, budget and available material. In principle, any country's decision to maintain and of the method and frequency to be used, is based on the economical benefit it will deliver. In practice this underlying principle may be partly obscured by the procedure adopted.

3.3 Maintenance Techniques

Maintenance may be regarded as either being equipment or labour intensive. The choice of the methods will depend upon what is available in the country, the funding, culture, availability of labour (which may be greater in some rural areas than in others), climate,

[14] Jones, T.E. (1987) "Optimum maintenance strategies for unpaved roads in Kenya", PhD thesis, The University of Birmingham.
[15] TRRL (1981) "Maintenance techniques for district engineers", Overseas Road Note 2, Department of Transport, UK.
[16] TRRL (1985) "Maintenance techniques for district engineers", Overseas Road Note 2, Department of Transport, UK.
[17] TRRL (1987) "Maintenance management for district engineers", Overseas Road Note 1, Department of Transport, UK.
[18] Draft TRH:20 1990, The structural design, construction and maintenance of unpaved roads., Committee of State Road Authorities.
[19] Unsealed Roads manual (RRU Technical Recommendations TR8) – A manual of repair and maintenance for Pavements, A.G. Ferry, 1986.

topography etc., and it is not proposed to undertake a lengthy discussion about which methods are most appropriate, as this has been covered extensively by projects such as the MART Initiative (a current project ‘The Management of Appropriate Road Technology’ being undertaken on behalf of DFID by the Construction and Enterprise Unit of Loughborough University in collaboration with Intech associates and I.T. Transport). Reference should be made to The Labour-Based Technology Source Book[20] for up-to-date publications in the field.

Maintenance can be split into those activities which are routine, periodic and emergency and those which are regarded as rehabilitation. Each country seems to have its own definition, with some using an extra split – recurrent maintenance. The following definitions are generic and have been taken from a number of sources[21] [22] [23] [24] and placed together to form an appropriate definition, although it is appreciated that many other definitions do exist:

Routine maintenance – continuous set of activities to correct deterioration, and maintain the road (or other item) to an acceptable standard as specified by the governing body. The maintenance will be required continuously, regardless of the engineering characteristics or actual traffic volume. Obviously however, the engineering characteristics and volume of traffic will have an effect on the pre-determined frequency of maintenance. Routine maintenance can be regarded as a fixed-cost item. Activities undertaken as part of routine maintenance include: removal of debris in drainage structures, cleaning drainage structures, repair to scour checks, filling and compacting pot holes, repair to shoulders, trimming and clearing of vegetation, reshaping small sections of the carriageway, road furniture maintenance.

Periodic maintenance – activities required after a certain period, possibly years, (the period being dependent on the engineering characteristics of the road and the level of traffic). The cost is variable, as the frequency is dependent on the characteristics of the road. Authors who use a further split of recurrent maintenance, describe it as maintenance which is required at intervals during the year, it is similar to periodic in that the frequency at which it is required will depend on the engineering and traffic characteristics of the road. Activities include reshaping of the carriageway, small reconstruction of drainage structures, regravelling of the road, provision of gravel stockpiles for use in routine maintenance.

Emergency maintenance – refers to sudden and unforeseen damage, usually requiring the immediate deployment of resources. It is difficult to budget for, and many countries do not have any emergency fund on which to draw. Damage can be caused by weather such as monsoons, hurricanes etc. The activities are likely to involve repair to damaged drainage structures, repair to damaged structures and carriageways, clearing of trees, clearing landslides etc.

Rehabilitation works – restoration work that is required to repair major damage, such as when a pavement has gone beyond the point where maintenance will work and it requires reconstruction. Rehabilitation can be required due to extreme events such as major weather events, or due simply to neglect and lack of maintenance.

T Jones (1987) and others carried out an analysis on the rate of return for maintenance and new road construction. They found that the average rate of return for maintenance was 63% with a range of 20-118% whereas the average rate of return for new roads was 22% with a

[20] ILO ASIST, The Labour-Based Technology Source Book, A catalogue of key publications, ASIST information service, 1998.
[21] Fiji Government Ministry of Infrastructure and public utilities Annual Road Maintenance Report, RMSU, 1994, p9.
[22] Marks, G. J., Maintenance Management Manual, Labour-based rural road maintenance, Lao, PDR, 1997, pp4-6.
[23] TRRL (1987) “Maintenance management for district engineers”, Overseas Road Note 1, Department of Transport, UK, p3.
[24] ASIST Bulletin No.5 September 1996, pp12-13.

range of 14-53%[25]. The importance of maintenance is very clear. Several different maintenance techniques are available to the pavement maintenance engineer, these are as follows.

3.3.1 Patching

Patching is a repair done to the wearing course when small-scale defects appear in the road surface. This includes potholes, soft spots, ruts and erosion gullies[26]. It is normally referred to as the work involving less than 1 or 2 truck or trailer loads of material per day (TRL, 1994)[27]. Patching is a labour-based activity, where the material used may also be delivered by labour-based or equipment based methods. Larger scale work is called spot regravelling or gravelling.

3.3.2 Dragging

Dragging is a routine maintenance task used to improve unpaved roads by smoothing the road surface and removing loose materials from it. Self-propelled graders, towed graders or specially made drags (such as brushwood, frame drags, beam drags, tolards and cable drags) towed by agricultural tractors can be used[28].

3.3.3 Grading

Grading is used to restore the original shape of the pavement which includes recovering gravel from the shoulder, filling the pot holes and corrugations and restoring a good camber to ensure good drainage. Light grading is normally carried out using a motor grader but in some countries towed graders or drags are employed[29]. Grading lightly trims the surface of the road to control roughness and corrugations. Heavy grading is used when roads are seriously deteriorated with potholes and ruts. It involves the use of motor graders, water tankers and compaction plant (TRL, 1994)[30].

3.3.4 Regravelling

Regravelling is a major maintenance operation used to replace a worn away gravel surface before the distress of the subgrade starts. It may also be used to correct other defects in the road when they become very severe and also may be used to transform an engineered earth road into a gravel road (OSRN 2, 1987)[31]. The frequency of regravelling depends on the traffic volume, environment conditions, type of wearing course materials and alignment of the road.

3.4 Material Selection and Specification for the Maintenance of Unpaved Roads

Some interesting observations are made in the Unsealed Roads Manual (RRU Technical Recommendations TR8) – A manual of repair and maintenance for Pavements: *'In most cases material quality standards arbitrarily imposed from outside are neither economical nor feasible on low volume roads. Each road has to be tailored to suit the local requirements and resources. Any suggestions or figures given…….are at best a guide. Standards do have their place in roading but not so much on roads carrying under, say, 30 vehicles per day'*[32].

'Materials may be either crushed or uncrushed, and range in hardness from soft shells to the hardest stone. Some partly decomposed or weathered rock is acceptable providing its

[25] Jones, T.E. (1987) "Optimum maintenance strategies for unpaved roads in Kenya", PhD thesis, The University of Birmingham.
[26] Jones, T.E. (1987) "Optimum maintenance strategies for unpaved roads in Kenya", PhD thesis, The University of Birmingham.
[27] International Road Maintenance Handbook, Volume II, Maintenance of unpaved roads, 1994.
[28] International Road Maintenance Handbook, Volume II, Maintenance of unpaved roads, 1994, pp117-127.
[29] Jones, T.E. (1987) "Optimum maintenance strategies for unpaved roads in Kenya", PhD thesis, The University of Birmingham.
[30] International Road Maintenance Handbook, Volume II, Maintenance of unpaved roads, 1994.
[31] Overseas Road Note 2, Maintenance techniques for district engineers, Overseas Unit TRRL, 1985, pp24-25.
[32] Unsealed Roads manual (RRU Technical Recommendations TR8) – A manual of repair and maintenance for Pavements, A.G. Ferry, 1986, p4-1.

durability is known'[33]. i.e.: a great variety of materials may be suitable for the construction and maintenance of roads, but it is dependent on what is available and what is acceptable.

Common aggregates used for unsealed roads may include: river gravel, terrace gravel beds or pits, quarry overburden, crushed quarry rock, slag and other by-products[34].

'Experience has shown that a great variety of gradings of natural gravel perform well in practice, and therefore no fixed grading envelopes can be stipulated as essential for good performance. The performance of the wearing course material is mainly influenced by the properties of the material passing the 2.00mm sieve although it is generally desirable for the natural gravel to be well graded and not to exceed a maximum size of 50mm, but 37.5mm is preferred (after compaction)'[35].

O'Reilly and Millard (1969)[36] recommended the following properties for various climate conditions:

Table 3-1 - Properties Relating to Climate

Climate	Liquid Limit %	Plasticity index %	Linear Shrinkage %
Moist temperature and wet tropical	35	4-9	2.5-5
Seasonal wet tropical	45	6 - 20	4 - 10
Arid and semi arid	55	15 - 30	8 - 15

It is reported (Jones 1987)[37]: that the South Africans adopt a design method based on experience rather than the structural mechanism of the road. They recommend a 150 mm thickness of wearing course compacted at 95% of the maximum dry density determined from modified AASHTO test and having a minimum CBR value of 20. The grain size distribution is shown in Table 3-2. Granular materials which are suitable for use as wearing course material in gravel roads are Lateritic gravel, Quatizitic gravel, calcareous gravel, crushed rocks, certain soft rocks and partly decomposed rocks. For the material, additionally, the liquid limit should be in the range of 20-35 preferably 30 with a Plasticity Index between 5 and 15. The recommended abrasion is 30-60% after 500 revolutions in the Los Angeles abrasion machine and the design crossfall for a pavement containing such material is 2.5%.

3.4.1 Australian Unsealed Roads Manual

According to the Australian Unsealed Roads Manual, 1993, the characteristics of a good wearing course material for an unpaved road are as follows: low permeability, stability in the climatic conditions predominant in the area, resistance to ravelling, scouring and skid resistance, (influenced by particle size distribution). The most suitable materials for pavements on unsealed roads are well-graded-sand mixtures with a small proportion of clayey fines. It states that the material should have:

- 100% passing a 26.5 mm sieve;
- Between 20% and 60% retained on a 2.36 mm sieve;
- Fines to sand ratio in the 0.20 to 0.40 range;

[33] Unsealed Roads manual (RRU Technical Recommendations TR8) – A manual of repair and maintenance for Pavements, A.G. Ferry, 1986, p1-5.
[34] Unsealed Roads manual (RRU Technical Recommendations TR8) – A manual of repair and maintenance for Pavements, A.G. Ferry, 1986, p4-16.
[35] TRH 14, Guidelines for road construction materials, Technical Recommendations for highways, 1985, p32.
[36] O'Reilly, M.P. and Millard, R.S, (1969) "Road making materials and pavement design in tropical and subtropical countries", TRRL, Laboratory Report LR 279, Crowthorne, UK.
[37] Jones, T.E. (1987) "Optimum maintenance strategies for unpaved roads in Kenya", PhD thesis, The University of Birmingham.

- Plasticity Index range of 4 to 15 with the lower end of the range being for wetter climates;
- A linear shrinkage range being within 2 to 8, again with the lower end of the range being for wetter climates[38].

Table 3-2 - Grain Distribution Size (After Jones 1987)

Sieve size	Percentage Passing by Mass for Wearing Courses		
	37.3 mm max size	**19.0 mm max size**	**13.2 mm max size**
37.5	100		
19.0	70-100	100	
13.2	60-85	70-100	100
4.75	40-60	50-75	60-100
2.00	30-50	35-60	45-75
0.425	15-40	18-45	25-50
0.750	7-30	7-30	7-30

There are many, many sources of advice relating to the characteristics of material for gravel roads, the fact that there is so much information, can itself cause confusion. In general the characteristics used to define the materials are CBR (soaked), Plasticity Index and sometimes gradings. Additionally, Plasticity Modulus (PI x % passing the 0.425 mm sieve), Grading Modulus (see section 3.4.4), Linear Shrinkage and Liquid Limit are also used. The last two are related to the PI, and will not be mentioned further. Below is a selection of the recommendations available from a variety of sources.

3.4.2 California Bearing Ratio a Variety of Advice relating to CBR

A minimum soaked CBR of 20 is required for the imported aggregate material, however the CBR may be dropped to 15 if the traffic is going to be less than 15 commercial vehicles/ day[39].

If traffic is over 30 vehicles/ day a CBR of 30 is used, if the volume of traffic is below 30 vehicles per day, a CBR of 20 is considered to be adequate[40].

'the minimum CBR of material (gravel subbase) used without chemical stabilisation shall be 45% at the specified insitu density……….*The following requirements shall apply to natural materials (subgrade) when used without any chemical stabilisation: Minimum CBR at specified insitu density:'*

- Upper selected subgrade = 15
- Lower selected subgrade = 7

'*Pavement materials shall be free from vegetable matter, lumps or balls of clay or other deleterious matter. Natural gravels, sands and soft or ripped rock shall comply with CBR% (min) Base = 80 and Subbase = 30 (4 days soaked CBR @ 98% modified compaction).'*

[38] Unsealed roads manual – Guidelines to good practice, ARRB, 1993, p3.12.
[39] Road Design Manual, Part III, Materials and Pavement Design for New Roads, Chapter 13, Gravel Roads, 1987, p13.1.
[40] A practical guide to pavement design for tropical countries, French Republic, The Ministry of Co-operation, 1984, p111.

3.4.3 A Variety of Advice Relating to Plasticity Index

Imported aggregate are permitted a PI in the range of 4 to 15 with the lower end of the range being for wetter climates[42].

For a moist tropical and wet tropical climates, a PI in the range of 4 – 9 % should be used, for a climate of seasonal wet tropical a PI in the range 6 –20% should be used and for an arid and semi-arid climate a PI in the range 15 – 30% should be used[43].

3.4.4 Grading Modulus

The grading modulus is defined as:

$$GM = \frac{P2{,}00mm + P0{,}425mm + P0{,}075mm}{100} \quad \textbf{Eqn. 1}$$

Where P2,00mm etc denotes the % retained on the sieve indicated[44].

The minimum grading modulus for selected subgrade is 0.75[45]. With regard to Gravel Subbase - 'The minimum grading modulus shall be 1.5 except where a material, having a lower GM but not less that 1.2 is approved for use either on the borrow pit plans or by the Engineer'[46].

3.4.5 Advice Relating to Plasticity Modulus

The Plasticity Modulus (PM) is the PI x the percentage of material by mass passing the 0.425mm sieve. The following advice is given, advice from TRH 20, suggests a value of Shrinkage Product in the range 100 – 365, with an ideal at 240. The shrinkage produce (Sp) is the Linear Shrinkage result x the percentage passing the .425mm sieve.

3.4.6 Generic Specifications

It can be seen that a variety of advice and information exists relating to material specifications. As already pointed out, it is difficult to specify (in a generic way) which materials will be suitable for the construction and maintenance of gravel roads, as it depends on the country and what is available – it is pointless using specifications which cannot be met. Most materials are quite moisture sensitive and, thus, may change their behaviour radically upon wetting. This opens the possibility of using sub-specification materials, but, for example, providing more attention to drainage provision or more frequent maintenance than anticipated by the specification.

For reasons such as this, the PI, PM, and CBR values quoted above should only be seen as indicative of general usage.

3.4.6.1 Material Types –Example Of Subjective Description Of Materials

Table 3-3 offers an alternative approach based on a subjective description rather than laboratory measured characteristics. Such subjective descriptions are useful as they provide an overview of a variety of materials, however they should not be considered as a replacement for specifications.[47].

[42] Unsealed Roads Manual – Guidelines to good practice, ARRB, 1993, p3.12
[43] Overseas Road Note 2, Maintenance techniques for district engineers, Overseas Unit TRRL, 1985, p24.
[44] TRH14:1985 Guidelines for road construction materials, Committee of State Road Authorities.
[45] Standard specifications for roads and bridges, Transvaal provincial administration, Roads Department, July 1980, p88.
[46] Standard specifications for roads and bridges, Transvaal provincial administration, Roads Department, July 1980, p104.
[47] Public Works Department, FRUP II RMSU April 1994, Page 2.4.

Table 3-3 - Overview of Material Types and the Value in Road Maintenance and Construction

Road Making Materials - Typical Names	Value as Subgrade	Value as Subbase	Value as Base Under Seal	Shrinkage or Swelling Properties	Drainage Properties	Workability as a Construction Material	Compaction Equipment
Well graded gravel or gravel sand mixtures, little or no fines	Excellent	excellent	Good	almost none	excellent	excellent	rubber tyred/ steel wheel/ vibrating roller
Poorly graded gravels or gravel sand mixtures, little or no fines	good to excellent	good	fair to good	almost none	excellent	good	rubber tyred/ steel wheel/ vibrating roller
Silty gravels	good to excellent	good	fair to good	almost none	fair to good	good	rubber tyred/ steel wheel/ vibrating roller
Gravel sand silt mixtures	Good	fair to good	good to not suitable	almost none	poor to practically impervious	good	rubber tyred/ steel wheel/ vibrating roller
Clayey gravels and gravel-sand clay mixtures	Good	fair to good	good to not suitable	almost none	poor to practically impervious	fair to good	rubber tyred/ steel wheel/ vibrating roller
Well-graded sands or gravely sands, little or no fines.	Good	fair to good	Poor	almost none	excellent	excellent	rubber tyred/ steel wheel roller
Poorly graded sands or gravely sands, little or no fines.	fair to good	fair	poor to not suitable	almost none	excellent	fair	rubber tyred/ steel wheel roller
Silty sands	fair to good	fair to good	Poor	slight	fair to good	fair	rubber tyred/ steel wheel/ vibrating roller
Sand Silt mixtures	fair	poor to fair	not suitable	almost none	poor to practically impervious	fair	rubber tyred/ steel wheel/ vibrating roller
Clayey sands, sand clay mixtures	poor to good	poor to good	good to not suitable	very slight	poor to practically impervious	fair to good	rubber tyred/ steel wheel/ vibrating roller

[50] Jones, T.E. (1987) “Optimum maintenance strategies for unpaved roads in Kenya”, PhD thesis, The University of Birmingham.

4. REVIEW OF STRUCTURAL ASSESSMENT METHODS

4.1 The Structural Approach to Design

4.1.1 Empirical Approaches

Relatively little work has been done on the structural analysis and design of unpaved roads as opposed to paved roads[50]. Mellier (1968) produced an equation for calculating the required thickness of base course, in which he empirically linked base course thickness to the maximum wheel load and the CBR of the subgrade[51]. For a typical case with a wheel load of 4 tonnes and a CBR of 15%, the equation gives a thickness of 200 mm which is higher than 150 mm adopted by many developing countries[52]. He suggested that this equation is valid for base course with a CBR of 15 or lower.

Ahlvin and Hammit (1975) produced the following empirically derived equation for calculating the required thickness of base course in terms of the axle load, tyre pressure, number of load applications and the CBR values of both the subgrade and base course[53].

$$Ta = 1.02165Log(L) + 0.6324Log(P) + 0.2148Log(L) + 0.2394Log(Tb) - 0.4028Log(c_1) - 0.3140Log(c_2) \quad \text{Eqn. 2}$$

Where		
	Ta	Thickness of base layer in inches
	L	Load in Lb
	P	Tyre pressure in psi
	Tb	Traffic passes
	c_1	CBR for subgrade
	c_2	CBR for base course

The equation is based on a rut depth failure criterion of 50-75.

O'Reilly and Millard (1969)[54] claimed that thickness of base course is not a critical factor in the design of a gravel road. Rather, it is the engineering properties of the layer (which is normally placed at a standard thickness of 150-200mm). This is based on the assumption that the road is constructed properly on a sufficiently strong subgrade to carry the traffic load provided that it is covered by 100-150mm of gravel (Jones 1987)[55].

In Kenya[56], unsealed roads are designed according to traffic loading:

- Class 1 road >150 commercial vehicle/day
- Class 2 road<150 commercial vehicle/day

The granular course must then have a Plasticity modulus: of 200-1200, a PI in wet areas of 5-20 and in dry areas of 10-30 and a CBR of more than 20 following compaction at 95% of MDD mod and soaking for 4 days. Kenyan practice does vary, granular layer thicknesses are shown in Table 4-1 (after Road Design Manual, Kenya (1987)[57]).

[51] Mellier, G. (1968), "La route en terre: structure et entretien maintenance", Eyrolles, pp. 36-57, Paris.
[52] Jones, T.E. (1987) "Optimum maintenance strategies for unpaved roads in Kenya", PhD thesis, The University of Birmingham.
[53] Ahlvin, R. G. and Hammit, G. M., (1975) "Load supporting capabilities\of low-volume roads", TRB Special Report 160, Washinghton DC.
[54] O'Reilly, M.P. and Millard, R.S, (1969) "Road making materials and pavement design in tropical and subtropical countries", TRRL, Laboratpry Report LR 279, Crowthorne, UK.
[55] Jones, T.E. (1987) "Optimum maintenance strategies for unpaved roads in Kenya", PhD thesis, The University of Birmingham.
[56] Ministry of Transport and Communication (MOTC) (1981) "Materials and pavement design manual for new roads", MOTC, Kenya, Chapter 13.
[57] Ministry of Transport and Communication (MOTC) (1981) "Materials and pavement design manual for new roads", MOTC, Kenya, Chapter 13.

Table 4-1 - Minimum Gravel Subgrade Thickness

Subgrade Strength Class	CBR Values (%)	Initial Daily Number of Commercial Vehicles (Both Directions)			
		<15	15-50	50-150	150-500
S1	2-5	350	425	500	585
S2	5-10	225	275	325	375
S3	7-13	175	225	250	275
S4	10-18	150	175	200	225
S5	15-30	125	150	175	200

The above CBR ranges correspond to the results actually obtained on materials of the same type along sections of a road considered homogeneous. They reflect both the variations of the characteristics of the soil which inevitably occur, even at small intervals, and the normal scatter of test results'[58]. As there is some overlap in some of the classes, it may be more appropriate to change to CBR range as follows:

Table 4-2 - Amended Subgrade Bearing Strength Classes

Soil Class	CBR Range (%)
S1	2-5
S2	5-7
S3	7-10
S4	10-15
S5	15-30
S6	>30

4.1.2 Fundamental Contribution of Materials

Performance-based specifications may allow the use of unconventional and marginal materials which will behave acceptably (or can be treated so as to behave adequately). This would require conventional specifications be relaxed to allow for the use of such materials. Modified specifications for "marginal materials" are not yet common in developed countries for granular materials, partly because of the difficulty of assessing the fundamental load spreading and rut-resisting characteristics of granular materials. TRL and Roughton International [59] have undertaken a detailed review of current use of marginal material with a view to greater promotion or use for low cost materials where appropriate.

Most of the current specifications and design manuals refer to some of the required qualities of aggregate used in road construction such as the aggregate should be sound, hard, strong, durable and free from deleterious material. There are some physical test procedures to relate directly or indirectly to the above properties but the link between these properties and road performance is not robust nor comprehensive (Metcalf, 1991). Amongst the most important properties required in the design and maintenance of roads are the shear strength and the stiffness (resilient modulus) of soil. While the stiffness (resilient modulus) of a road layer reflects its load distribution ability, the strength may indicate the resistance of a soil to permanent deformation and, hence, rutting. The main objective of the structural layers of a

[58] Ministry of Transport and Communication (MOTC) (1981) "Materials and pavement design manual for new roads", MOTC, Kenya, Chapter 13, p6.1.

[59] Transport Research Laboratory (TRL) and Roughton International (RI) (2000) " Promoting the use of marginal materials)

road is to distribute the traffic load to a stress level at the subgrade level that can be withstood by the subgrade without excessive permanent deformation. Furthermore, permanency and deformation within the aggregate will manifest itself as rutting which can be overcome by regrading – an option not available for the subgrade. Therefore the stiffness (resilient modulus) is more important to pavement layers while the shear strength is more important for the subgrade. Although for a specific material the stiffness (resilient modulus) generally increases with the increase in strength, the correlation between the two parameters is not precise. In addition, both strength and stiffness values are very dependant on the in-situ moisture condition.

The stiffness (resilient modulus) of a pavement layer reflects its ability to distribute load. Analytical methods for pavement analysis use material stiffness (resilient modulus) to compute the stresses at various depths in the pavement. An important aim is to make sure that the stresses at the subgrade level do not exceed the shear strength of the subgrade soils (or some threshold stress limit at which subgrade rutting builds up too rapidly). In rural unbound roads, the aggregate layer is of limited thickness and therefore the subgrade soil becomes part of the structure of the road and consequently the stiffness (resilient modulus) of the subgrade soil plays an important role in the performance of the road.

4.2 Equipment for Structural Assessment

4.2.1 Principles and Purpose of Structural Assessment

Laboratory testing does not represent field conditions and therefore test results do not exactly reflect the situation in the field. For this reason, various field tests have been adopted to measure various properties of aggregate forming road layers and formation soil. In order for the field test to give realistic and more useful data the test should simulate, as closely as possible, the time of loading (frequency), the load magnitude and the area of the critical loadings. If all these conditions apply then the resulting rate and magnitude of strain should be similar to that produced under the real, damaging traffic loads – and, hence, any effects of material non-linearity will be minimised. Any deviations from these conditions will result in some errors that need to be properly allowed for in the interpretation when using a device with such deviations.

Current practice is to measure field density (described in terms of dry density) and moisture content as a means of controlling compaction on site since key properties like strength and stiffness (resilient modulus) can be related to dry density of road layers. This relationship, between moisture content/degree of saturation and degree of compaction varies with the type of material, so the relationship for the type of material used must first be established before it can be used. During construction of long roads the variability in natural soil type and supplied aggregate is inevitable and therefore one relationship may not be valid. Moreover, the measured dry density of subgrade soil during construction may be changed when road layers are placed on top of it and compacted and as environmental factors take hold. For these reasons a direct method for measuring strength and/or stiffness (resilient modulus) becomes more desirable.

4.2.2 Criteria for Structural Assessment

The problem being dealt with in this element of the project is the field assessment of rural roads in developing countries, where new technology tends to be expensive and not readily available. In addition, the labour force is likely to be of simple education and unable to deal with complex (or, even, some relatively simple) equipment in terms of using and maintaining it. Based on these facts the following criteria were set for a field test:

- The equipment should be simple to use and maintain and should provide results which do not require extensive analysis, particularly using computers
- It should be relatively cheap and able to be manufactured or, at least, maintained locally
- It should be easy to handle, light and portable
- It should not be sensitive to human errors
- The test should be fast to perform

- The test should be accurate
- The test should not require advanced support (e.g. accurate scales, electricity).

4.2.3 Available Assessment Tests

In this section, available field test equipment is reviewed and comments on fit to the criteria set above are given. Based on this assessment, either one of the available devices and/or a modified version is selected to carry out the required work.

Fleming and Rogers (1990) have divided field assessment equipment into five categories namely:

Static Loading, Vehicular loading, Vibratory loading, Impact tests and others. These categories and examples of equipment belonging to each are shown below, with two additional categories introduced[60]:

Static Loading

- Field CBR
- Plate Loading

Vehicular Loading

- Benkelman Beam
- La Croix Deflectograph

Vibratory Loading

- Dynaflect
- Natural Vibration Method
- Compactionmeters

Impulse Loading

- Falling Weight Deflectometers

Impact Tests

- Clegg Hammer

Penetrometers

- Dynamic
- Static

Others

- Surface Wave Measurement
- Pressuremeter
- Ground Radar

The mechanism of each of these tests will be presented, in brief, with advantages and disadvantages. in the light of the criteria set in the following section. On the basis of this, the Dynamic Cone Penetrometer (DCP) has been selected as the preferred tool. Thus a closer examination is made and the possibility of modification is discussed with a view to improving the DCPs suitability for the required task.

[60] Fleming, P. R. and Rogers, C. D. F. (1990) “Literature review of methods of field measurement of elastic stiffness and resistance to permanent deformation”, Loughborough University of Technology, Loughborough, UK.

4.2.3.1 Field CBR Test

The Californian Bearing Ratio (CBR) involves the penetration of a 38mm diameter plunger into the ground surface at a rate of 1mm per minute, recording the load against penetration. Surcharge weights can be placed around the plunger to simulate overburden pressure. The loads measured at 2.5 and 5mm are compared to the corresponding ones measured on a standard aggregate and the higher ratio is given as a percentage. The test is not suitable for coarse aggregates as it gives false readings if the plunger is obstructed by large stones. There are many variants, one of which needs selecting when performing the test (such as soaking, surcharges, overburden pressure).

The test is relatively inexpensive and moderately time consuming. The area of loading is very small and the rate of loading is much slower than real traffic conditions. The test does not reflect exactly stiffness (resilient modulus) or strength but gives something in between (DoT, 1994). In spite of all these disadvantages, the CBR value is the most common parameter used in conventional methods of pavement design and maintenance. With such a large body of information, most of the later developed testing apparatus have been calibrated to CBR value. Some published correlations between CBR and strength and stiffness (resilient modulus) are available, but these are of a very approximate and/ or material and condition specific nature[61].

4.2.3.2 Plate Loading Test

This test is used to determine the resilient modulus of soil or as an indirect measure of CBR, in-situ. It involves the application of load increments to a circular steel plate and the measurement of the resulting deflection at the end of each loading increment. Then the load is decreased in increments as well and the change in deformation is measured. The test is described fully in BS 1377 (1990)[62] and its use for testing is described in the DoT specification (MCHW1, Series 600, (1993))[63].

Resilient Modulus is measured during the unloading stage of the test. Plate diameters of 300, 450, 600 and 760 may be used. While plates of small diameter reflect the stiffness (resilient modulus) of the surface layer, the larger plates give a cumulative response of the surface and deeper layers of the road. The loading-unloading cycles may be repeated a few times until a relatively constant gradient of the unloading line is obtained. The elastic modulus is calculated using the following theoretical equation[64] which applies to a uniform half-space with a circular load

$$E = \frac{\pi p r (1 - \nu^2)}{2y} \qquad \text{Eqn. 3}$$

Where:	E:	Elastic Modulus
	p:	Stress applied
	r	Radius of plate
	y	Plate deflection
	Λ	Poissons Ration

The modulus of subgrade reaction K can be calculated from the plate bearing test data as the pressure reading at a mean plate deflection of 1.25 mm for a 760 standard diameter plate (Fleming & Rogers 1990)[66] CBR can then estimated using the following empirical equation:

[61] Brown, S. F., Loach, S. C., O'Reilly, M. P., Repeated loading of fine grained soils, Dept of Environment, Dept of Tranpsport., TRRL Contractor Report 72, 1987.
[62] BS 1377 Part 9, 1990, pp26-29.
[63] Manual of contract documents for highway works, Volume 1, Series 600, 1993.
[64] Design Manual for Roads and Bridges, Pavement Design and Maintenance, Volume 7, (DMRB 7) HD 25/94, Foundations, (1994), HMSO, London.
[66] Fleming, P. R. and Rogers, C. D. F. (1990) "Literature review of methods of field measurement of elastic stiffness and resistance to permanent deformation", Loughborough University of Technology, Loughborough, UK.

$$CBR = 6.1 \times 10^{-8} \times \left(K_{762)}\right)^{1.733} \quad \text{Eqn. 4}$$

Where: CBR Californian Bearing Ratio (%)
K_{762} Modulus of subgrade reaction from 762 mm plate

This test can be used to monitor the effectiveness of compaction by comparing the elastic modulus of the first and last loading cycles and if the ratio of last/first modulus is more than 2.0 then compaction is deemed inadequate[67].

The test is relatively slow, laborious to set up and perform and its loading time is very slow compared to traffic loading (Fleming & Rogers, 1990[68]; Georgiadis, 1988[69]). The various bits of apparatus are quite heavy and require motorised transport to move. A heavy reaction is required, typically the chassis of a lorry. The equipment uses hand operated hydraulic pumps and sensitive dial gauges which may not be amenable to long-term use in a rural road situation.

4.2.3.3 Benkelman Beam

The principle of the Benkelman beam is the measurement of deflection and rebound of a road surface when subjected to full-scale vehicular loading. The beam consists of an elongated tripod and a long arm. The arm rests on a pivot on the top of the tripod. A dial gauge bears on the rear end of the arm while the other end of the arm rests on a probe, which is in contact with the ground surface where deflection needs to be measured. A truck is then driven forward with the beam positioned between the twin tyres in one wheeltrack. The point under the probe rebounds upwards which makes the arm rotate and the dial gauge records the corresponding down movement.

The Benkelman beam is relatively expensive, time consuming and has been found specifically unsuitable for unbound layers. Upward heave might happen due to surface instability which causes the device to give false reading for deflection. It requires a vehicle to apply the loading and a team of staff to take readings from the sensitive dial gauges and hence the equipment will require maintenance, which may make it unsuitable for long-term use in the rural roads setting. [70] [71]

4.2.3.4 La Croix Deflectograph

This device is an automated version of the Benkelman beam originally developed in France. It comprises a truck with two beams measuring deflection in both wheeltracks. Deflections are measured with the aid of transducers connected to a data acquisition unit housed in the loading truck.

This device has the same disadvantages as the Benkelman beam but it is quicker to use. However, in addition to the complication of electronic equipment, it is considerably more expensive than the Benkelman Beam and is likely to require very specialised maintenance. It also requires trained operatives to process the computerised data. It is therefore unsatisfactory for long-term use on rural roads, and also has limited suitability for use on unbound materials.

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[67] Design Manual for Roads and Bridges, Pavement Design and Maintenance, Volume 7, (DMRB 7) HD 25/94, Foundations, (1994), HMSO, London.
[68] Fleming, P. R. and Rogers, C. D. F. (1990) "Literature review of methods of field measurement of elastic stiffness and resistance to permanent deformation", Loughborough University of Technology, Loughborough, UK.
[69] Georgiadis, M., (1988) "Strain rate influence on the interpretation of plate bearing tests", ASTM Geotechnical Testing Journal, Vol. 11, No. 4, Dec., pp. 293-295.
[70] Fleming, P. R. and Rogers, C. D. F. (1990) "Literature review of methods of field measurement of elastic stiffness and resistance to permanent deformation", Loughborough University of Technology, Loughborough, UK.
[71] Cobbe, M.I., (1986) "Development of acceptable criteria for subgrade improvement layers", Special report for Dtp, Engineering Intelligence Division.

4.2.3.5 Dynaflect

Dynaflect is a vibratory loading device which applies cyclic loading on the pavement surface and the resulting cyclic deflections at radial distances from the centre are measured. Back-calculation methods are used to work out various layer moduli of the pavement. The load is transmitted to the pavements through two steel wheels, which are 400mm in diameter, 50mm wide and 450mm apart[72]. Although the stress type and magnitude simulate traffic loading quite well, the applied loading area is small and the resulting deflection at the centre is not measured. Neither is the rolling-wheel effect replicated. In addition, the equipment is quite expensive with electronic units which will be difficult to support and maintain. Furthermore the equipment is towed by a four wheel drive van, therefore making it even more expensive.

4.2.3.6 Natural Vibrations Method

This method is based on applying natural frequency vibration to the road surface and uses the elastic theory applied to the vibrations of a mass on a spring to work out the stiffness (resilient modulus) of the pavement[73]. By applying various loads, various natural frequencies are obtained and therefore a relationship between E values and stress level can be obtained which enable a non-linear stress dependent analysis of the pavement to be carried out. A good correlation was found between Natural Vibrations Method (NVM) and plate loading test[75]. More research needs to be carried out to confirm the validity of the results which can then be used for advanced analysis of pavements. The test is obviously expensive, partly because it is relatively new and partly due to the equipment. The NVM testing equipment is heavy, therefore making it more difficult to transport, and resulting in the need for extra staff. It is also has electrical components and therefore inappropriate for use on rural roads.

4.2.3.7 Compactionmeter

This method is based on measuring the vertical dynamic force applied by the drum of a vibrating roller on the road surface. The value of this force depends on degree of compaction and dry density which can be correlated to the bearing capacity of the road[76]. The speed and frequency of the roller have to be kept constant in order to get repeatable results. It is mainly used to control compaction and, due to the electronics and the special plant required, it is probably too expensive and innappropriate for use in the rural roads context.

4.2.3.8 Falling Weight Deflectometers

There are various types of deflectometers but generally they are based on the same principle where a load is raised mechanically to a defined height and dropped on a set of rubber cushions. The load is transmitted to the road surface. The load is measured by a load cell while the deflection bowl resulting from the falling weight is measured by a number of velocity transducers which touch the pavement surface. These are placed along a line starting from the centre of loading and ending at a radial distance where the deflection becomes insignificant. The load applied ranges from 10 to 250 kN with plate diameters of 300 or 450mm. The resilient moduli of various layers of the pavement are determined using back calculation methods. The FWD is very popular in road assessment since it simulates,

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[72] Scrivner, F. H., Swift, G. and Moore, W. M. (1966) “A new research test for measuring pavement deflection”, Highway Research Board, HRR 129.

[73] Ehrler, O.M.Ch., (1989), “Measurement of non-linear deformation characteristics of unbound aggregates by natural vibration method”, Proc. 3rd Int. Symposium on Unbound Aggregates in Roads. Nottingham University. Nottingham, UK.

[75] Fleming, P. R. and Rogers, C. D. F. (1990) “Literature review of methods of field measurement of elastic stiffness and resistance to permanent deformation”, Loughborough University of Technology, Loughborough, UK.

[76] Fleming, P. R. and Rogers, C. D. F. (1990) “Literature review of methods of field measurement of elastic stiffness and resistance to permanent deformation”, Loughborough University of Technology, Loughborough, UK.

closely, the amount and area of loading and the speed of load application (although not the rolling-wheel effect). However, in the present case it is not suitable in a rural road situation because it is expensive and complicated and not very reliable on unsealed roads due to surface looseness[77].

4.2.3.9 Clegg Hammer

The Clegg hammer could be said to be a dynamic CBR test. The standard Clegg hammer uses a hammer of 50mm diameter and 4.5 kg weight is dropped from a height of 450mm and the deceleration of the hammer is measured by an accelerometer and displayed on a digital display unit. The standard use of the equipment is reported to be for soil stiffness (resilient modulus) and uniformity testing, especially in relation to compaction control. The unit of measurement is multiples of 10 times the acceleration due to gravity and is called the Clegg Impact Value (CIV). This has shown a good correlation with CBR, as reported in 1986 by Dr Baden Clegg Pty Ltd[78]. There are alternative forms of the Clegg hammer, such as the 0.5kg 'Light' Clegg Impact Soil Tester for use on soft materials, the 2.25kg 'Medium' and the 20kg 'Heavy' Clegg Impact Soil Tester (for use on harder materials). The Clegg hammer is relatively simple to use, it is portable and is quick to operate. However, it has electronic components and is therefore likely to require regular maintenance and calibration, which could make it less suitable for use in low cost road maintenance. Although cheap compared with many of the preceding alternatives, it is still relatively expensive.

4.2.3.10 Static Cone Penetrometers

The static cone penetrometer was first developed by the British army as a light-weight rapid instrument to assess the strength of soil. In this test a cone is pushed by a hydraulic jack into the ground at a constant speed. Normally related to CBR in the range of 0-15, it is suitable for fine-grained subgrade soil of low to medium strength. In addition to CBR, some of the newer types measure the cone index which is an indicator of soil trafficability (TRRL, 901 1979)[79]. Although the cone penetrometer is lightweight, easy to use and has no electronics. It can only be used on soils with low CBR. More importantly in the context of low cost roads, it is more expensive than a DCP. Current estimates show that it can cost 3 times as much as a DCP.

4.2.3.11 Surface Wave Measurement

The velocity of Rayleigh waves are used to measure shear modulus and hence elastic modulus. The wave is used to detect rigid layers at depth in engineered and natural ground. Research in many countries is being employed to apply the technique to the more complex structure of a pavement. *'Measurement and interpretation are relatively slow processes, and the view of users is that development is needed too make it an efficient tool for "production" testing'*.[80] The method requires geophones, accelerometers and a significant dead weight, it is rather complex and therefore not suited to the rural roads application.

4.2.3.12 Pressure Meter

The pressure meter (developed by Menard in France 1956) is mostly used for foundation design. The procedure involves drilling a hole and inserting a probe into the hole. The probe is then inflated and it records the increase in pressure and volume. The soil modulus may be obtained from the pressuremeter curve. The test is *'…relatively inexpensive and it can give the proper layer moduli to calculate small pavement deflections which correlate generally well with FWD deflections. However, the test is intrusive, relatively slow and suitable only for*

[77] Fleming, P. R. and Rogers, C. D. F. (1993) "Literature review of methods of field measurement Assessment of resistance to permanent deformation of granular materials", Loughborough University of Technology, Loughborough, UK, p4.
[78] Clegg Impact Soil Tester - Newsletter No.2 July 1986.
[79] Black, W. P. M., The strength of clay subgrades: its measurement by a penetrometer, 1979.
[80] Fleming, P. R. and Rogers, C. D. F. (1990) "Literature review of methods of field measurement of elastic stiffness and resistance to permanent deformation", Loughborough University of Technology, Loughborough, UK, p16.

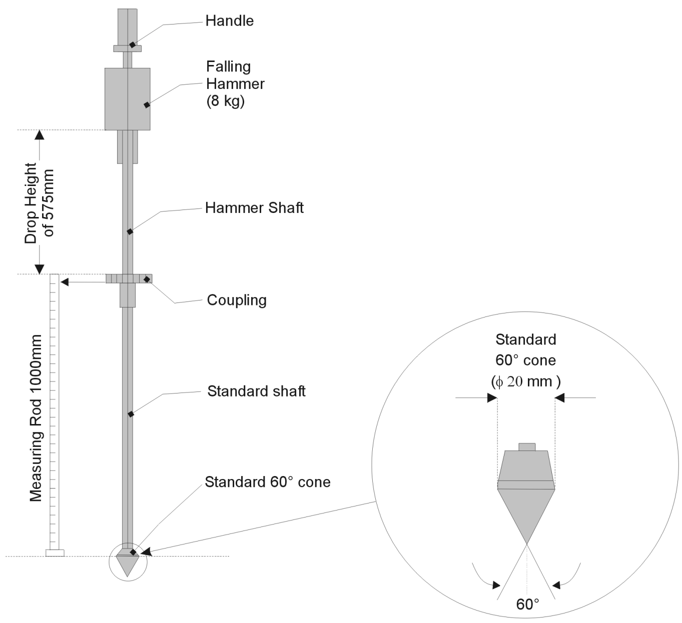
full pavement constructions'[81]. Due to the complexity of analysis, the fact that it is currently only of use for full pavement constructions, coupled with the necessity for maintenance and calibration, it is unlikely to be of use in a rural road's scenario.

4.2.3.13 Dynamic Cone Penetrometer

The Dynamic Cone Penetrometer (DCP) is an instrument designed for the rapid in situ measurement of the structural properties of existing road pavements with unbound granular materials. Continuous measurements can be made easily to a depth of up to 900 mm or too much greater depths with extension road and couplings. The DCP is easy to use, easily repairable, economical to purchase and an almost non-destructive test. It has been adopted in official specifications as a strength measurement test (TRRL, 1991)[85].

The underlying principle of the DCP is that the rate of penetration of the cone, when driven by a standard force is inversely related to the strength of the material as measured by, for example, the California Bearing Ratio (CBR) test. Where the pavement layers have different strengths, the boundaries between the layers can be identified and the thickness of the layers determined (Chua et. al., 1987)[86].

Figure 4-1 - The Dynamic Cone Penetrometer Apparatus



A limitation of DCP testing is that it might give a false reading if a large stone comes under the tip of the cone. This problem can be overcome to some extent by the relative easiness of conducting a number of tests and analysing them statistically.

[81] Fleming, P. R. and Rogers, C. D. F. (1990) "Literature review of methods of field measurement of elastic stiffness and resistance to permanent deformation", Loughborough University of Technology, Loughborough, UK, p17.

[85] TRRL 1990, A users manual for a program to analyse dynamic cone penetrometer data, Transport and Road research laboratory, 1990, Overseas Road Note 8, Crowthorne, Bershire.

[86] Chua, K.M., and Lytton, R.L., Dynamic analysis using the portable pavement dynamic cone penetrometer, Transportation Research Record, Number 1192, 1988, pp27-38.

4.2.4 Interpretation of DCP Test Results

Since CBR is the most conventional strength measure used by road engineers in many countries including the UK, many researchers have correlated the DCP number in mm/blow to the CBR value for various materials and conditions.

Kleyn (1975) performed an extensive series of two thousand laboratory tests using the standard CBR mould and measured the CBR and DCP number. Plotting the results on a log-log scale he found that there is a clear linear relationship between the two parameters[87], as follows:

$$\log(CBR) = 2.632 - 1.280\log(mm/blow) \qquad \text{Eqn. 5}$$

Kleyn (1975) studied some of the factors that may affect the DCP readings when performed in a standard CBR mould. He also found that the length of the mould did not have any influence on the DCP readings whereas the diameter of the mould has some effect due to confinement. DCP showed more resistance towards the end of the mould (by 10 to 25% depending on type of material)[88]. He also found that DCP and CBR showed a similar trend of variation with moisture content.

$$\log(CBR) = 2.555 - 1.145\log(mm/blow) \qquad \text{Eqn. 6}$$

Smith and Pratt (1983) also performed CBR and DCP field tests on natural subgrade of mottled clay and imported yellow clay with sandstone[90]. The following equation was derived from regression analysis with a correlation coefficient of 0.85:

Van Vuuren (1969) derived a similar relationship to the above equations[92].

$$\log(CBR) = 2.503 - 1.115\log(mm/blow) \qquad \text{Eqn. 7}$$

Livneh and Ishaia (1987) derived a slightly different equation for the 30^o cone[93]:

$$\log(CBR) = 2.200 - 0.71 * [\log(DCP)]^{1.5} \qquad \text{Eqn. 8}$$

Livneh (1989)[96],[97] showed that the overburden pressure has no effect on the DCP readings within the range of the depth of road layers. This applies only to shallow depths as, in road applications, where the interlocking between particles is the dominant factor. At deeper levels a “shaft” friction effect between the rods and previously penetrated layers should be

[87] Kleyn, E.G., (1975) “The use of dynamic cone penetrometer (DCP)”, Transvaal Roads Development, Report L2/74, Pretoria.

[88] Kleyn, E.G., “The use of dynamic cone penetrometer (DCP)”, Transvaal Roads Development, Report L2/74, Pretoria, 1975.

[90] Smith, R.B. and Pratt, D.N. (1983) “A field study of in situ California Bearing Ratio and dynamic cone penetrometer testing for road subgrade investigations”, Australian Road Research, Vol. 13, No. 4, pp. 285-294.

[92] Van Vuuren, D.J. “Rapid determination of CBR with portable dynamic cone penetrometer”, The Rhodesian Engineer, September, 1969.

[93] Livneh, M. And Ishai, I. “Pavement material evaluation by a dynamic cone penetrometer”, Proceeding of the Sixth International Conference on Structural Design of Asphalt Pavements, University of Mechigan, (1987) Ann Arbor, Vol. I, pp. 665-676.

[96] Livneh, M., Validation of correlations between a number of penetration tests and in situ california bearing ratio tests, TRR 1219, 1989, pp56-67.

[97] Livneh, M., The correlation between Dynamic Cone Penetrometer values (DCP) and CBR values, Transport Research Institute Israel, 1989.

allowed for. McElvaney et al (1985)[98] found the following correlation that agrees with the majority of researchers in the form of the equation:

$$\log(CBR) = -1.32\log(DCP) + 2.81 \qquad \text{Eqn. 9}$$

Some research has been conducted on variations of the DCP for example Scala (1956) proposed a log/log relationship between CBR and DCP[99] based on an Australian type DCP, in accordance with the associated Australian Standard. This apparatus has a correlation between CBR and DCP penetration of :

$$\log(CBR) = 2.966 - 1.518\log(mm/blow) \qquad \text{Eqn. 10}$$

Belgian work, reported by the CRR (1980)[100] shows a correlation between CBR and penetration using a 'Belgian' type DCP, which comprises a 10 kg drop hammer through a 500 mm dropping distance such that:

$$\log(CBR) = 2.58 - 1.31 * \log(mm/blow) \qquad \text{Eqn. 11}$$

Various equations are presented graphically in Figure 4-2 which shows that the relationship between CBR and DCP penetration can be generalised as follows:

$$\log(CBR) = a - b * \log(m/blow) \qquad \text{Eqn. 12}$$

Where a and b are constants which depend not only on soil type and conditions but also on the specific characteristics of the apparatus.

The DCP test appears to be mostly a test in which failure is induced as the conical tip causes large shear displacements of one soil/aggregate particle relative to another. For this reason, a better correlation with strength than with stiffness (resilient modulus) is anticipated. (In an attempt to increase the assessment of the stiffness element adaptations to the equipment were made. This work is reported in Section 4.2.6. There is a well known, but imprecise and variable relationship between soil/aggregate strength and stiffness (resilient modulus) it is not surprising that some correlation could be determined as in Equation 14. It may be, also, that the dynamic element of loading is partly resisted by a resilient element which is observed in the correlation of DCP value and stiffness (resilient modulus)

With the relatively high correlation coefficient for the data the above relationships can reasonably be used for materials with high CBR. The relationship is less certain for soils with low CBR values. In such cases a correlation should be found for that specific soil. Fortunately, for the current application, CBR values are not likely to be very small for candidate aggregate.

Ayers et al (1989) studied the correlation between DCP and triaxial shear strength on a variation of materials ranging from clay to ballast. Correlation coefficients were very high when correlation's were made for separate materials. However, the correlation for all the tested materials was very weak. They concluded that the triaxial strength is related to DCP penetration linearly as follows[101]:

$$S = a - b * (mm/blow) \qquad \text{Eqn. 13}$$

Where S Triaxial deviator stress at failure

a & b Constants (dependent on soil and confining pressure)

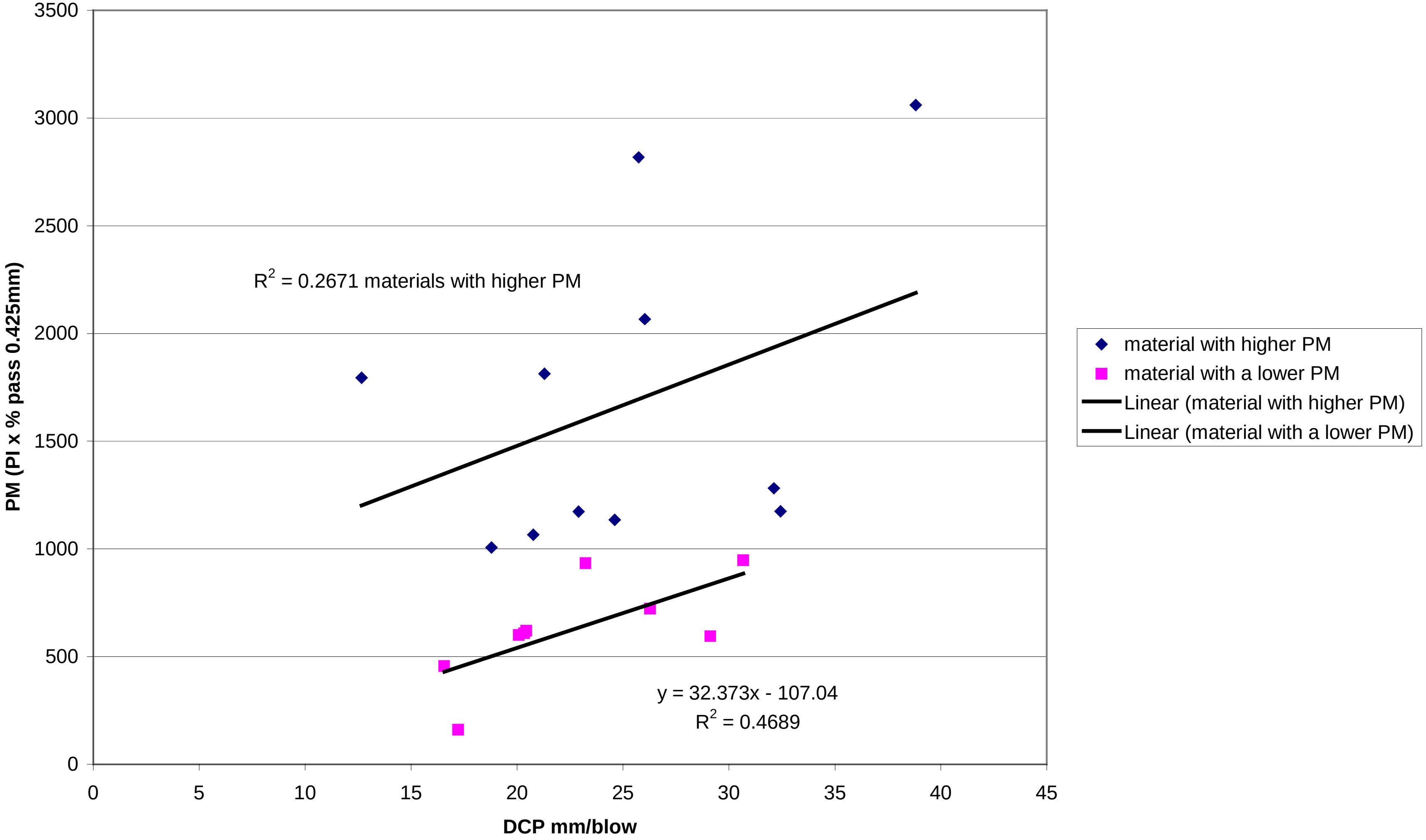
[98] McElvaney, J., Jayaputra, A., Harison, J., Correlation of CBR and dynamic cone strength measurements of soils, Proceedings of 3rd Indonesian Conference on geotechnics Volume 1, Jakarta 1985.

[99] Scala, A.J. (1956) "Simple methods of flexible pavement design using cone penetrometers", New Zealand Engineer, Vol. 11, No. 2, pp. 34-44

[100] CRR (1980) "Light Percussion sounding apparatus" CRR leaflet A25, Centre de Recherches, Routieres Belgium.

[101] Ayers, M. E., Marshall, R., Uzarski, D. R., "Rapid shear strength evaluation of in-situ granular materials", TRB 1227, 1989, pp134-147.

Figure 4-2- Small Barrel Test With DRY Material - Plot of Average Dcp against Plasticity Modulus Uganda and Fiji Materials Plotted together



The elastic modulus of road construction materials is essential in the mechanistic design of pavements. Du Plessis and De Beer (1996) found the following equation, which is of similar form to the DCP-CBR equations above, although with a greatly reduced correlation to those prevailing in the preceding relationships to strength.

$$\log(E_{eff}) = 3.04758 - 1.06166 * \log(mm/blow) \qquad \text{Eqn. 14}$$

Where E_{eff} is the resilient modulus of the material

The elastic modulus in the above equation was back-calculated from Heavy Vehicle Simulator (HVS) measurements [102]

4.2.4.1 Selecting The Equipment

Consideration was given to each of the pieces of equipment and a criteria was used to assess the appropriateness of each. A set of criteria for an *ideal* condition assessment instrument for rural roads was developed. The assessment was based upon each factor (such as "is the equipment simple to use and cheap?". If the answer was YES then the equipment was "good" and thus 2 points were awarded. If the answer was NO then the equipment was not satisfactory and 0 points were awarded. If the equipment partially met the requirements of the question only 1 point was awarded. The assessment was obviously subjective, however it was felt to be the most appropriate method of choosing a piece of equipment to adapt.

On this basis, as shown in Table 4-4, the DCP scored the most points using the given criteria and was therefore adopted for further investigation.

- CBR = California Bearing Ratio 0: not satisfactory
- FWD = Falling Weight Deflectometer 1: satisfactory
- DCP = Dynamic Cone Penetrometer 2: good

Table 4-4 - Compliance of Various Road Assessment Apparatus to the Selected Criteria

Criteria / **Apparatus**	**Simple to use and cheap to buy**	**Easy to manufacture and maintain in a**	**Light and Portable**	**Not sensitive to human error**	**Fast to perform the testing**	**Accurate**	**Measure stiffness (resilient modulus)**	**Measure strength**	**No advanced support**	**Total points awarded**
CBR	1	1	0	1	1	0	1	1	1	7
Plate loading	2	2	1	1	0	1	1	0	2	10
Benkelman Beam	0	1	0	0	1	1	1	0	0	4
Deflectograph	0	0	0	1	2	1	1	0	0	6
Dynaflect	0	0	0	0	2	2	1	0	0	5
Natural Vibration	1	0	0	1	2	1	1	0	0	6
Compacttion-meter	0	0	0	0	2	1	1	0	0	4
FWD	0	0	0	1	1	2	2	0	0	6
Surface wave	0	0	0	0	0	0	1	0	0	1
Pressure-meter	1	0	2	1	0	1	2	2	0	9
Clegg Hammer	2	0	2	2	2	0	1	0	2	11
Cone Penetmeter	2	1	2	2	2	1	1	1	1	13
DCP	2	2	2	2	2	1	1	1	1	14

[102] Du Plessis, 1997 – a verbal communication).

Since the DCP was the device which scored highest in the criteria, it was adopted at the start of the experimental stage of the project with the aim of modifying it to measure stiffness (resilient modulus) more reliably as well as the strength of gravel roads. It also had the potential with suitable modifications, for use in assessing material sampled from borrow pits.

4.2.5 Perth Sand Penetrometer

An Australian standard details the Perth Sand Penetrometer (PSP), which is a flat tipped piece of equipment similar to a DCP used specifically to measure the penetration resistance of sand. *'A flat ended rod of 16 +/- 0.2mm diameter is driven with a 9kg mass, dropping 600mm. Use of the method is limited to granular soils with a maximum particle size not exceeding 2mm, and a layer thickness of at least 450mm'*[103]. The PSP is similar to the DCP, but uses a slightly heavier weight, dropped from a slightly greater distance.

It was decided to try and use the DCP as the piece of apparatus for modification purposes, rather than the PSP, but the issue of using a flat tip was considered to be worthy of further investigation. A paper produced by Glick and Clegg (1965)[104], details the calibration of the equipment, with a particular emphasis on the effect of density and moisture and the driving resistance. The equipment was intended to be used on sand, and therefore the results were not directly of interest, as it was not intended to test sand as part of the current project. However some important points were noted from the work and considered when the modifications were made to the DCP and the subsequent testing was undertaken.

A box was used to test the flat tipped penetrometer and it was noted that the tests undertaken in the centre of the box gave a driving resistance which was greater than that at the corners of the box. This was found to be due to the lower density at the corners of the box.

The effect of drop height in relation to relative density was considered and the testing indicated that the resistance to driving was approximately proportional to the drop height, except where lower densities were encountered. Additionally an approximate linear relationship was found between blows and depth below surface.

The modified DCP has some similarity to the PSP. However the novelty is in the range and size of tips investigated. This work is reported in Section 5.3.

4.2.6 Resilient Modulus or Elastic Stiffness:

The term used to describe the resilient stress strain ratio in this report is resilient modulus (M_r). The word 'elastic' is inappropriate due to the non-linear hysteretic nature of resilient behaviour in soils and aggregates and because of some time-dependency indicating a viscous component. In evaluating it numerically it is defined as the ratio of the applied transient stress pulse to the strain recovered upon the unloading part of the pulse. Often an equivalency with Young's Modulus (E) is adopted for convenience even though this is not defensible on the grounds of differing definitions.

In roads the resilient modulus of a layer and its thickness are directly related to the load distribution ability of that layer, (see Section 4.1.2).

Boussinesq (1885) published an equation for calculating stresses and displacement in an elastic half-space under point loading[105]. This was followed by an analytical solution for a three-layer pavement structure[106] (Burmister, 1943). With the advent of computers, the numerical modelling of pavements (which takes into account various parameters and models)

[103] Australian Standard, Methods of testing soils for engineering purposes, Method 6.3.3: Soil strength and consolidation tests – Determination of the penetration resistance of a soil – Perth Sand Penetrometer test, AS 1289.6.3.3 – 1997, p1.
[104] Glcik, G.L., Clegg, B., use of a Penetrometer for site investigation and compaction control at Perth, W.A., The Institution of Engineers, Australia, 1965.
[105] Boussinesq, J., (1885) "Applications des potentiels a l'etude de l'equilibre et du mouvement des solides elastiques", Gauthier-Villars.
[106] Burmister, D.M., (1943) "Theory of stressess and displacements in layered systems and applications to design of airport runways", Highway research Board, pp. 126-148.

was adopted. In all these methods, the stress-strain relationship of pavement materials and formation soil are expressed in terms of the basic elastic parameters, resilient modulus and Poisson's ratio (E and ν respectively). Due to damage by trafficking of the pavement materials resulting in the reduction of resilient modulus, the quasi elastic response of the pavement is found to be a fundamental indicator of the remaining life of the pavement. This effect, in addition to the direct relationship between M_r and the load distributing ability, has led to the recognition that M_r is a key parameter in the design and performance assessment of roads.

In essence, damage by trafficking of the pavement materials results in the reduction of the resilient modulus, and the elastic response of the pavement is found to be a fundamental indicator of the remaining life of the pavement. This effect in addition to the relationship between E and the load distributing ability has led to the recognition that E is a key parameter in the design and performance assessment of roads. In its simplest form the resilient modulus of road construction materials can be approximately related to the first invariant of stress (theta) as follows:

$$M_r = k_1.\left(\frac{\theta}{P_a}\right)^{k_2} \qquad \text{Eqn. 15}$$

Where:	2	3p, p is the first invariant of stress =(σ1+σ2+σ3)/3
	P_a	Atmospheric pressure = 100 kPa
	k_1,k_2	Material constants

This model is known as the kθ model and it has been found that k_1 and k_2 depend on the water content of the material as well as the degree and type of compaction and the material itself.

The resilient modulus of soils is often estimated using the CBR. Heukelom & Foster (1960) derived a the following relationship[107]:

$$E = 11 * CBR \qquad \text{Eqn. 16}$$

Shell (1978)[108] use a similar equation in their design code, namely:

$$E = 10 * CBR \qquad \text{Eqn. 17}$$

TRRL (1984) derived another equation based on comprehensive analysis of wave propagation data and cyclic triaxial testing:

$$E = 17.6 * (CBR)^{0.64} \qquad \text{Eqn. 18}$$

However, Brown et al (1987)[109], while agreeing that the shape of the E:CBR relationship is better modelled by the last equation, showed that the actual relationship was still very sensitive to soil type, speed of loading and transient stress pulse magnitude. The last equation can only be considered as giving a crude mean value.

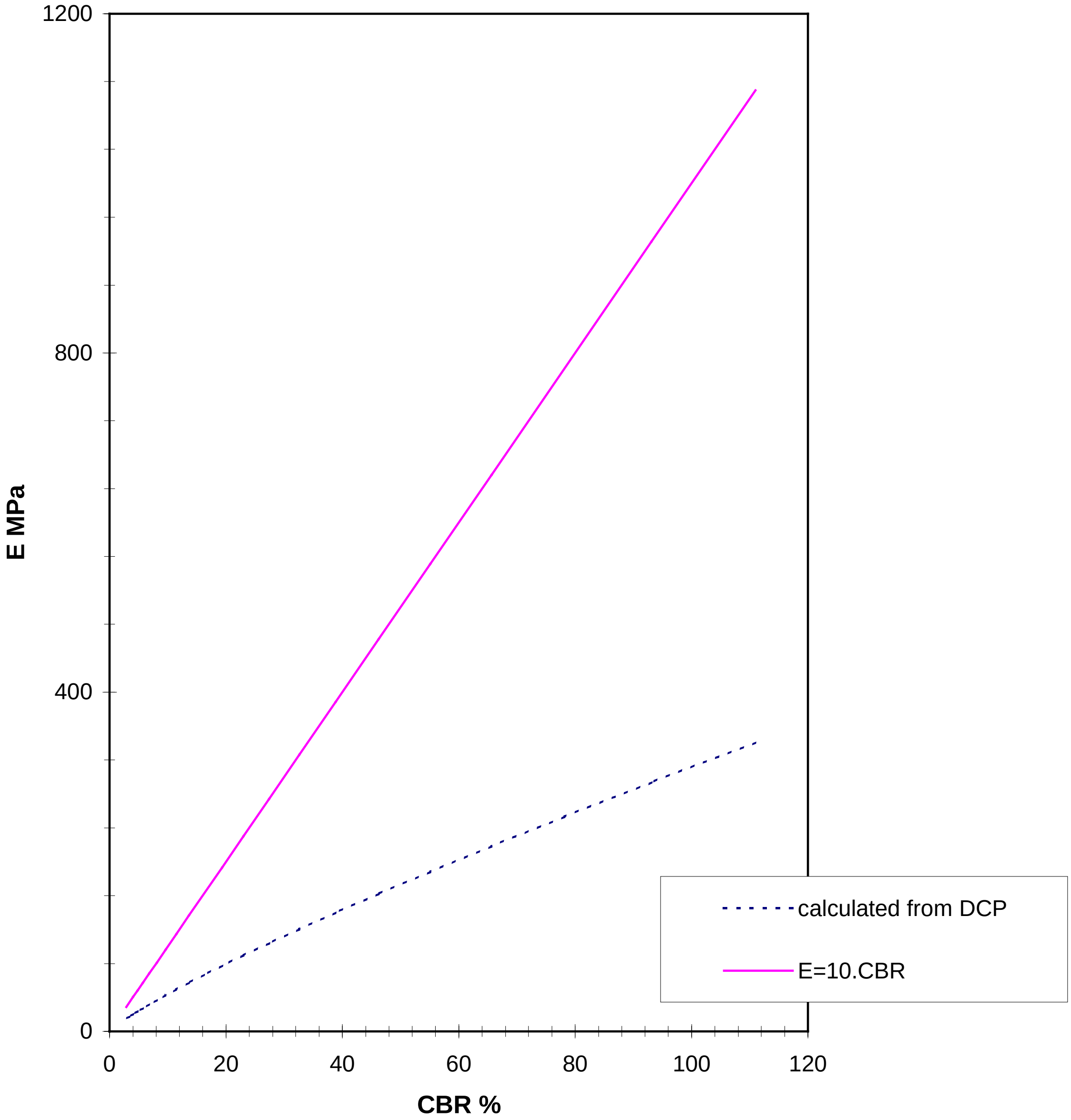
CEBTP (1984) proposed that only orders of magnitude for the dynamic (or static) modulus for materials forming pavements may be obtained. The initial approximation of the dynamic modulus may be measured on site with vibrator or wave propagator or alternatively in the laboratory using intact samples.

During the work reported here a relationship between E and CBR was derived using an average value from the relationships between CBR and DCP (Excluding Livneh and Ishaia (1987) equation because it has a different form). Then the stiffness was calculated from

[107] Heukelom, W. and Foster, C. R. (1960) "Dynamic testing of pavements", ASCE Journal of Structural Division, SMI, pp. 1-29
[108] Shell International Petroleum Company, The Shell Pavement Design Manual, 1978.
[109] Powell, W.D., Potter, J.F., Mayhew, H.C., Nunn, M.E., The structural design of bituminous roads, TRRL, Report LR1132, 1984.
[109]BROWN, S F, LOACH, S C and O'REILLY, M P, "Repeated loading of fine grained soils", TRRL Contractor Report 72, 1987.

Du Plessis and De Beer's equation[110.] This relationship is plotted in Figure 4-3 along with other equations. The figure shows that this relationship derived from the DCP data fitted well within the range of other equations that have been derived by using advanced testing techniques.

Figure 4-3 - Relationship between Resilient Modulus and CBR



[110] CEBTP, A practical guide to pavement design for tropical countries. French Republic, The Ministry of Cooperation, 1984, pp100-101.

[110] Scala, A.J. (1956) "Simple methods of flexible pavement design using cone penetrometers", New Zealand Engineer, Vol. 11, No. 2, pp. 34-44.

[110] Van Vuuren, D.J. (1969) "Rapid determination of CBR with portableDu Plessis, L. and de Beer, M. (1996) "Characterisation of road building materials using the dynamic cone penetrometer", The Rhodesian Engineer, September.Presented at the 75th annual meeting of the Transportation Research Board, Washington D.C.

4.3 Summary

The main points put forward in Section 4 of the report were discussed and may be summarised as follows:

- The structural approach to design and the fundamental contribution of materials was introduced.
- The importance of field tests as compared to laboratory tests was discussed and a brief overview of a number of different field tests was given, and their suitability for use in a low cost road application was assessed.
- Criteria for measuring the suitability of different types of equipment were developed, and the different field tests were considered against the criteria.
- A detailed description of the DCP penetration rate against CBR has been undertaken.
- Criteria were used to allow the choice of suitable equipment for modification to be made and the DCP was chosen as the most appropriate.
- Lastly the resilient modulus and stiffness were explained. In addition to the direct relationship between E and the load distributing ability E is a key parameter in the design and performance assessment of roads, because it reflects pavement damage and thus indirectly remaining life.

5. EQUIPMENT DEVELOPMENT AND ASSESSMENT

5.1 Equipment Development Work

The development work had two aspects. The first used the existing DCP and the second modified it. Work with the DCP also involved some un-compacted candidate road-building material in a barrel (as a means of assessing materials prior to their use in the road). The aspects of the work are:

i) Assessment of strength (in terms of CBR),

ii) Assessment of resilient modulus (existing and modified DCP)

iii) Assessment of permanent deformation (existing and modified DCP)

iv) Modified usage of the DCP (e.g. drop heights, tips etc).

v) Technique for the assessment of un-compacted road building materials

5.2 Preliminary CBR/ DCP Assessment Work

As discussed in the previous chapter, much research has been conducted in the past to obtain a correlation between CBR and DCP penetration rate. Correlation between these two parameters found by various researchers are presented in Figure 4-2. During this project, CBR tests were performed in the laboratory on soils of various grain sizes including clay, sand and coarse aggregates in a standard CBR mould and then DCP tests were performed in the CBR mould. The results obtained for sand did not follow the same trend as for the aggregate and clay, so the results for sand will be discussed separately in section 5.2.1. The results for cohesive fine grained soils and aggregates showed quite a good agreement with those obtained by previous workers. The results for cohesive fine grained soils and aggregates are superimposed on Figure 4-2 and presented in figure 4-3. The figure shows that there is a reasonable agreement between these results with those obtained by previous workers especially those by McElvaney (1985)[111]. In general, the CBR values obtained in this project were slightly higher than would have been expected, i.e.: those obtained by other researchers. This might, partly, be due to some densification that occurred during the CBR test before performing the DCP.

A relationship between E and CBR was derived using the DCP results as follows: an average value of relationship between CBR and DCP from Figure 4-3 was obtained (Excluding Livneh and Ishaia (1987) equation because it has a different form). Then the resilient modulus was calculated from Du Plessis and De Beer's equation[112] by performing DCP. Because of this good match between the current study's results and previous work on the CBR:DCP penetration rate relationship, it is assumed that the E:DCP penetration rate relationship given in Figure 4-3 and described in Section 4.2.6 is applicable to the current study.

5.2.1 Assessment of DCP-CBR Relationship for Sand

It was mentioned in the previous section that sand did not follow the same trend as other materials. A brief study was conducted to study the behaviour of sand when tested with the DCP. Since the strength of sand is mainly due to friction between grains, it depends on the confining stress. The effect of surcharge weight on the DCP results in a CBR mould was therefore studied by placing surcharge weights with a base plate that had a hole in the centre to allow for the penetration of the DCP rod. The results of this study showed that the surcharge weight led to a reduction in DCP penetration rate. This effect was more apparent in the first drop and up to a surcharge weight of 12 kg. For dry sand, under zero surcharge weight, there was a considerable penetration under the static weight of the DCP. In general moist sand gave a much lower DCP penetration rate compared to dry sand probably

[111] Smith, R.B. and Pratt, D.N. (1983) "A field study of in situ California Bearing Ratio and dynamic cone penetrometer testing for road subgrade investigations", Australian Road Research, Vol. 13, No. 4, pp. 285-294.

[111] McElvaney, J., Jayaputra, A., Harison, J., Correlation of CBR and dynamic cone strength measurements of soils, Proceedings of 3rd Indonesian Conference on geotechnics Volume 1, Jakarta 1985.

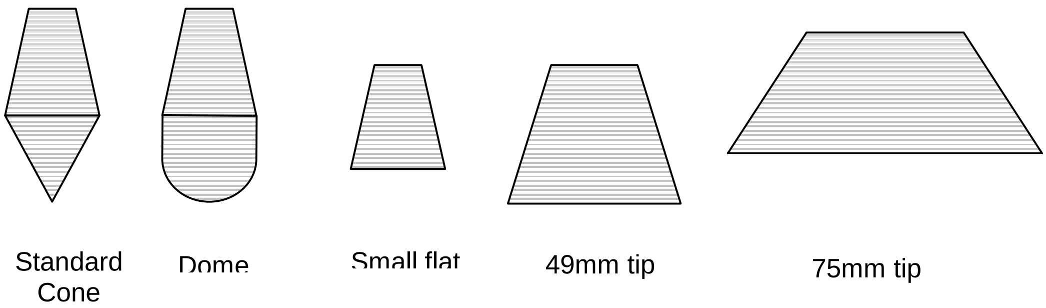
[112] Du Plessis, L. and de Beer, M. (1996) "Characterisation of road building materials using the dynamic cone penetrometer", Presented at the 75th annual meeting of the Transportation Research Board, Washington D.C.

because of the effect of confinement produced by negative pore pressure. Therefore it is concluded that the available shear strength at the surface of a sand layer which is not covered by any other pavement material will be somewhat over estimated by the CBR (in that it delivers some confining stress in order to load the specimen) and the DCP result may, therefore, be more indicative of the direct trafficking case.

5.3 Development of a Modified DCP

The relationship between stiffness (resilient modulus) and DCP penetration rate, as summarised in Figure 4-3and discussed in the preceding Section 5.2, was judged to be of moderate quality. It has already been observed (Section 4.2.4) that the standard cone tip is likely to, primarily, generate localised shear failure and that any relationship of DCP result to stiffness is likely therefore to be coincidental. It was, therefore, considered that the stiffness of the ground would have a greater effect of the response if an alternative shape of cone were to be used. Accordingly several modified tips were developed for the conventional DCP hammer and rods. These are shown in Figure 5-1

Figure 5-1 - Modified DCP Tips



The work carried out on these tips included :

i) Correlating results with the small flat and dome tips and with the existing cone tip, to decide which tip was worthy of a more detailed investigation;

ii) Resilient modulus - DCP assessment using predominately the standard cone tip and the small flat tip;

iii) Rutting - DCP assessment using predominately the standard cone tip and the small flat tip;

iv) Investigations into the predictive apparatus using two different large flat tips (49 mm and 75 mm diameter) in a barrel of uncompacted material.

Initially, test work was performed in the laboratory with the different tips. It became apparent that, although these were useful in developing a broad understanding of the likely behavioural characteristics it would be necessary to test on site for meaningful interpretation. Accordingly DCP tests were carried out alongside resilient modulus assessments in a field trial in Bardon, near Loughborough, U.K. The trial was part of a research project carried out by Loughborough University for the UK Highways Agency. An agreement of collaboration between the team and that of Loughborough was made so as to exchange data between the two projects. Much of the data obtained is, thus, used with the permission of the Highways Agency and Loughborough University which is very gratefully acknowledged.

5.3.1 The Bardon Trial

The Bardon Trial took place on an area of undisturbed waste ground adjacent to a quarry waste tip at Bardon Hill Quarry, Leicestershire. The purpose of the trial was to provide formations of different resilient modulus upon which to compact sub-base. A range of thicknesses of capping were constructed (see Figure 5-2 and Figure 5-3). The trial section comprised a road section with five bays covered with thicknesses of capping ranging from 0 - 400mm and with a 150mm subbase layer. The test bays were 3m long by about 3m wide and three rows of test points were undertaken with three points on a row in each bay. When the section was trafficked the road had to be widened and hence row C was dug out. The subgrade was the natural subgrade at the site - a very gravelly silty clay known as Keuper Marl. Figure 5-3 shows the coding system used for identifying DCP tests. The remaining elements indicate the location : Test Bay; Number; Layer (subbase/capping/subgrade); line in pavement and cross section number.

Figure 5-2 - Bardon Trial

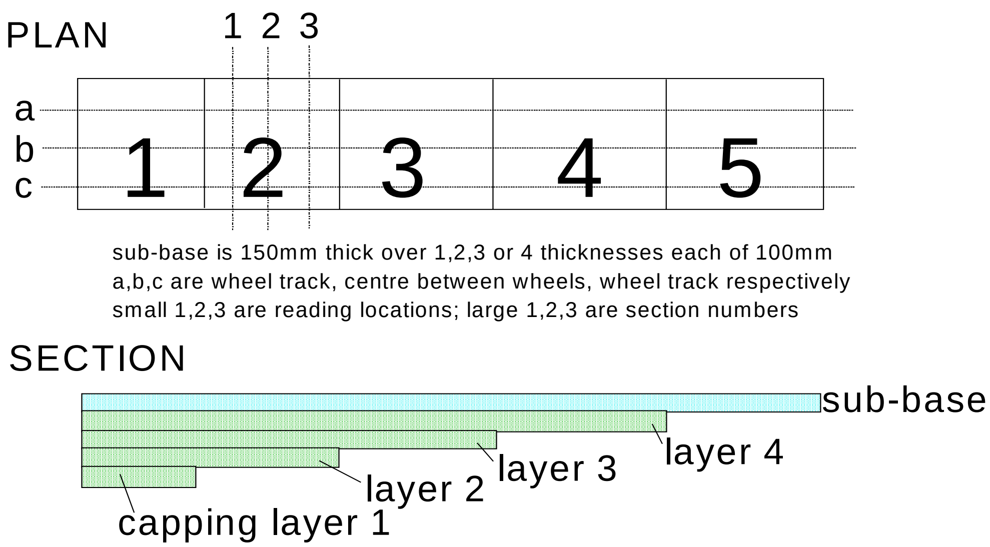
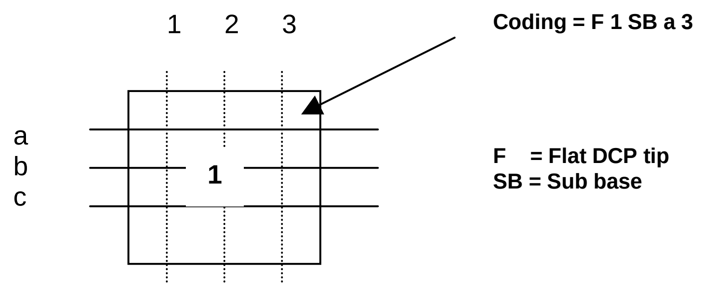


Figure 5-3 - Example of Coding Used for Bays



The site investigation at the initial trial site showed the subgrade to have a CBR of approximately 5% so a range of capping thickness from 400 mm down to 0 mm was considered suitable to provide a sufficient range of formation stiffnesses. The trial width was set at 2.4m to allow two full width runs of the proposed compaction plant with minimal overlap of the roller runs at the centre of the trial. The width was also sufficient to allow three rows of test points to be inset at a sufficient distance from the trial edges to minimise any edge effects on the testing. To provide a level top surface on which to compact the 150 mm sub-base layer, the subgrade excavation was stepped and then the capping was constructed in 100 mm thick layers. Three rows of test points were marked along the length of the trial, with one row along the centreline of the trial (Row b) and two rows offset 0.75m either side from the centre row (Rows a and c). This positioned Rows a and c at the centre of roller runs and Row b at the overlapping joint between the roller passes. Three points 0.75m apart were then marked along each row in each bay (giving nine test points in each bay)[113].

The materials chosen for the trial were standard UK highway construction materials, in accordance with the current Specification for Highway Works (Vol. 1 MCDHW)[114]. The capping used was a Type 6F1, well-graded Porphyritic Andesite, from Bardon Quarry. A finer grading (75 mm down) within the 6F1 envelope was selected by Loughborough University to ensure that no large particles could affect the compaction of the 100 mm layers proposed in design. The sub-base used was a Type 1, well-graded, Porphyritic Andesite from the same supplier. Subsequent grading tests by Loughborough University (BS1377 Part 2, 1990)[115] performed on the supplied 6F1 capping and Type 1 sub-base showed that the 6F1 capping was marginally outside the required grading of the limits given in Table 6/4 of the Specification for Highway Works (Vol. 1 MCDHW). The capping had insufficient fines material so the grading curve lay on the coarse side of the standard grading curves. The sub-base grading complied with the specification for Type 1 (Table 8/2, Vol. 1 MCDHW) Compaction was carried out by Loughborough University with a Benford 1300HV vibrating roller. The 100mm layers of capping were compacted using three passes of the roller (in accordance with the Specification for Highways Works, Table 6/4, Vol. 1 MCDHW) and the sub-base was compacted with eight passes of the roller (in accordance with Table 8/1 Specification for Highways Works, Vol. 1, MCDHW).[116].

Firstly, DCP and resilient modulus tests were carried out on the subgrade soil in each bay in Rows a and c (six tests) before placing the granular layers. The resilient modulus of the subgrade was measured in each bay on all points marked in Rows a and c (i.e. six tests per bay). In Bay 5 (the bay with no capping layer) all test points in all three rows were utilised (nine points).

The resilient modulus was assessed by Loughborough University using the German Dynamic Plate – GDP - (German Specification, TP BF-StB Part B 8.3, 1992) and the TRL Foundation Tester – TFT - (Brown et al., 1993). Both involve dropping a mass onto a plate placed on the ground and using advanced electronic processing to interpret the waveform response of both load and deflection. Resilient modulus measurements were taken using the TFT and the GDP on the sub-base during its compaction. Tests were performed on all points in Rows A and C, in all bays, after two and four passes of the roller. Then after eight passes all points in all rows in all bays were tested.

Next, the DCP tests were repeated at various stages of construction. The DCP tests were carried out at the side of the assigned points to avoid disturbance that might be caused to resilient modulus measurements

113 Confidential Report to HA by Loughborough University, 1999.
114 Specification for Highway Works, HMSO, London, 1990
115 BS 1377, British Standards Organisation, London, 1990
116 Confidential Report to HA by Loughborough University, 1999.

Figure 5-4 - DCP-Stiffness Relationship for Flat Tip (Weighted Ave with Correction)

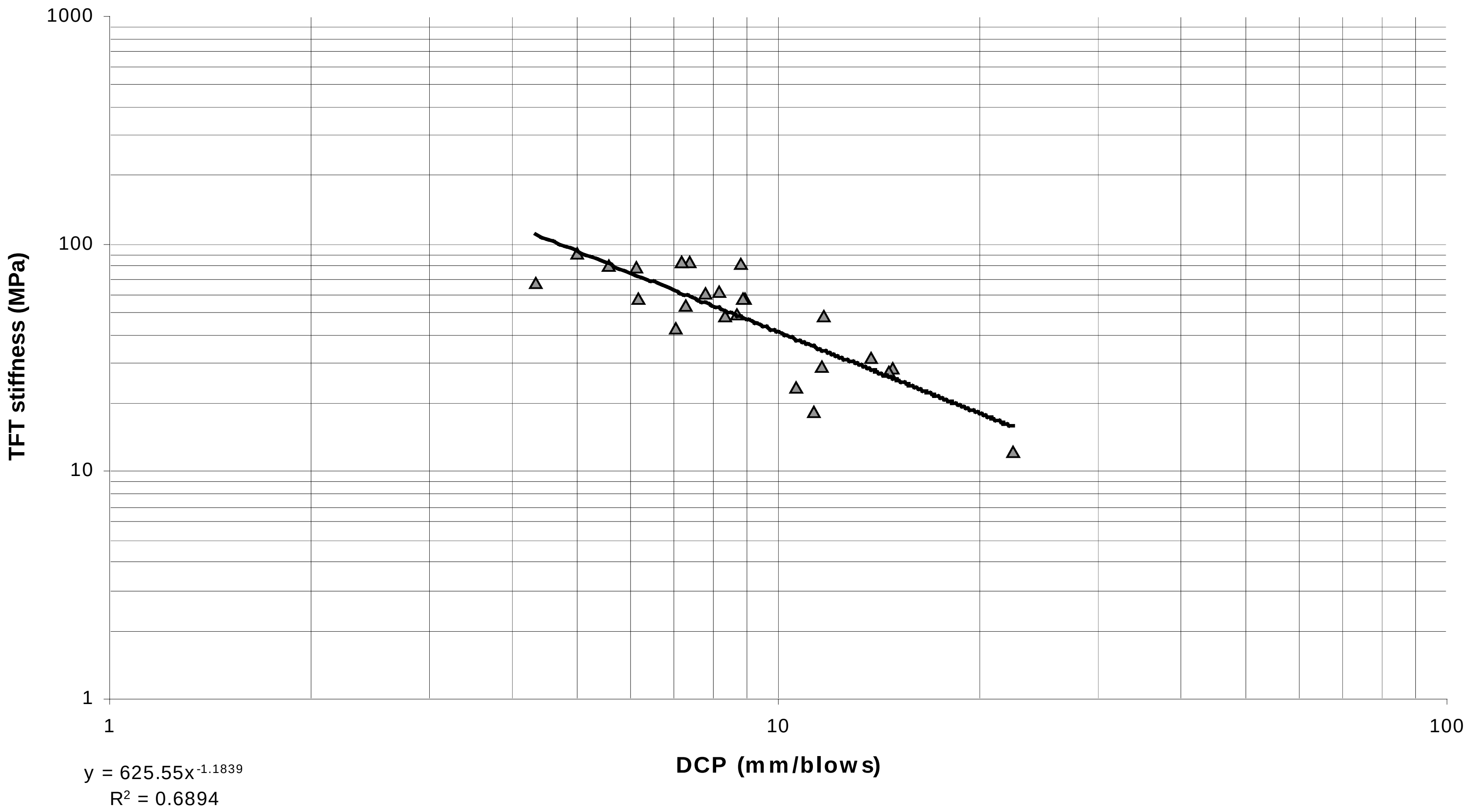


Figure 5-5 - DCP-Stiffness Relationshi for Flat Tip (Weighted Ave with Correction)

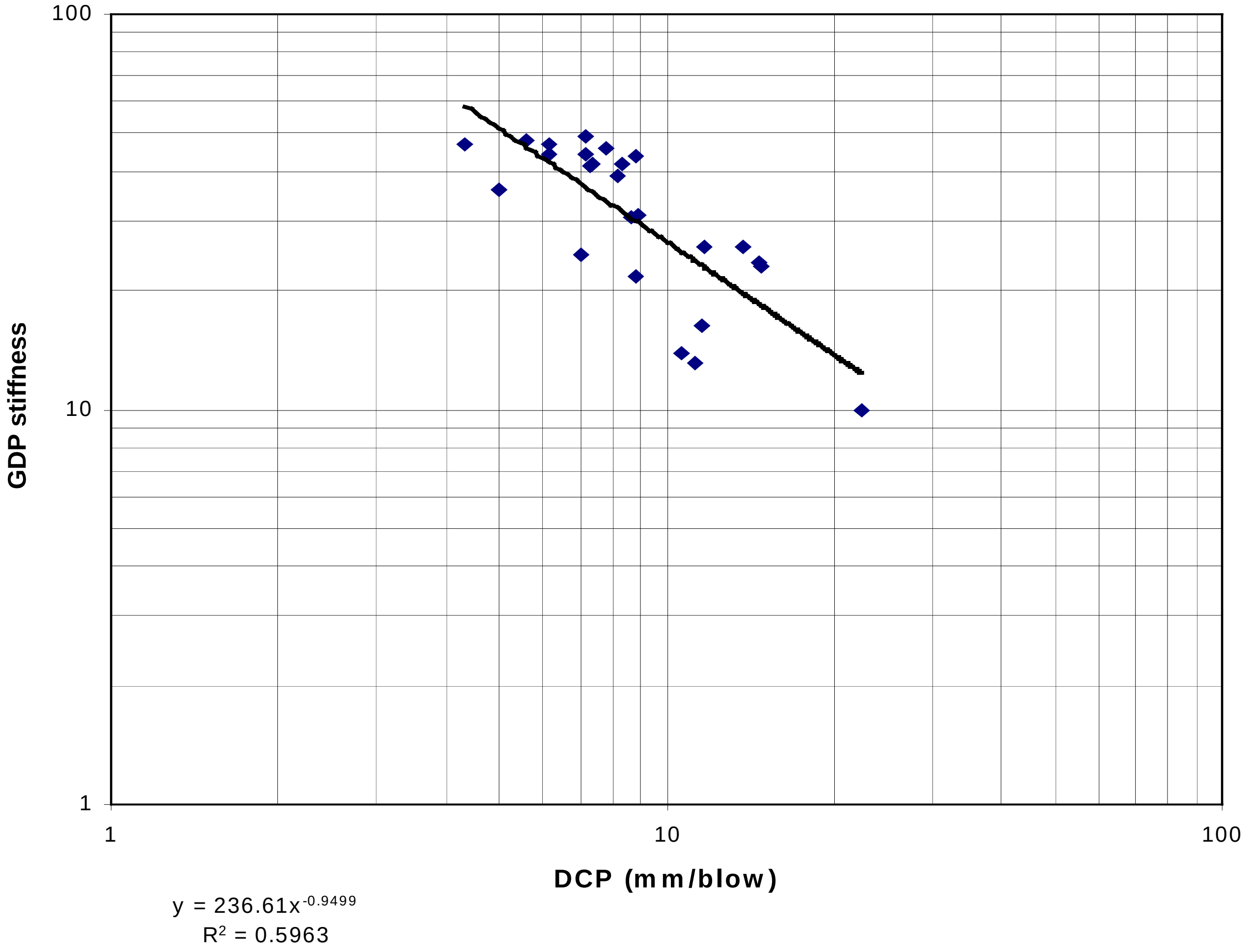


Figure 5-6 - Relationship between GDP And TFT Stiffnesses for Subgrade Soils

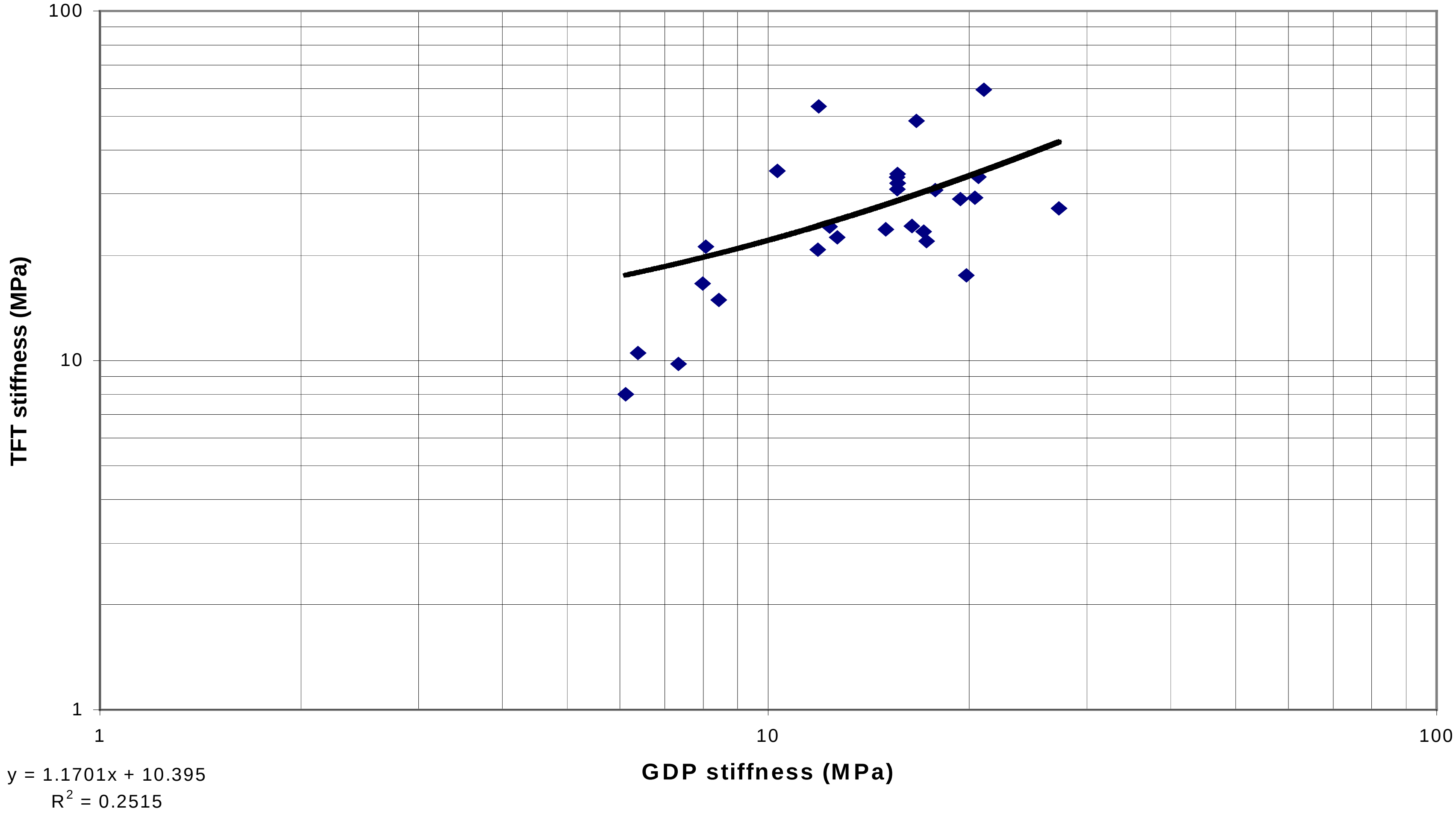
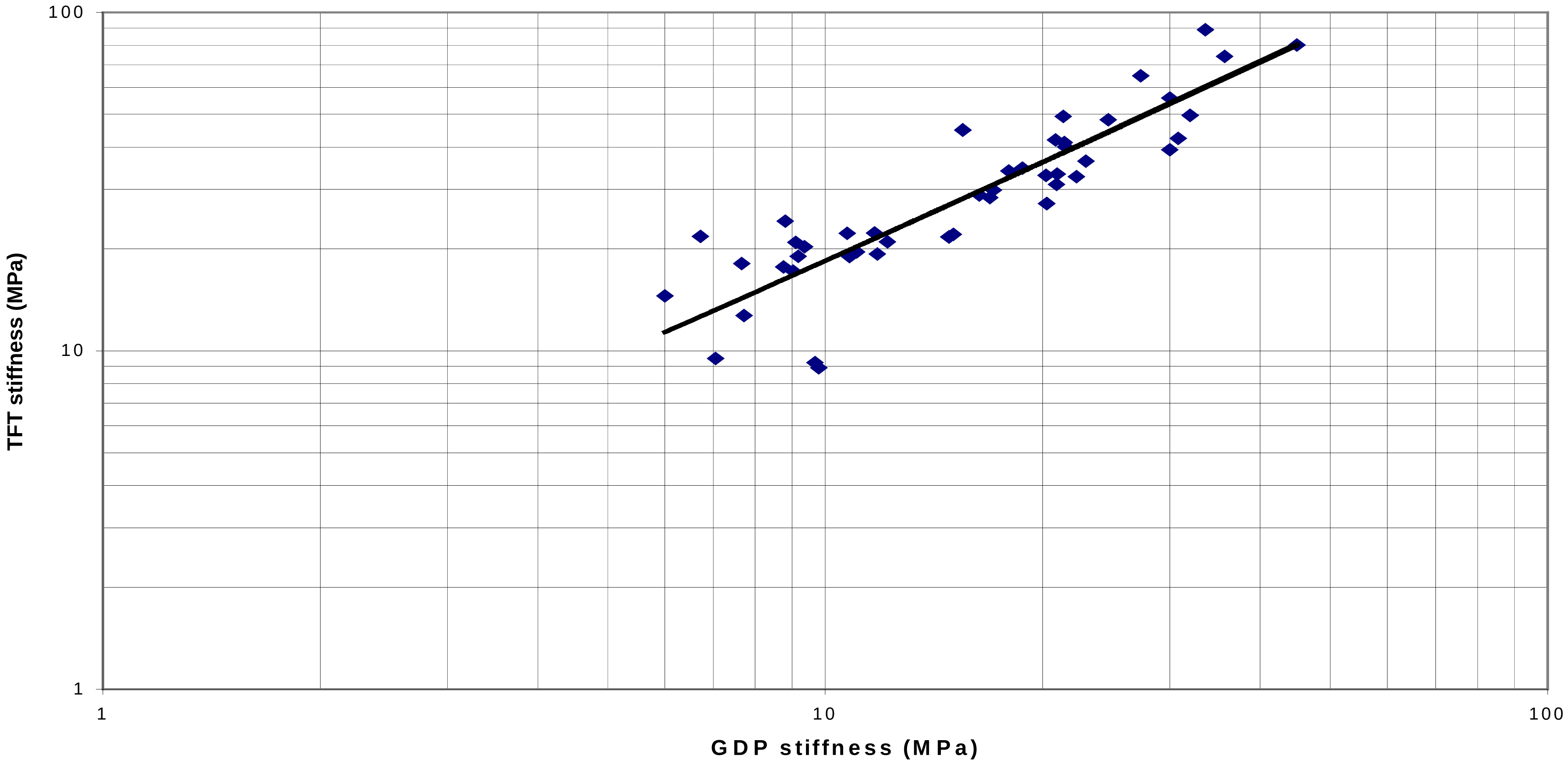


Figure 5-7 - Relationship between GDP and TFT for Unbound Road



y = 1.7681x + 0.7548

$R^2 = 0.7892$

5.3.2 Rutting Trials

In addition to using the data from the Bardon Trial to assess resilient modulus, the data was also used to undertake an assessment of rutting, by using the measured rutting data taken by Loughborough University and by the modified DCP data to assess whether the DCP could predict the susceptibility to rutting of a particular construction. The Bardon trial bays were trafficked with a laden twin axle lorry, which had a single wheel front axle and a dual wheel rear axle. The specification of the lorry are as follows in Box 1:

Box 1 - Details of Lorry Used in Bardon Rutting Trial

Weight of lorry:

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

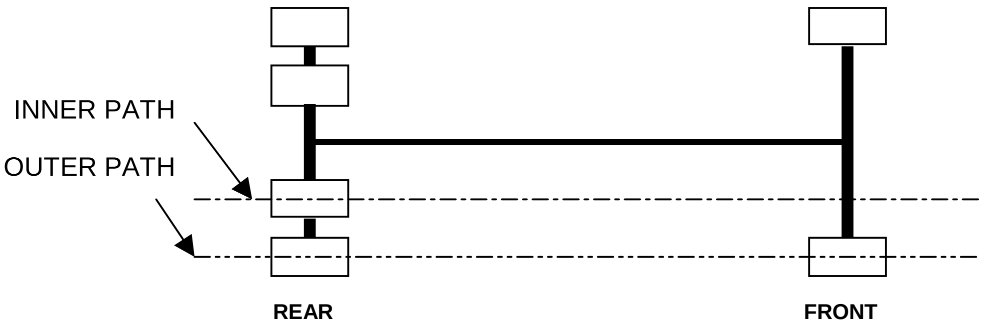
Lorry Dimensions:

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

Tyre Foot Prints:

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

Figure 5-8 - Layout of Lorry Wheels Showing Inner and Outer Wheelpaths



Loughborough University analysed the rutting data as part of their project and noted that when rutting was shown on a logarithmic graph it displayed an approximately linear relationship beyond 10 lorry passes, this is shown in Figure 5-9, Figure 5-10, Figure 5-11 and Figure 5-12. However, the bay with 150mm of subbase and 300mm of capping showed increased rutting (Bay 4), and had trafficking continued to perhaps 500 passes then it was predicted that the sub base would have exhibited a rut depth equal to it's thickness. In the bay with 150mm of sub base and 400mm of capping, the rutting became stable and it was estimated that had trafficking continued to 1000 passes, the resulting surface rut would not have exceeded 50 to 60mm.The thin bays of zero capping and 200mm capping exhibited serious rutting after 20 lorry passes. However the bay with 100mm of capping and 150mm subbase appeared to reach a stable state after 50 passes, although the magnitude of the rut was large. (Trafficking of the bay was however stopped due to damage to the adjoining bays). A summary of the rutting information is provided below in Table 5-1 and Table 5-2.

It is obvious from the graphs, and the observations undertaken by Loughborough that Bay 3 (with 150mm of subbase and 200mm of capping) gave poor performance and rutted severely.

Table 5-1 - Summary of Findings for Outer Path

Passes	Bays	Comments
0-2	1-5	Rutting appeared to be independent of bay thickness
2-12	2	Most rutting seen in this bay
	3, 4, 5	Rutting occurred at a similar rate
12-22	1 & 2	A reduction in the rate of rutting was seen
	Bay 3 & 5	Developed rutting at a rapid rate
	Bay 4	Similar rate to that seen between 2 – 12 passes
After 22	Bay 3 & 5	Failed and Bay 5 was excavated
After 32	Bay 3	Continued to rut and was excavated.
	Bay 1	Reduction in rate of rutting.
	Bay 2 & 4	Rutted at a similar rate to previous.
After 52	Bay 4	No further rutting, at limit and therefore excavated.
	Bay 2	Also reached limit, partly due to problems with Bay 3
	Bay 1	Trafficking ceased prior to limit of Bay 1 being reached.
50 – 150	Bay 2	Reduced rutting but then more progressive after 150.
52 – 250	Bay 1	Very slow rutting

Figure 5-9 - Sub-base Rut Formation in the Inner Wheel Path for Trial Sections having Different Supporting Capping Thickness

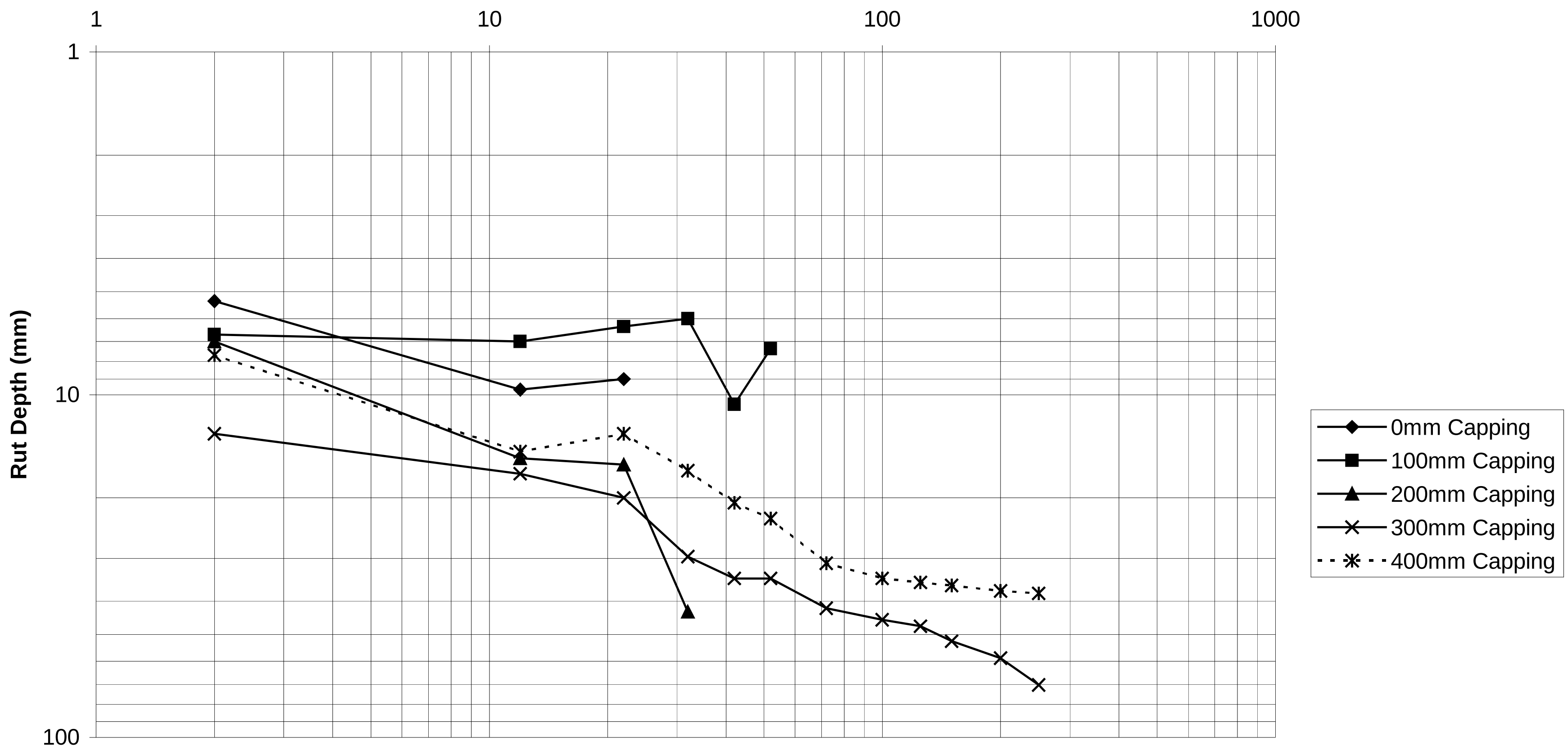


Figure 5-10 - Sub-base Rut Formation in the Outer Wheel Path for Trial Sections having Different Supporting Capping Thickness

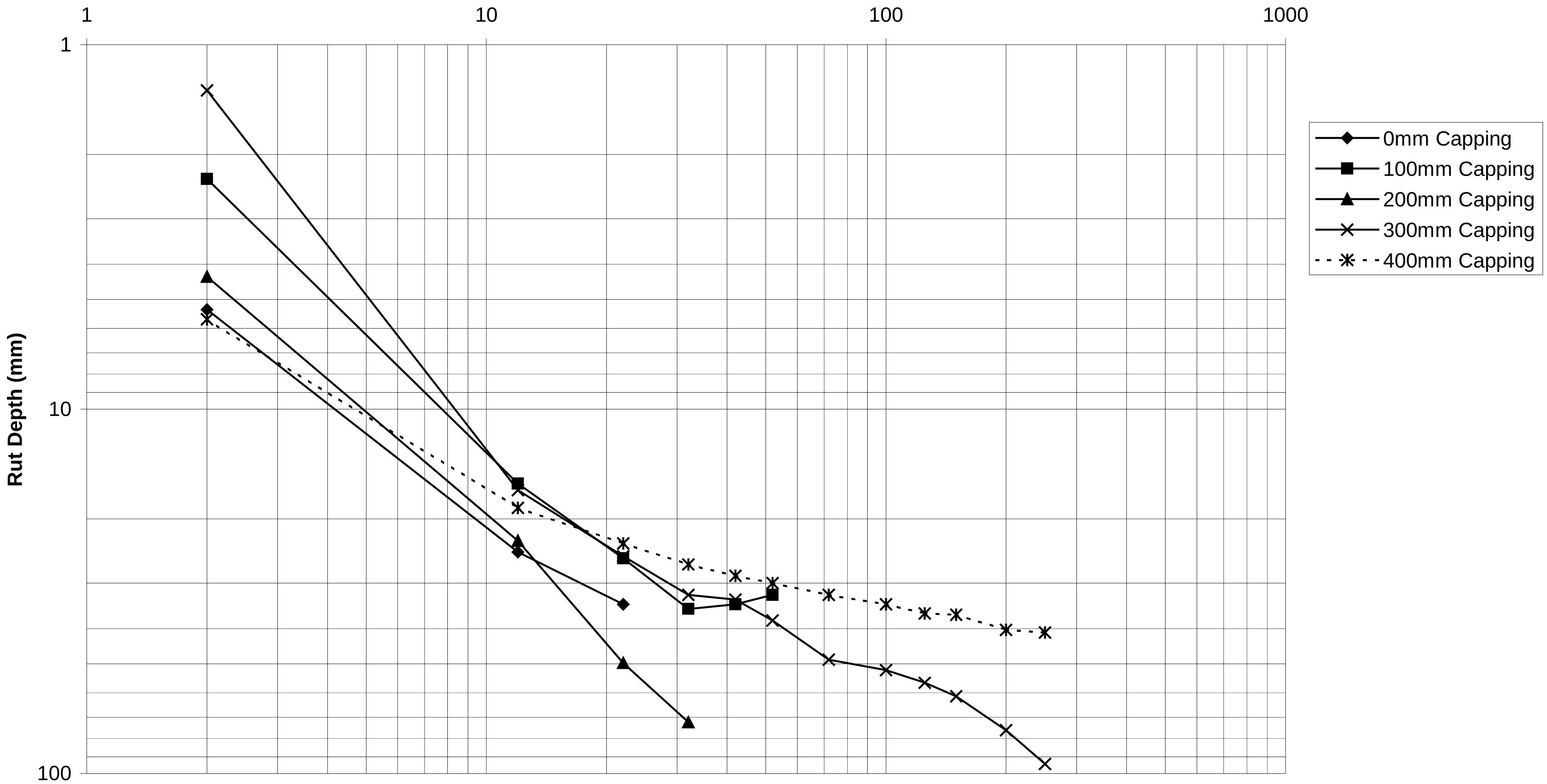


Figure 5-11 - Bardon Rutting Inner Wheel Path

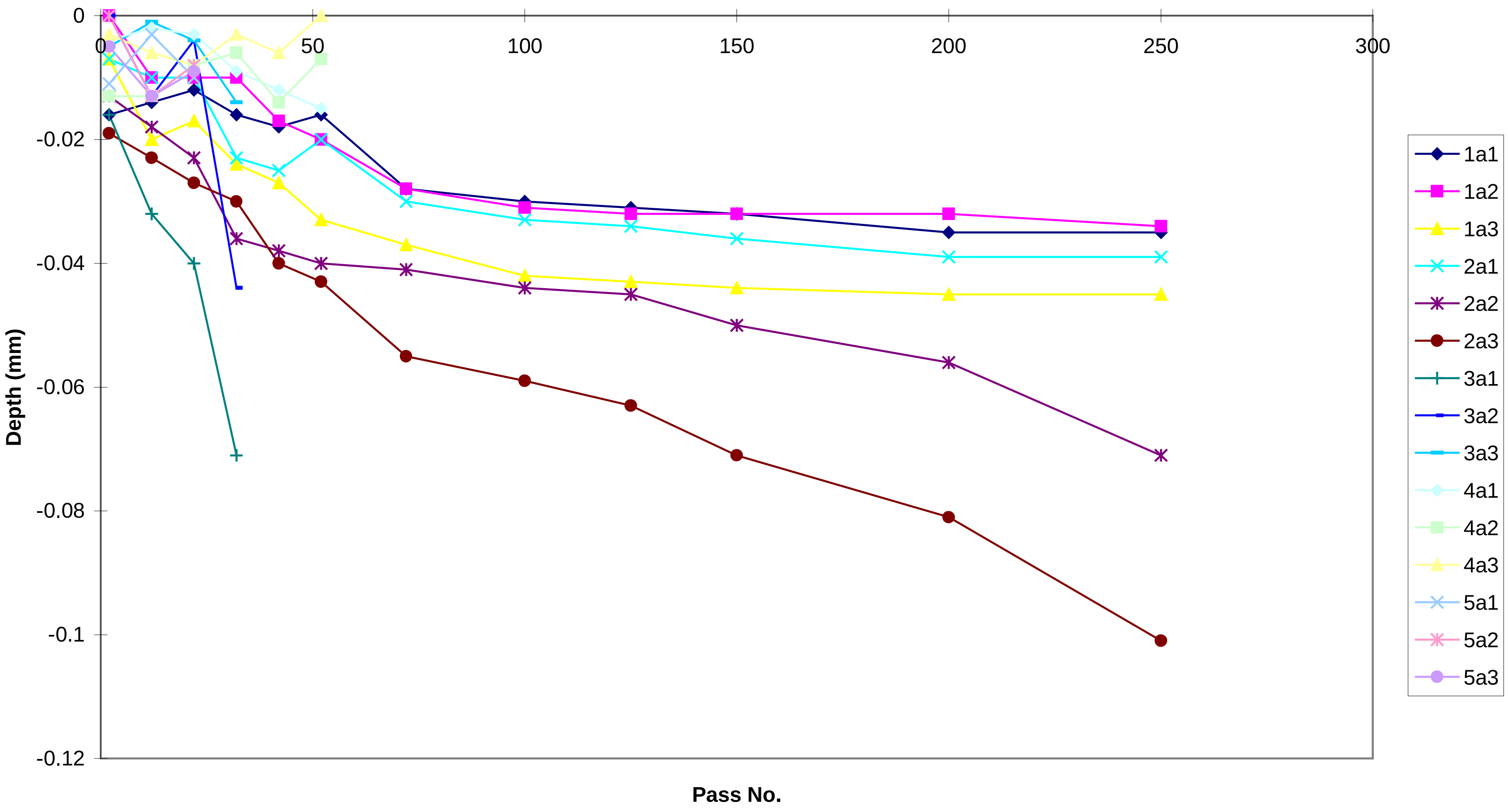


Figure 5-12 - Bardon Rutting Outer Wheel Path

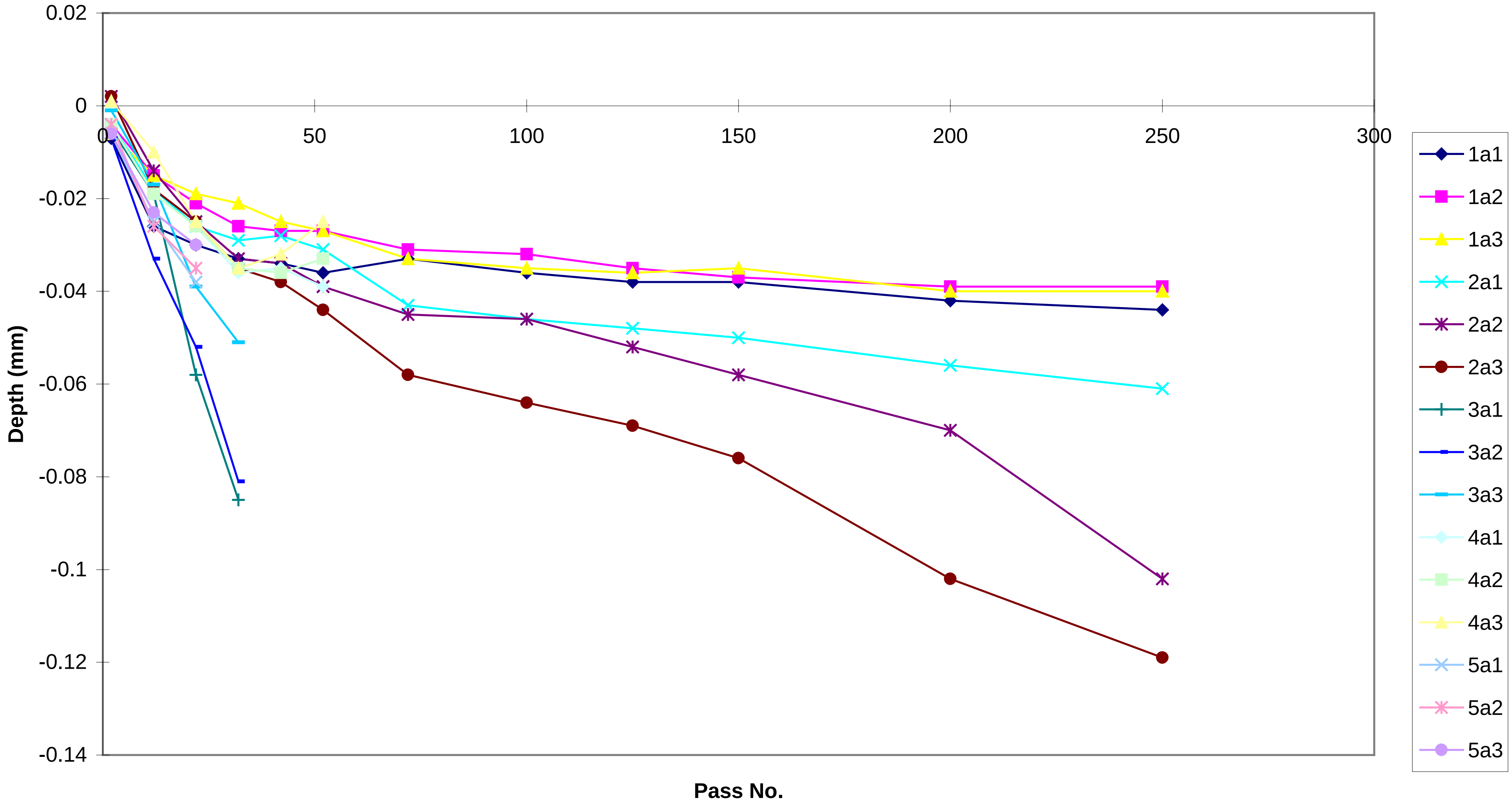


Table 5-2 - Summary of Findings for Inner Path

Passes	Bays	Comments
	Bay 4 & 5	Little rutting at all during trafficking – could be due to build up of rutting in outer path
After 10	Bay 3, 4, 5	Heave at the edges
0 – 22	Bay 3	Showed 10mm ruts but a rapid increase over the next 10 passes meant it was excavated.
	Bay 1 & 2	Similar pattern to outer path shown - faster initial rate which then slowed
50 – 150	Bay 2	Small increase in rutting then an increase to failure
	Bay 1	Little change in rut depth after 100 passes, with magnitude being similar to the outer path.

Table 5-3 - Penetration in Layers for Each Bay using the Flat Tip DCP.

	Bay 1			**Bay 2**			**Bay 3**			**Bay 4**			**Bay 5**		
	F1SBA1			F2SBA1			F3SBA1			F4SBA1			F5SBA1		
	cum blows	Blows/ layer	mm/blow	cum blows	blows/ layer	mm/blow	Cum blows	blows/ layer	mm/blow	cum blows	blows/ layer	mm/blow	cum blows	blows/ layer	mm/blow
SB	25	25	6.0	20	20	7.5	15	15	10.0	16	16	9.4	13	13	11.54
Cap 1	44	19	5.3	42	22	4.5	30	15	6.7	23	7	14.3			
Cap 2	68	24	4.2	64	22	4.5	41	11	9.1						
Cap 3	83	15	6.7	73	9	11.1									
Cap 4	106	23	4.3												
SG	**113**	**7**	**2.1**	**90**	**17**	**19.6**	**69**	**28**	**15.5**	**40**	**17**	**18.8**	**49**	**36**	17.20

	Bay 1			Bay 2			Bay 3			Bay 4			Bay 5		
	F1SBA2			F2SBA2			F3SBA2			F4SBA2			F5SBA2		
	cum blows	Blows/ layer	mm/blow	cum blows	blows/ layer	mm/blow	Cum blows	blows/ layer	mm/blow	cum blows	blows/ layer	mm/blow	cum blows	blows/ layer	mm/blow
SB	25	25	6.0	19	19	7.9	13	13	11.5	15	15	10.0	10	10	15.00
Cap 1	46	21	4.8	42	23	4.3	30	17	5.9	22	7	14.3			
Cap 2	79	33	3.0	70	28	3.6	46	16	6.3						
Cap 3	94	15	6.7	82	12	8.3									
Cap 4	108	14	7.1												
SG	**121**	**13**	**16.9**	**100**	**18**	**17.8**	**77**	**31**	**13.3**	**56**	**34**	**15.7**	**27**	**17**	35.60

	Bay 1			Bay 2			Bay 3			Bay 4			Bay 5		
	F1SBA3			F2SBA3			F3SBA3			F4SBA3			F5SBA3		
	cum blows	Blows/ layer	mm/blow	cum blows	blows/ layer	mm/blow	Cum blows	blows/ layer	mm/blow	cum blows	blows/ layer	mm/blow	cum blows	blows/ layer	mm/blow
SB				16	16	9.4	15	15	10.0	13	13	11.5	12	12	12.50
Cap 1				36	20	5.0	37	22	4.5	22	9	11.1			
Cap 2	No data available			61	25	4.0	50	13	7.7						
Cap 3				72	11	9.1									
Cap 4															
SG				**85**	**13**	**25.0**	**82**	**32**	**13.3**	**72**	**50**	**10.4**	**53**	**41**	15.40

5.4 Comparison of In-Situ Results

The first three or four areas of investigation as detailed in Section 5.3 are now discussed in the light of the in-situ stiffness and rutting measurements from the Bardon trial, together with the DCP results

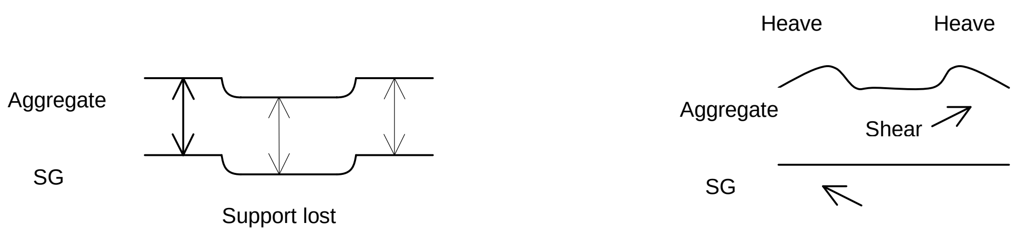
DCP tests for four stations in Bay 1 of the Bardon field trial (150mm subbase, 400mm capping, subgrade) were tested using different tips. The general trend was similar for the three tips but the flat tip gave the lowest DCP penetration rate readings while the dome gave readings in between the flat and the cone. The result is logical since the flat tip is expected to offer the highest resistance, the dome tip to offer less resistance than the flat, while the cone will have little bearing resistance, its resistance being mainly frictional. All the three tips showed generally higher DCP values (penetration rates) in the first 100mm because of the lack of confinement.

5.4.1 Rutting Results and DCP Testing

DCP testing using the small flat tip was perfomed at the Bardon trial to see if a relation existed between the results from the DCP and the susceptibility to rutting displayed by each bay of the trial, (the DCP data was measured in the outer path prior to the rutting trial taking place). If the plot of passes against rutting (see Figure 5-9, Figure 5-10, Figure 5-11 and Figure 5-12) is considered for the inner and outer paths, it can be seen that Bay 3 generally showed the greatest rutting and failed early, along with Bay 5. Bay 4 lasted slightly longer than Bays 3 and 5. The reason why a 150mm layer of sub base underlain by 100mm of capping should appear to perform better than the bay with 200mm of capping is unknown – according to the analysis undertaken by Loughborough University, the subbase in Bay 4 appeared to reach a stable condition after 50 passes, but the trafficking was discontinued due to the failure of Bays 3 and 5[117].

It is known (Dawson, 1997)[118] that there are at least 2 mechanisms of rutting. In the situation at Bardon, the subgrade support is being lost or there is shear in the aggregate (see Figure 5-13). The results strongly suggest that Bays 5 and 4, with thin aggregate cover, suffer only from the loss of subgrade support. Thus the DCP values in the aggregate are hardly relevant for Bays 5 and 4, although the DCP value in the subgrade may be. For Bays 1, 2 and 3 the DCP in the aggregate is more important. With regard to the DCP results, Bay 2 has a higher mean penetration rate than Bay 1 and Bay 3 a higher mean penetration rate than Bay 2.

Figure 5-13 - Example of Mechanisms of Failure



A) Bad support/ poor subgrade **B) Good support/ poor aggregate**

Furthermore, the result of Bay 4 is, clearly, of a very different order than of bays 1, 2 & 3 (which show broadly similar curves in the Figure 5-14 and Figure 5-15 graphs, confirming that rutting control by a different mechanism is a reasonable explanation there.

[117] Confidential report to HA, Loughborough University, 1999.
[118] Dawson, A. D., University of Nottingham, 1997.

Figure 5-14 - Plot of mm/Blow for Layers Outer Wheel Path (Position A1)

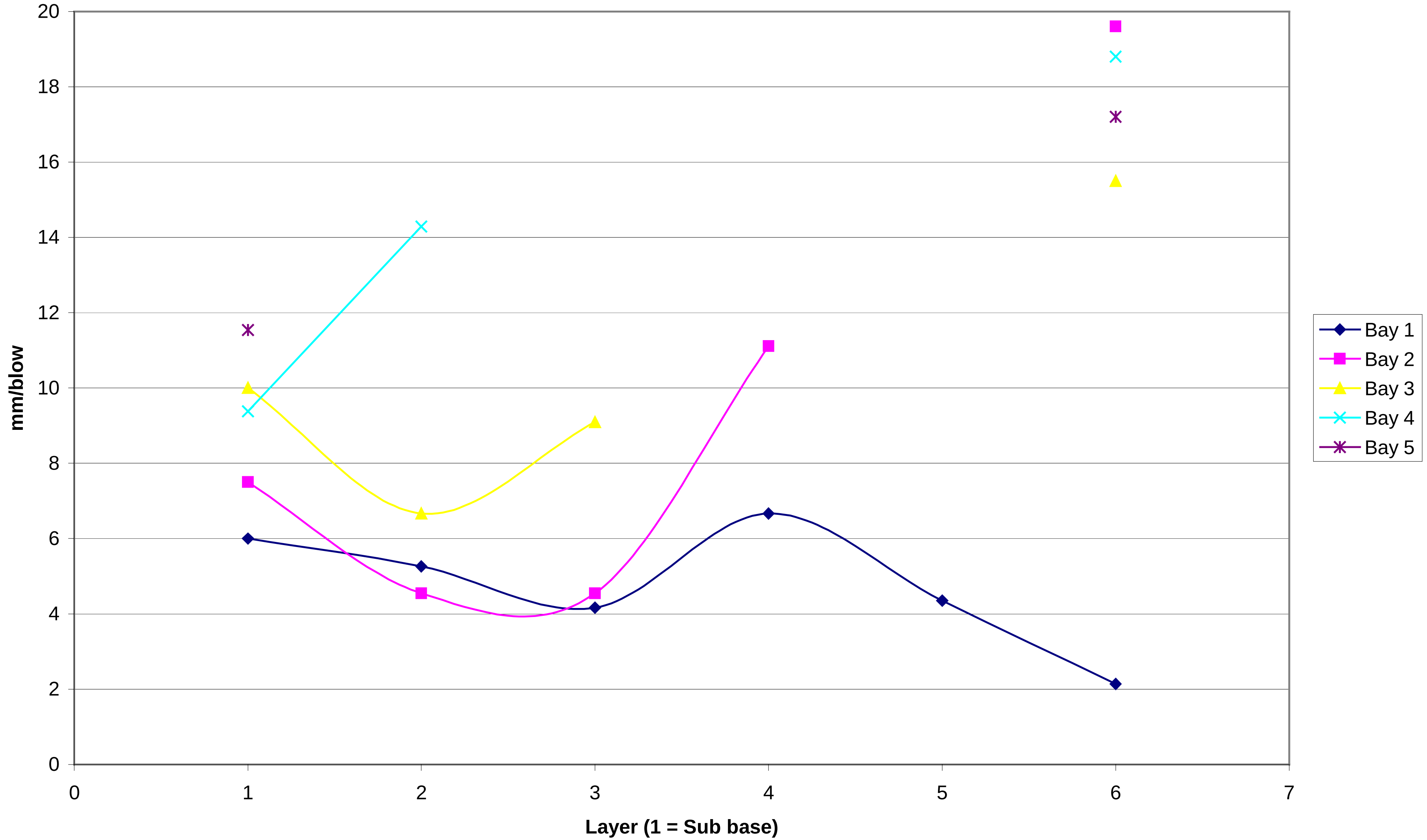


Figure 5-15 - Plot of mm/Blow for Layers Outer Wheel Path (Position A2)

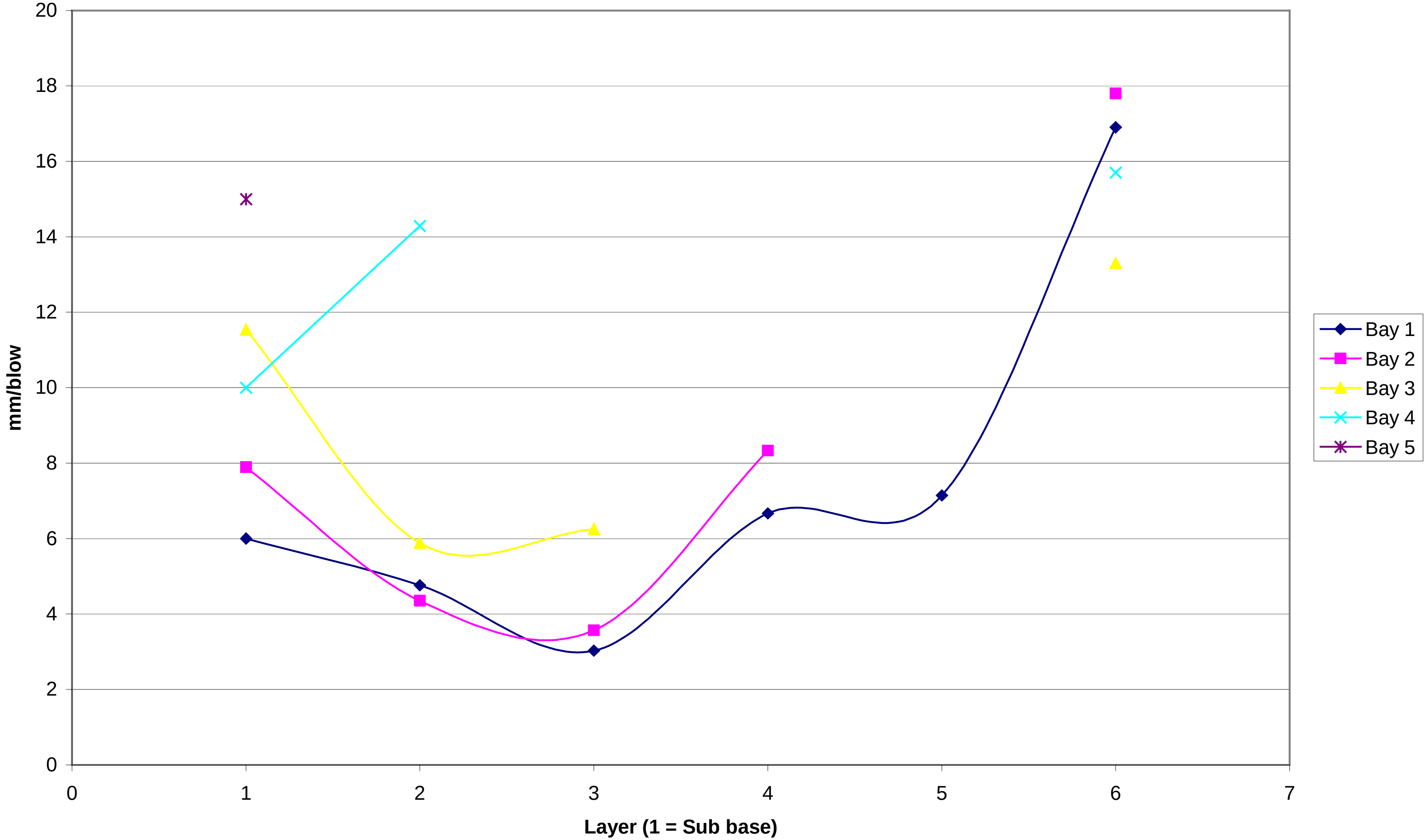


Table 5-4 - Outer Wheel Path Rutting

Bay	Depth Of Capping	Amount Of Rutting
5	0	Most
2	300	Some
4	100	Some
1	400	Little

Bays 4 and 5 do not benefit from improved layers of support and therefore it is assumed that subgrade support was lost and rutting occurred as a result. However, Bays 1 and 2 did have improved layers of support and therefore the rutting was caused by a failure in the aggregate. Loughborough University was not able to directly measure the contribution of the subgrade to the rutting. However they were able to measure the subgrade rutting at the end of the trial by exhuming the sections (see Table 5-5). On this basis the explanation of two rutting mechanisms as described above appears justified for Bays 1, 2, 4 and 5. Bay 3 appears to show an intermediate response. The DCP results suggest that aggregate shear should be a large problem but the exhumation suggests that loss of subgrade has been critical. It is suggested that the former may have contributed to the latter with initial aggregate thinning allowing the soil to be over stressed and consequently leading to subgrade rutting.

Table 5-5 - Transfer of Rutting to Subgrade

Bay	Estimated Transfer Of Ruts To Subgrade
1	No transfer
2	No transfer
3	33 – 50% of rut transferred to subgrade
4	40% of rut transferred to subgrade
5	50% of rut transferred to subgrade

5.5 Resilient Modulus and DCP

Correlation between DCP and resilient modulus for subgrade soils showed that DCP results relate reasonably to resilient modulus. As is shown in Figure 5-20 and Figure 6-2. The scatter in the results is partly due to the natural variability in the field conditions, especially in that the DCP tests were not carried out exactly at the points of the resilient modulus measurements. Additionally the subgrade soil contained cobbles which increased the variability in the test results. However, it is interesting to note that R^2 for the DCP-stiffness relationships were higher than that between the two resilient modulus measurement methods used (GDP & TFT), in the case of the subgrade material.

Thus, there appears to be an inherent difficulty in measuring subgrade soil resilient modulus, independent of the difficulties of using a DCP to do so, and the DCP seems to be no worse than the competing (and far more sophisticated) equipment, such as the GDP and TFT, at least as far as the Bardon situation is concerned.

The DCP value for each tested point used above was the average of the DCP readings (mm/blow) for that point. The values are given in Table 5-3. This method worked well with the subgrade because it was one layer with fairly uniform properties. When aggregates were placed the situation was changed in three ways. Firstly, a 2-layer system is introduced with the possibility of interference effects between the two layers. Secondly, unbound aggregates generally give unrealistically low DCP values because of lack of confinement. Thirdly, DCP results reflect shear strength more than resilient modulus. For these reasons, the correlations between resilient modulus and DCP values were not good with very low R^2 values. Therefore, four improvements were made to the equivalent DCP values (where a DCP value is a single value in mm/blow which represents the entire test in the layer) as follows:

1. The first improvement was to find the rate of penetration in mm/blow for each 100 mm of the penetration depth and then take the average of these readings as the equivalent DCP.
2. The second improvement was to find a weighted average rather than the simple average to take into account the effect of stress distribution so that shallower depths have more influence on the resilient modulus than deeper ones. Using the above two techniques led to a slightly improved correlation between DCP and resilient modulus. However, the overall correlation was still poor.
3. Regarding the top layer correction, the weighted average was further corrected. The correction factors for the top layer for various tips were derived from the gradient of the regression line between the DCP for the top 100mm readings with that of the 200mm.
4. In addition to undertaking the correlation exercise on the cone tip, the same exercise was also undertaken on the newly developed small flat tip and the dome tip. The effect of using dome and flat tips gave lower DCP readings especially at the first two drops. This is due to the bearing effect of the dome and flat tips as compared to the cone tip which has frictional resistance only.

A summary of the deduced relationships before and after these corrections is given in Table 5-6 and Table 5-7.

The testing undertaken as part of the Bardon trial and the subsequent analysis showed that the small flat tip related to resilient modulus measurements using GDP and TFT better than the other tips. The use of a modified tip improved the correlation between DCP readings (mm/blow) and stiffness. The resilient modulus to flat tip relationship having the highest correlation.

5.6 Interim Conclusions

On the basis of the foregone studies, the flat-tipped DCP was adopted as the preferred testing tool for in-situ conditions. In most cases the results were relatable to the conventional CBR values of strength (if desired), but, in the context of this project, related better to the in-situ assessed values of stiffness (resilient modulus) than conventional cone-tipped DCP. With regard to rutting, the flat-tipped DCP gave the same ranking as the observed rutting providing rutting occurred largely within the aggregate. Thus the modified equipment should be valuable as an indicator of pavement rutting problems where the aggregate will be thicker than a certain amount (about 250mm in the Bardon trial) The modified apparatus is illustrated in Figure 5-16.

Figure 5-16 Modified DCP Apparatus

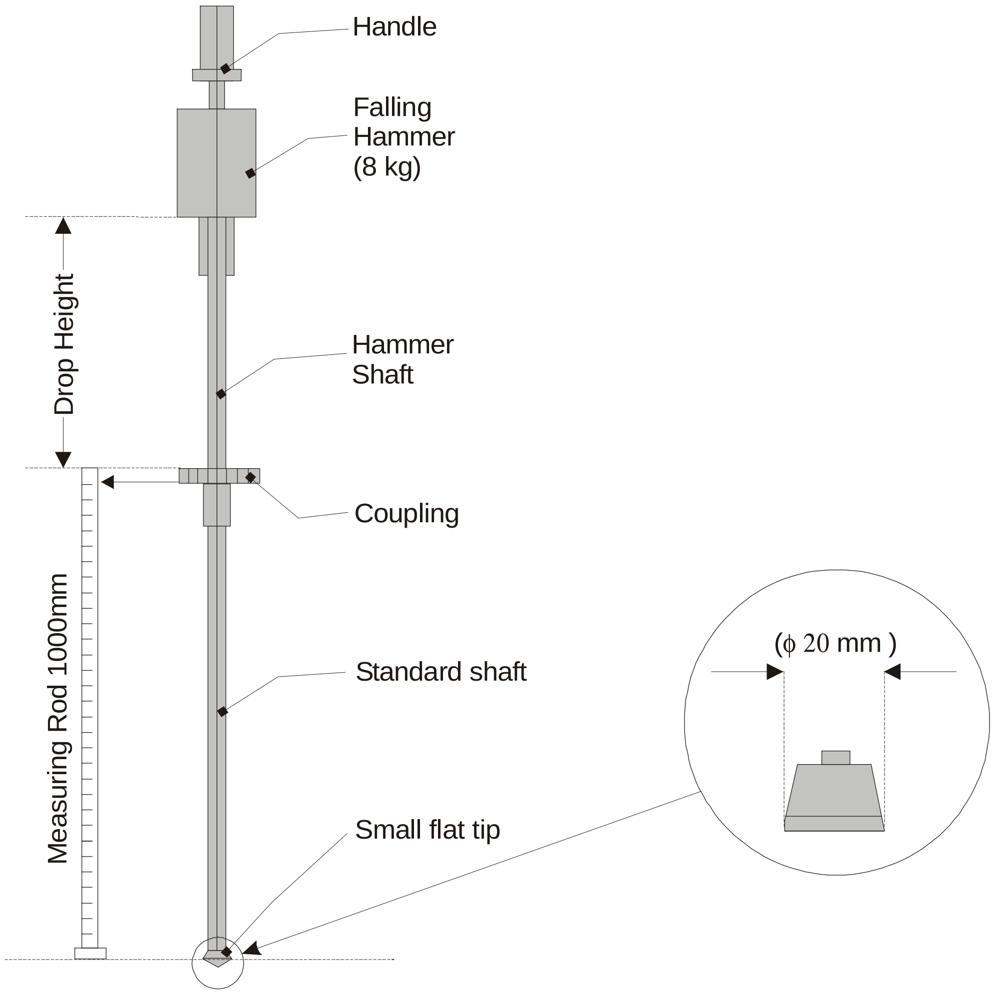
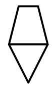
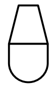


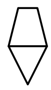
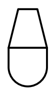
Table 5-6 - DCP and GDP Resilient Modulus Correlations

R^2 for the DCP and GDP resilient modulus correlations for various tips and various modifications of the DCP results.

	Simple Average	Weighted Average	Weighted Average With Top Layer Correction	(Not To Scale)
Cone	0.0712	0.1181	0.1229	
Dome	0.1424	0.3369	0.386	
Flat	0.3992	0.5543	0.5963	

R^2 for the DCP and TFT resilient modulus correlations for various tips and various modifications of the DCP results.

Table 5-7 - DCP and TFT Resilient Modulus Relationship

	Simple Average	Weighted Average	Weighted Average With Top Layer Correction	(Not To Scale)
Cone	0.1222	0.2045	0.2139	
Dome	0.0953	0.2552	0.2913	
Flat	0.5806	0.6168	0.6894	

5.7 Assessment of Road Materials using a Barrel

The DCP was further used to test various types of materials, with a view to developing a predictive Road Materials Assessment test (RMA) for use at borrow pits.

5.7.1 Initial Barrel Tests in the Laboratory

The testing was performed in a barrel (see Figure 5-19), 300mm diameter and 610mm high (a smaller version of a standard oil drum). A barrel was chosen rather than a box because it was felt a shape with no corners would not suffer a reduction in density (at the corners) and therefore affect the measurement of penetration. Such an effect has been noted in the work undertaken by Glick and Clegg (1965)[119]. Furthermore such equipment is readily available in most countries of likely use. The initial laboratory work involved materials being placed in the barrel to a depth of 200mm (this height had been chosen at it allowed the barrel to be rolled without a lid, without material falling out of the barrel. A greater depth of material tended to flow onto the floor when the barrel as turned on it's side). The barrel was then turned over and rolled half a turn then stood upright until the material in the barrel became approximately level again, and hence in a standard 'loose' condition. The procedure had to be performed with minimal vibration to minimise disturbance and to ensure that the density of the aggregate inside the drum was prepared to a standardised condition, as far as possible. The DCP test was then performed in the centre of the barrel using the modified tip but of larger diameter than used in the Bardon Trials (a flat tip of 49mm diameter). Repeatability analyses were employed to study the confidence in the test results and showed that a well-graded crushed granite aggregate yielded acceptable results. See Figure 5-17 and Figure 5-18, which shows the average mean plot for the penetration against blows in the barrel and the minimum and maximum standard deviation.

[119] Glcik, G.L., Clegg, B., use of a Penetrometer for site investigation and compaction control at Perth, W.A., The Institution of Engineers, Australia, 1965.

Figure 5-17 - Repeatability for Barrel Test (Initial Laboratory Work)

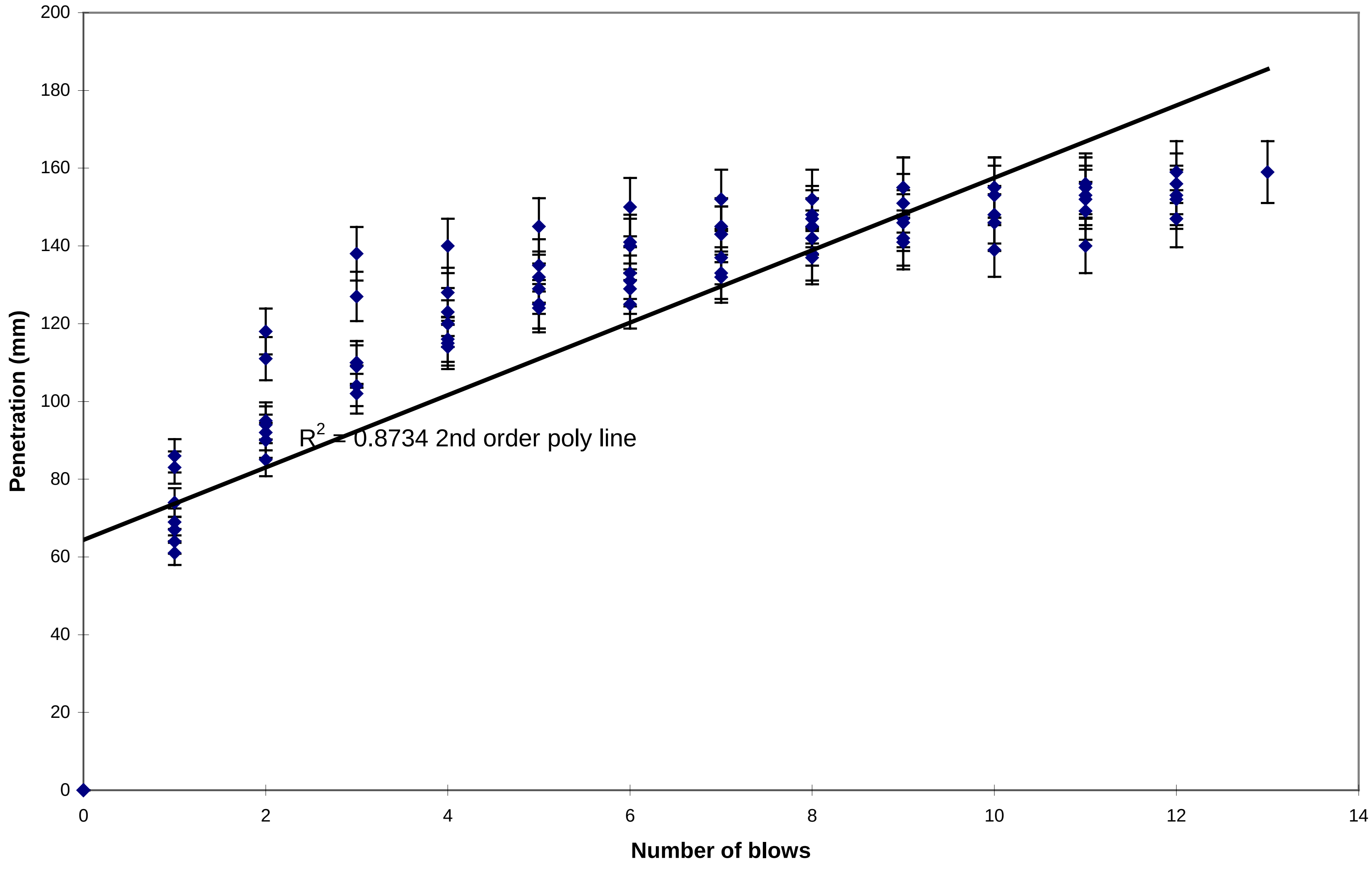


Figure 5-18 Plot of Mean of Barrel Tests with Min and Max Points to Show Repeatability (Initial Laboratory Work)

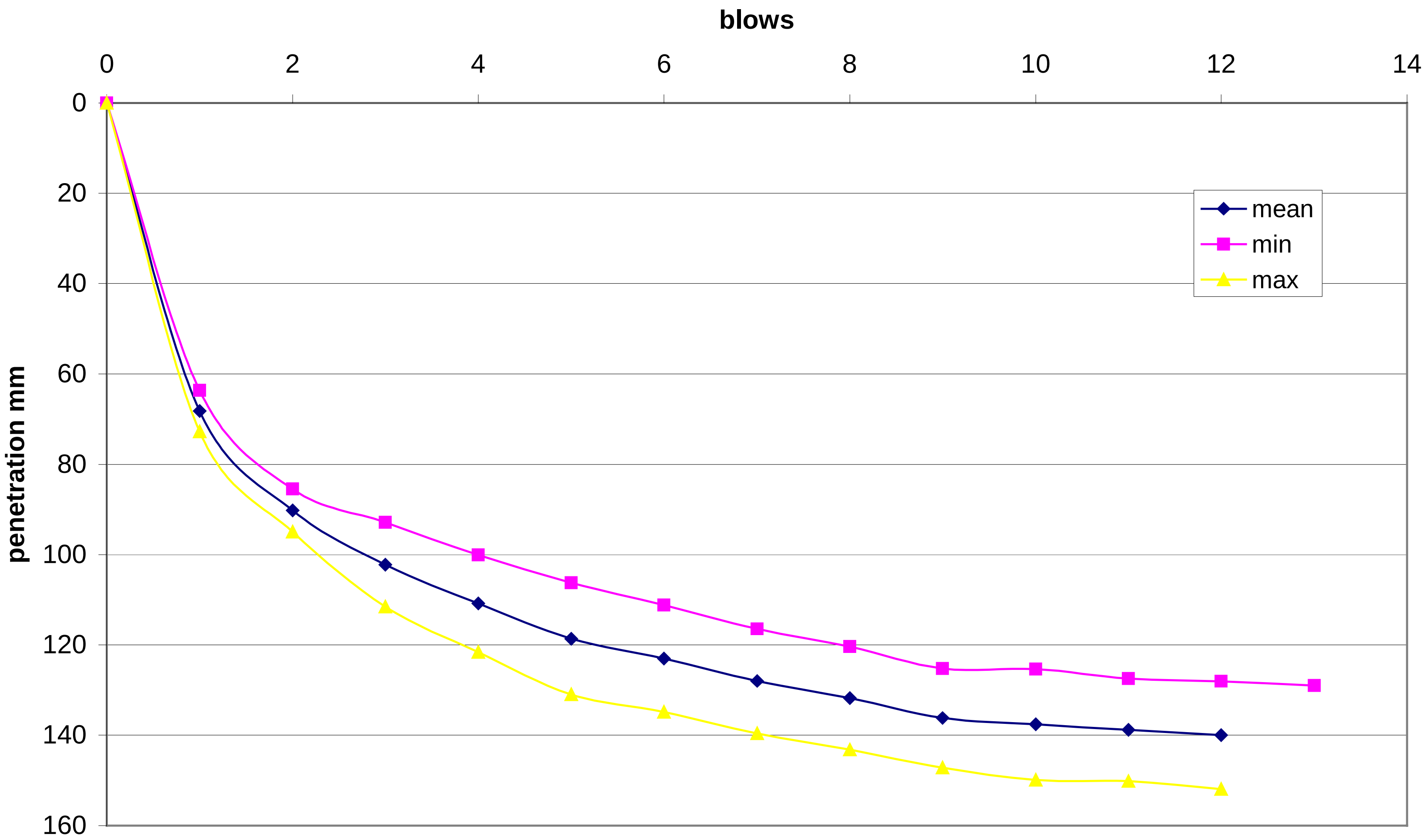
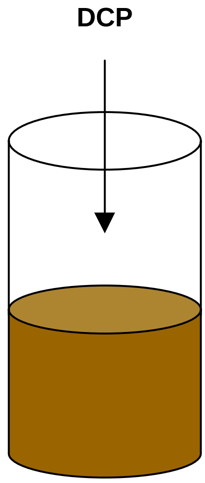


Figure 5-19 - Small Barrel Used for RMA



Barrel used = 300mm X 610mm
Soil depth = 200mm
Flat Tip used = 49mm ∅

5.7.1.1 Wet and Dry Tests

As wetter conditions may be experienced by aggregate in roads than at their source, various materials were tested under both dry and wet conditions. A pattern of 7 holes had been drilled into the bottom of the barrel of approximately 8mm diameter each to allow drainage when undertaking the wet tests). Again the test was an initial trial to assess the effect of wetting samples on the penetration in the barrel. The wet condition was achieved by adding enough water so that the excess water drained away through the holes, thus creating a standard 'suction' condition. The percentage fall in strength, relative to the dry result, is obviously a measure of how the material is affected by the environmental condition, (an issue which is particularly important in countries which have significant rainfall). The tests undertaken on a variety of UK materials revealed that moisture in the material would indeed be an issue for investigation overseas. The materials tested in a wet and dry condition were granite, limestone, well graded shale, and 5 – 10mm granite and 20-37.5mm granite. The results were as follows:

- The wet granite had a slightly greater initial penetration rate
- The limestone yielded a slightly greater penetration rate in a wet condition than the same material in a partially dry condition
- The well graded shale had a larger initial penetration rate in a wet condition than the same material in a partially dry condition
- The 5-10mm granite gave similar results in a wet and dry condition
- The 20-37.5mm granite gave similar results in a wet and dry condition

The results were not conclusive, the similarity between the penetration in a wet and dry condition for the 5-10mm and 20-37.5mm materials may be due to lack of finer material to fill in the gaps (as in the case of a well graded material). It was obvious that further work on overseas soils would be necessary.

5.7.1.2 Modified Usage of DCP (Drop Heights)

An initial (ad hoc) investigation was undertaken as part of the Bardon trial to assess the effect of a change in drop height, when using the DCP. At this stage, the purpose was one of general interest. Both the cone tip and small flat tip were used at 1/3 and ½ height blows together with full blows. The observation was made that results were, essentially, linear and thus reduced drop heights could be used where necessary without introducing any 'non-linearity' of results. However, as the investigation was brief, it was decided to investigate linearity further, both when using the DCP to test roads and also when using it as part of the Road Materials Assessment apparatus (RMA). Sections 6.1.3 and 6.4.4.

5.7.2 Further Barrel Tests in the Laboratory

An overseas visit to Malawi was undertaken, primarily to undertake tests with the cone and small flat tip and DCP on roads so as to assess the correlations with resilient modulus. (See 6.2). Also tests were undertaken in a large barrel (an oil drum). The reason for using a larger barrel was to assess the ease with which one could be used, as it was felt that an oil drum would be more freely available.

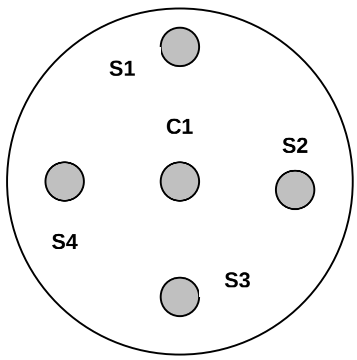
Samples were taken in Malawi from the Mwela gravel pit and sent to the project teams laboratories in the UK. The material was a laterite, pale red/ brown in colour. The material was used to undertake an initial comparison assessment between two different sized flat tips.

A further large tip (75mm) was developed to overcome the problems of excessive rates of penetration sometimes experienced in using the DCP in the barrel, (which the smaller tip (49mm) had suffered in the testing undertaken in a large barrel in Malawi). The smaller tip had yielded initial 'zero' readings which were very high, due to the tip sinking under self-weight. Thus leaving only a few blows until the tip reached the bottom of the barrel). It was therefore necessary to establish if a larger tip could overcome the problem, and if so, whether it would yield comparative results. A lid had been obtained (initially for the purpose of transporting the barrel more effectively overseas), but this proved useful in the testing. The same barrel was used, as previously (section 5.7.1). The material was emptied into a large tray to facilitate mixing to ensure a representative sample from the bag. The material was a mixture of the large partly 'cemented' pieces of laterite and hard clasts together with fine granules and (dry) clay. There appeared to be no conventional aggregate within the sample.

Five assessments in the barrel were performed, in the positions indicated in Figure 5-20. One test consisted of using the DCP with the modified tips in the positions shown, (using either full or 1/3 blows). The results for each set of tests, for example: Test 1 (S1, S2, S3, S4, C1) were plotted on the same graph and a 2nd order polynomial curve fitted to obtain a value for R^2. The purpose was to establish how repeatable the test was when the tip was placed at different positions in the barrel. If a low R^2 value had been yielded it would mean that the test was susceptible to the positioning of the tip in the barrel.

Figure 5-20 - Arrangement of Tests in Barrel

Test 1 at
C1 S1 S2 S3 S4
Test 2 at
C1 S1 S2 S3 S4
Test 3 at
C1 S1 S2 S3 S4



The plots for penetration (mm) against blows were made on a simple scatter plot for each of the tests. (The 2nd and 3rd tests were simply repeat tests). For example: Test 1, used a 49mm tip on dry material and the initial set of tests yielded an R^2 of 0.9693 using full blows, the repeat test yielded an R^2 of 0.8943 using full blows and the 2nd repeat set of tests yielded an R^2 of 0.9216 using 1/3 blows. The values of R^2 obtained for the tests are given in Table 5-8.

Table 5-8 - Barrel Tests to Assess Repeatability

Test	Tip	Wet/Dry	Initial R^2	Blows	Repeat R^2	Blows	2nd Repeat R^2	Blows
1	49mm	Dry	0.9693	Full	0.8943	Full	0.9216	1/3
2	75mm	Dry	0.8239	Full	0.5664	Full	0.8840	1/3
3	75mm	Wet	0.8483	Full	0.9922	Full	0.9509	Full
4	49mm	Wet	0.8947	1/3	0.9445	1/3	0.8532	1/3

It is apparent from the results that a good level of repeatability was achieved in all cases. However a lower value resulted from Test 2 (Dry test using the Large Tip), but the reason for this poorer repeatability is unknown. The readings for the first and last test positions in the barrel (S1 and C1) do not appear to follow the same trend as the other results for that test (S2, S3, S4). Less than 10% of the assessment are so affected so this is not viewed with great concern.

In the dry tests there seemed little advantage in using the large tip as opposed to the small for the type of material tested. Due to the large size of the 75mm tip in comparison to the barrel it would probably be wise to undertake less tests in each set, with perhaps central tests being sufficient. With regard to the wet tests, the small tip sank rather quickly and it was necessary to use 1/3 blows for all the sets (the issue of drop heights is covered in section 6.4.4, but here three 1/3 blows are assumed to cause the same penetration as one full height blow). The large tip tended to disturb the soil around it, when it was removed from the barrel, and therefore it was necessary to undertake fewer tests, S1, S2 and S3 only. The R^2 values for the large and small feet were, however, quite similar. Having undertaken testing of the 49mm and 75mm diameter feet in the small barrel, it was then necessary to see which barrel (this, or a full-size barrel) gave the best results. This was undertaken overseas in Uganda, (See 6.3), where both a large and small barrel and a large and small tip were used.

On the basis of the work undertaken in the laboratory, the preliminary findings of the barrel test, (using a small barrel and two different sized feet) were as shown in Table 5-9.

Table 5-9 – Preliminary Advantages/ Disadvantages of Different Sized Feet

Advantage	Disadvantage
The 49mm tip was of a more reasonable size, compared to that of the barrel.	The 49mm tip sunk rather quickly when testing a wet sample
The 75mm tip did not sink as quickly when testing a wet sample.	The 75mm tip seemed large in comparison to the size of the barrel and therefore the number of tests which could be fitted into the area was less, however if the tip gave repeatable results with only one central test, it was not a problem.
	When testing a wet sample, there was significant disturbance as the 75mm tip was removed, reducing further the number of tests, which could be achieved in one barrel filling to three. (Probably material dependent).
	The 75mm tip appeared to compact the material. If an aggregate had been used, it is difficult to decide whether the tip would break up the aggregate or whether it would push it sideways.

5.8 Findings of the UK Laboratory and Site Work

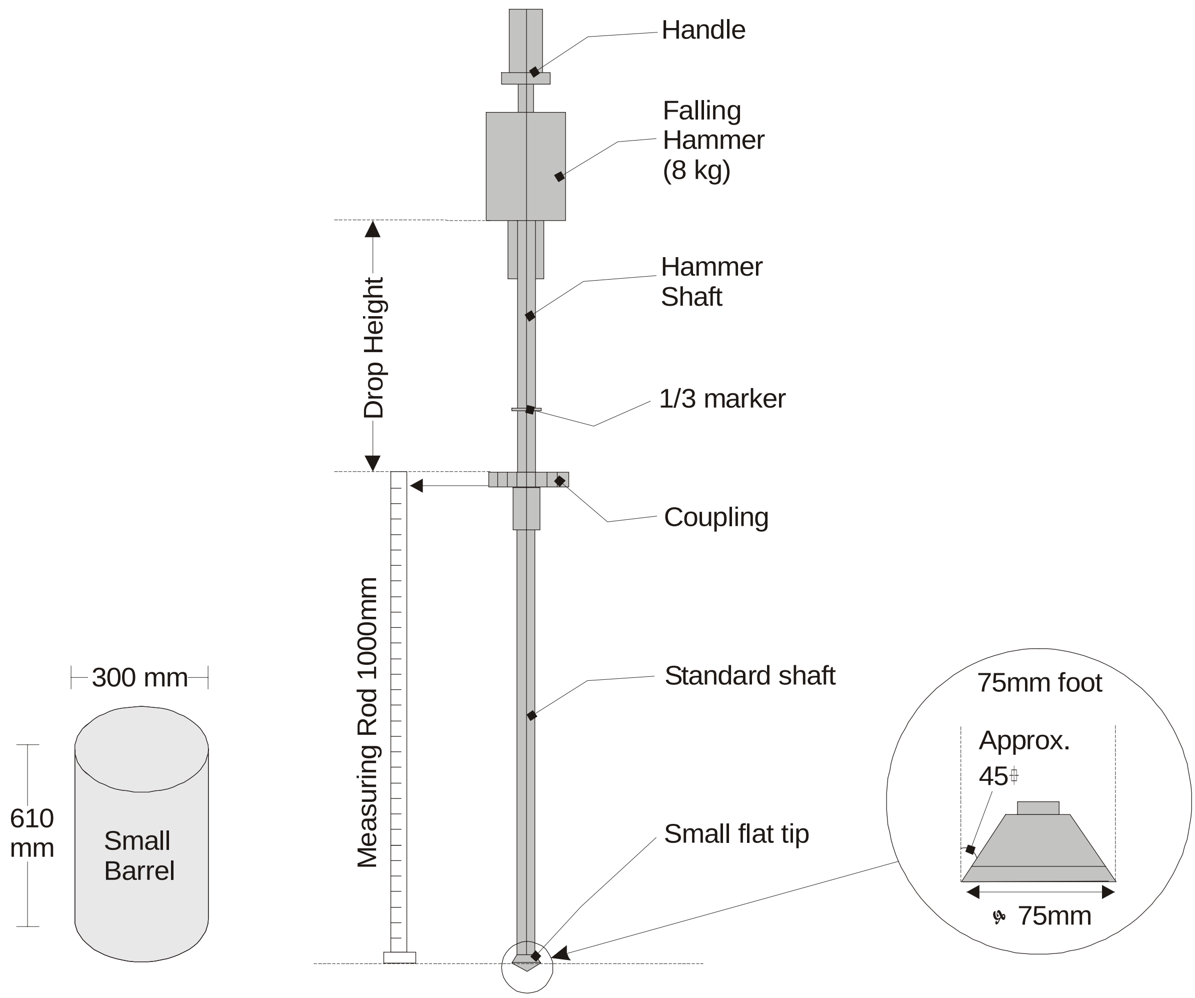
The experimental work as discussed in the preceding sections allowed the work to be developed to a stage where it was ready to be tested fully overseas. The findings may be summarised as follows:

- There was a good agreement between the CBR/ DCP results and other researchers work using the standard cone tip.
- Due to the different behaviour of sand, it was decided to concentrate on clay and gravel materials.
- Modified DCP tips were manufactured and assessed.
- The Bardon trial allowed an interesting relationship between the DCP small flat tip and the results from TFT and GDP resilient modulus measures to be developed, for the materials assessed at that site.
- The tip size and shape giving the most valuable results with regard to the resilient modulus was determined to be the small flat tip and, therefore, it was decided not to test the dome tip any further.
- Flat tipped DCP results ranked aggregate similarly to observed rutting under trafficking was an aggregate only phenomenon
- A possible linear relationship was found between the DCP and blows of different height (1/3, ½, 1), when testing in the barrel
- The repeatability of the barrel test results was reasonably high for the materials tested.
- The barrel size and tip size combination required further investigation, as did the issue of moisture content and its effect on the results

6. TRIAL IMPLEMENTATION ON OVERSEAS SITES

The trial implementation on site involved using the modified equipment in a number of developing countries. The purpose was twofold - an assessment of different types of materials at different moisture contents had to be undertaken, but also an assessment of using the equipment in locations which were far removed from the UK and thus locations which posed logistical problems. Visits were made to countries where Roughton International Ltd., could offer support on their schemes and projects. The modified DCP is shown in Figure 6-1, and a close up photo of the large flat tip in Photograph 5.

Figure 6-1 Modified DCP Apparatus for use with the Barrel



6.1 Mozambique Trial

6.1.1 Purpose of Testing

The purpose of the testing undertaken in Mozambique was to establish whether a linear relationship existed, when different blow heights were used with the DCP when used for road testing. The investigation was carried out prior to other site work. It had been envisaged that reduced drop heights could be used in later testing for barrel and road testing as the UK work had shown that the relationship between drop height and penetration was expected to be linear. Therefore the effect of drop heights on the penetration in different materials also had to be investigated.

6.1.2 Testing Undertaken

A total of 8 different materials were tested in borrow pits (not the barrel) using blows of 1/3 of a full height blow, ½ of a full height and 1 full height blow (using a standard cone tip). The materials tested were as follows:

Table 6-1 - Material Types Tested in Mozambique for Drop Height Assessment

Description Of Material	Blows Administered
Clayey well graded coarse sand	1, ½, 1/3
Slightly silty well graded to coarse sand (low plasticity material)	1, ½, 1/3
Silty sand and fine quartz gravel and cobbles	1, ½, 1/3
Clayey well graded sand	1, ½, 1/3
Slightly clayey/ sandy fine to medium gravel (laterite and quartz)	1, ½, 1/3
Clean well graded sand (non plastic)	1, ½
Clean sand over sandy gravel (quartz and laterite)	1, ½, 1/3
Clayey sandy gravel (quartz)	1, ½, 1/3

6.1.3 Drop Heights

A possible reason for adopting reduced blows in the barrel, was to reduce penetration and thus yield more readings. The results of the investigation are detailed in Table 6-2. The results showed that in general, 1/3 blows gave a greater penetration rate (measured as penetration per full blow) than ½ blows, which in turn gave greater penetration rate than full blows. A different result may have occurred had more clayey soils been used. Obviously, such a relationship would have to be taken into account if reduced blows were adopted in testing. It was subsequently found that the relation between reduced drop heights and penetration when testing in the barrel, (through the work reported in 5.7.1.2) was generally linear. Thus the link between full height and 1/3 height blows testing may be material dependent or dependent on the confining effect provided by the barrel. The work previously reported in Section 5.6 suggests that the latter is more likely and that linear penetration versus drop height is associated with barrel tests and a non-linear relationship for in-situ tests.

Table 6-2 - Results of Investigation into Drop Heights in Mozambique.

Material	No Of Full Blows And Resulting Penetration	No Of 1/2 Blows And Resulting Penetration	No Of 1/3 Blows And Resulting Penetration	Full Equivalent Mm/Full Blow	1/2 Equivalent Mm/Full Blow	1/3 Equivalent Mm/Full Blow
Clayey well graded coarse sand	102 full blows resulted in 812mm	101 (1/2) blows resulted in 560mm	101 (1/3) blows resulted in 431mm	7.96mm/blow	5.54 x 2 = 11.08mm/blow	4.27 x 3 = 12.81mm/blow
Slightly silty well graded to coarse sand (low plasticity material)	32 full blows resulted in 920mm	61 (1/2) blows resulted in 920mm	44 (1/3) blows resulted in 903mm	28.75mm/blow	15.08 x 2 = 30.16mm/blow	20.52 x 3 = 61.56mm/blow
Silty sand and fine quartz gravel and cobbles	37 full blows resulted in 775mm	31 (1/2) blows resulted in 559mm	64 (1/3) blows resulted in 666mm	20.95mm/blow	18.03 x 2 = 36.06mm/blow	10.40 x 3 = 31.20mm/blow
Clayey well graded sand	26 full blows resulted in 850mm	46 (1/2) blows resulted in 845mm	57 (1/3) blows resulted in 855mm	32.69mm/blow	18.36 x 2 = 36.72mm/blow	15.00 x 3 = 45.00mm/blow
Slightly clayey/ sandy fine to medium gravel (laterite and quartz)	41 full blows resulted in 800mm	47 (1/2) blows resulted in 812mm	68 (1/3) blows resulted in 897mm	19.51mm/blow	17.28 x 2 = 34.56mm/blow	13.19 x 3 = 39.57mm/blow
Clean well graded sand (non plastic)	38 full blows resulted in 452mm	120 (1/2) blows resulted in 925mm	Not assessed	11.89mm/blow	7.71 x 2 = 15.42mm/blow	Not assessed
Clean sand over sandy gravel (quartz and laterite)	45 full blows resulted in 550mm	67 (1/2) blows resulted in 625mm	88 (1/3) blows resulted in 541mm	12.22mm/blow	9.32 x 2 = 18.64mm/blow	6.148 x 3 = 18.44mm/blow
Clayey sandy gravel (quartz)	13 full blows resulted in 285mm	11 (1/2) blows resulted in 220mm	18 (1/3) blows resulted in 307mm	21.92mm/blow	20.0 x 2 = 40.0mm/blow	25.58 x 3 = 76.74mm/blow

6.1.4 Achievements

The trial in Mozambique showed that the relationship between drop height and penetration with the DCP when used in natural ground tested was not linear. The penetration of 3 x (1/3) blows did not yield the same penetration as one full blow, and the same was true of 2 x (1/2) blows. This obviously means that the minimum blow which should be used when testing with the DCP in the pavement is one full blow, rather than part blows (assuming standard DCP values are required from the testing). It also implies that penetration is not energy related in this case, but more impact velocity related.

The earlier indication (Section 5.7.1.2) that this finding didn't apply to the assessment of the loose material in the barrel decided the project team that it could usefully undertake an investigations into the affect of drop height when using the DCP in the barrel (See 6.4.4).

6.2 Malawi Trial

During the initial trials of the modified DCP equipment, some testing was undertaken alongside a Roughton International pavement evaluation of roads in Malawi. Whilst the DFID work was primarily involved with data collection on unsurfaced roads, it was felt that the opportunity to try and link work carried out in the Bardon Trial in the UK, with overseas conditions, was a worthwhile task to undertake. The trials undertook DCP testing with a small flat tip and a standard DCP tip.

6.2.1 Purpose of Testing

The purpose of undertaking testing work in Malawi was:

- To obtain data using the modified tip (small flat) alongside the testing undertaken by Roughton International Ltd, in particular their deflection bowl tests which would provide measurements of resilient modulus. This would enable the earlier work relating the small flat tip measurements to resilient modulus (using the GDP and TFT) to be checked using overseas soils.
- To try and undertake testing of rutted and unrutted sections of road using the standard cone and the small flat tip to explore the issue of permanent deformation/ DCP relationship.
- To undertake the first set of site testing of materials in the barrel with a 49mm diameter tip in a large oil drum as part of the RMA testing.

6.2.2 Testing Undertaken by Roughton International.

As part of a rehabilitation design programme the Roughton pavement team were undertaking DCP testing on over 1100 km of road in varying states of distress and construction.

Sample lengths were identified along the project roads with, depending on the length of the road, 1 to 9 lengths selected per road, as being a typical representation of the current state of the road. The sample lengths were on average 1-2km long. In each of the sample lengths, certain tests were performed. These tests were as detailed in Table 6-3.

Table 6-3 - Test programme in Malawi

Test	Frequency	Variable Measured
Deflection Bowl (RI Method)	One per sample length	Mr
Radius of Curvature	Every 100m	Mr
Peak Deflection	Every 100m	
DCP Testing	Every 100m, + extras	In situ CBR
Trial Pit Excavation	One per sample length	Material Identification, Lab CBR, PSD

6.2.2.1 Deflection Bowls

Roughton International method of measuring traffic induced Deflection Bowl is based on a method developed by TRL (Smith et al)[120] for peak deflection and involves the use of photographs to collect a record of vehicle position and of pavement deflection (using the Benkleman Beam) at the same instant. The method collects a full bowl, so an approach Mr and rebound Mr can be obtained. In the US and Australia, the test is purely performed as a rebound test.

6.2.2.2 Radius of Curvature (ROC)

This test method was presented by Grant et al (1999), and is used to test integrity of the pavement layers. It is conducted at the same time that a peak deflection test is performed, but by using a small piece of equipment and a dial gauge. A value of less than 70m, is generally accepted as a value of a poor quality layer in the base. Relationships have been developed that permit Mr to be calculated from this test.

6.2.2.3 DCP Testing

The DCP tests are analysed by an in house computer package, that uses the TRL equations to calculate a CBR value based upon the penetration rate of the DCP. Following on from this CBR calculation, a further TRL equation links CBR to Mr values. This equation was developed in English clay soils and so can be considered quite site specific, however, the form "$aCBR^{b}$" which the equation takes has been used by other research organisations. It

[120] Smith et al, 1984, Transport Research Laboratory Report LR 935, TRL, UK.

would seem that the TRL equation is almost a "middle average" value, and for the purpose at present is considered sufficient. It will be recalled that the results in Section 1, confirmed the broad pattern of CBR:DCP relationships for the present study.

6.2.2.4 Modified DCP - Small Flat Tip

During the testing on the sample pavement lengths, tests were repeated with a modified tip. From the results obtained, it was seen that the standard cone and small flat foot produced in a number of cases, almost identical results, so no advantage could be seen in using the modified equipment in this situation. However, in the UK it had been found that the small flat tip produced a slightly better correlation with stiffness readings than a standard cone. However, it must be noted in the UK cases, differing results with standard cone and small flat tip were obtained.

6.2.2.5 Trial Pit Excavation

At the location where a bowl test was performed, a trial pit was excavated. This is used to identify the materials, the layer thickness and to obtain specimens for soil index properties testing in the laboratory. This information can then be used to assess the readings obtained from the DCP, Bowl Test, Peak Deflection and Radius of Curvature. At Trial Pit locations, three DCP tests were carried out, the standard test which was performed every 100m in the same location as the ROC measurement, plus two extra tests at either side of the pit, to establish a better idea of the DCP consistency.

6.2.2.6 Results Processing

After the tests had been performed, the results were tabulated and graphical representation presented, these show the correlations of the Resilient Moduli (Mr) from the different methods. This work was then be compared to the DCP:Mr (stiffness) relationship that was developed during the trials at Bardon in the UK, as part of the Highways Agency testing programme. This work is summarised in Report IV[121]

6.2.3 Other Testing Undertaken

With regard to the testing of rutted and unrutted sections, not as much data as was hoped for was obtained. The reason for the lack of suitable data was due to that fact the roads were seldom found to contain both rutted and unrutted sections built of the same material, on the same section of road – as had originally been envisaged. Thus sensible comparisons between rutted and unrutted sections were not normally possible. Further work was undertaken as part of the Uganda field work, See 6.3.

The first overseas tests with a barrel and a 49mm flat tip were undertaken. A large oil drum was used, as it was more freely available in Malawi than the smaller barrel which had been used in the UK and it was decided that obtaining data from a larger barrel would be useful. Data was obtained to compare with some of the testing undertaken on the road. (Unfortunately, however, the sparseness of matching laboratory data from the road test pit soil/aggregate meant that the barrel test results were not very helpful). However the work with the barrel was a very useful exercise, as using the equipment in the field made it clear that both the barrel and tip sizes needed further development and consideration in order to produce a workable test. These issues were investigated further in Uganda and during laboratory work at the University of Nottingham, See 5.7.2.

6.2.4 Achievements

6.2.4.1 Investigation by Roughton International

The work undertaken in Malawi, has shown that the DCP consistently over-reads the stiffness compared to the ROC deduced value by a factor of 6, whilst the deflection bowl method shows a closer match with an under-read factor of 0.5. The results and plots are taken further in the Report IV [121] . The modified tip may have further uses when performed in

[121] KAR 6852, 2000. Appropriate and Efficient Maintenance of Low Cost Rural Roads, Report IV, Field Trials of Modified Equipment, DFID, London

a borrow pit, or loose compaction stage. With a well compacted, laterically bound material, the results were almost identical, between the cone and the flat tipped versions.

Photograph 6-1 DCP Test in Barrel

From the consultant's experience in these countries, and the subsequent laboratory analysis, it was found that the pavement was in a very dry condition, producing artificially high DCP readings. From work in the wet and dry seasons, in various countries, a conditioning index has been developed that is applied to results and based upon laboratory testing, in situ testing and a visual assessment of the pavement. During the Malawi study a value of 6 was used. These values have ranged between 10 and 1, depending on the time of year and the country.

Photograph 6-2 Small Flat Tip on Road

6.2.4.2 General

The Malawi fieldwork was the first 'live' trial of the equipment and allowed the problems in using the barrel (such as it's size and the size of the tips) to be established and rectified for the next set of testing. The lack of laboratory data from the samples taken did cause a subsequent problem, as there were insufficient CBR results for comparison purposes. The laboratory in Malawi did not live up to expectations – it was struggling to undertake the quantity of testing required by the Roughton International Ltd project, and the laboratory staff did not use the recommended coding system for the samples, therefore many laboratory tests were not undertaken and many samples were lost. The issue did not arise until the group had left the country, by which time it was too late to solve the problem. The lesson learnt from this experience was that supervision of the laboratory testing would have to be adequate in other countries. Thankfully, the same problem did not occur in Uganda or Fiji as the laboratories were set up and run in a far more efficient manner. The trial in Malawi allowed the next trials to be set up and run more effectively.

The tips for the modified DCP, (49 mm and small flat tip) were manufactured easily and cheaply by a local engineering firm in Malawi. This was a positive step as it indicates that there would not be a problem if the equipment were ever applied in practice in a developing country. A large barrel was found easily, yet it was interesting to note the problems with transporting it (due to it's size). This suggested that the smaller barrel, as used in the UK, would be more appropriate. Obviously the issue of transportation is a potential problem noted with the equipment, as the availability of vehicles is quite scarce in many developing countries, especially at the low cost end of the market. The Malawi testing was successful as it allowed the subsequent trials to produce more useful data.

6.3 Uganda Trial

6.3.1 Purpose of Testing

The purpose of undertaking testing work in Uganda was:

- To obtain data using the small flat tip alongside the standard cone, particularly in rutted and unrutted sections to provide data for the permanent deformation/ DCP relationship. No measurements of resilient modulus were possible alongside the testing this time.
- To obtain data with typical tropical soils and aggregates.
- To undertake a study of the DCP used in different types of barrel and with different types of tips and to compare the results to tests undertaken in the road.

- To obtain test data, from the barrel, when using soils in a wet and dry condition to assess the influence of 'seasonal' factors.
- To perform repeatability testing on all barrel samples and obtain data which would aid a decision on which barrel and tip combination would be the most effective.

6.3.2 Testing Undertaken

Table 6-4 details the testing undertaken on a dry road, this involved testing using the standard cone, the small flat tip and also the barrel tests.

Table 6-4 – Programme of Work on Dry Road (Rutted and Unrutted Section)

	Cone	**Small flat**	**49mm flat**	**75mm Flat**
Road	Y single blows	Y single blows	N	N
Barrel (Small) Dig sample from the road – place in barrel.	N	N	Y 1/3 blows 2 repeats	Y 1/3 blows 2 repeats
Barrel (Large) Dig sample from the road – place in barrel	N	N	Y 1/3 blows 2 repeats	Y 1/3 blows 2 repeats

Table 6-5 details the testing undertaken on a dry road, this involved testing using the standard cone, the small flat tip and also the barrel tests.

Table 6-5 – Programme of Work on Wet Road (or Artificially Wetted Road (Rutted and Unrutted Section)

	Cone	**Small flat**	**49mm flat**	**75mm Flat**
Road	Y single blows	Y single blows	N	N
Barrel (Small) Dig sample from the road – place in barrel.	N	N	Y 1/3 blows 2 repeats	Y 1/3 blows 2 repeats
Barrel (Large) Dig sample from the road – place in barrel	N	N	Y 1/3 blows 2 repeats	Y 1/3 blows 2 repeats

Table 6-6 details the testing undertaken in the borrow pits, this involved testing using the barrel only.

Table 6-6 – Programme of Work in Borrow Pit Dry and Wet

	Cone	Small flat	49mm flat	75mm Flat
Barrel (Small)	N	N	Y 1/3 blows 2 repeats	Y 1/3 blows 2 repeats
Barrel (Large)	N	N	Y 1/3 blows 2 repeats	Y 1/3 blows 2 repeats

6.3.3 Testing of Unrutted and Rutted Sections In Uganda

Sections of road were found which displayed both rutting and little or no rutting, so that both could be tested with the small flat tip to ascertain whether the DCP could detect material differences between the two sections, as they would in other respects, be the same.

The envisaged practical use of such an investigation was to use the DCP as a device to assess gravel roads for susceptibility to rutting. It is realised that in an ideal world, such an investigation would have tested sections of road prior to any rutting and then monitored the roads for a period to see if they rutted. The results of the DCP tests could then have been analysed to ascertain whether there were any obvious differences between the results for the pavement that did not rut and the results for the pavement that did rut. However the current project was unable to undertake such an investigation and so it relied on the data from the Bardon trial and the data from Uganda to explore the issues.

The data gathered in Uganda was analysed and the results are discussed in 6.3.3.2.

6.3.3.1 Testing of Wet And Dry Surfaces

The decision to test in a wet and dry condition on the road had initially been based on the supposition that after rain, the penetration with the DCP would increase due to the material being softer. However, on closer inspection of the roads after rain it was noted that it was simply the top (approximately 100mm) of the surface which was 'wet' whilst the material beneath remained dry, this was observed in area with good drainage and appropriate crossfall. (Admittedly, this was after relatively short periods of heavy rain of approximately 12 hours). The materials noted were generally clayey gravels, with some silt and sand. The wetting of the surface caused the material to rut quickly under the effect of trucks (as shown Photograph 6-3 Rutted Surface - Uganda). The heavy vehicles caused the surface material to become very weak and soft, permitting the surface to become deeply rutted. It had been intended to try and imitate this effect, if the rains did not occur. In countries with a pronounced wet and dry season, deeper wetting might be expected and a different type of rutting characteristic could be of concern. Such a scenario was not assessable in Uganda.

Photograph 6-3 Rutted Surface - Uganda

After the initial tests were undertaken the weather changed and it did not rain again. Therefore, as an experiment, the road was wetted by hand to try and imitate the effect of rain. It was soon realised that such a method would not work, as the material was too hard and dry, the water simply ran off and would not soak in. A significant amount of water would have to be added and the action of vehicles would have to be used to create the same situation as occurred in the rain. As a water supply was not available, it was decided not to pursue testing the road in a wet condition further. The material tested in the barrel was however wetted and moisture content samples were taken, so a measure of the effect of moisture could be explored.

6.3.3.2 Outcome of the Road Testing

Tests undertaken using the DCP with a small flat tip at the Bardon rutting trial had shown that two possible mechanisms of rutting seemed to exist. Plots were therefore made of the penetration rate in mm against the number of blows for the rutted and unrutted Uganda sections. Data was also available for the same sections using the 75mm tip in the small barrel on material dug out from the sections. A summary of the useful results is given in Table 6-7

Unfortunately the plots were inconclusive. Four sections had been tested and in 3 out of the 4 cases, the penetration rate yielded by the rutted sections was lower than that of the unrutted sections. This may be due to the fact that the rutted sections tested were more compacted and thus yielded a slower penetration. In the case of the sample which displayed a rutted section with a greater penetration rate, this may have been due to an isolated case of loosening of the material, possibly as a consequence of rutting. It is realised that such an investigation would have been more beneficial if the modified DCP testing had been carried out on a freshly constructed section of the road, which was subsequently trafficked. However the time and resources for such an investigation were not available. To ensure that the results obtained were representative, the tests were conducted adjacent to the wheelpaths, not in them.

With the investigation undertaken, nothing was known of the maintenance history of the sections. A significant unknown was also the fact that the sections may not have been compacted adequately in the first place and therefore the rutting seen may have been a consequence of poor compaction. The disturbance upon rutting would prevent a DCP from detecting this difference. As a result the modified DCP tests were not able to reveal very much. Tests on the same material were undertaken in the barrel with the 75mm tip, to ascertain whether there were any obvious differences between the material from a rutted and an unrutted section. The barrel test did not indicate that poor material was the cause of the rutting, as no pattern was evident from the plots made.

Rutting of the type seen in Uganda probably occurred due to the surface material softening during wet weather. This can probably be attributed to the lack of coarse material on the surface, (lack of regravelling as part of a maintenance programme). The fact that the ruts were still present when the wet weather returned meant that they held water and no doubt led to further weakening of the pavement. Poor surface drainage can also lead to the softening of the surface, and the development of ruts. Removal of the ruts by blading and the addition of material would have gone a long way to preventing the very deep ruts which were seen in Uganda. It is interesting to note how the Soaked CBR for the rutted sections tested tends to be lower than the unrutted sections tested, suggesting that the material in rutted areas may be more susceptible to softening than in unrutted sections. In such a case, DCP readings on the un-wetted material would not be useful in predicting rutting potential.

Table 6-7 - Data for Uganda Rutted and Unrutted Samples

Location	km	Rutted/ unrutted	PI	PM	Soaked CBR	Unsoaked CBR
Nebbi –Arua	41	Unrutted	21	1134	25	60
	40	Rutted	19	1007	23	65
Nebbi-Pakwach	9.0	Unrutted	Non P	-	Estimated 15	-
	8.9	Rutted	Non P	-	14	45
Pakwach-Olwiyo	3.8	Unrutted	19	608	53 and 20	74 and 59
	3.0	Rutted	20	620	9 and 6	37 and 20
Nebbi-Arua	8.4	Unrutted	25	1175	19 and 14	72 and 62
	7.4	Rutted	21	1281	5	45 and 38

6.3.4 Testing of Barrel and Flat Tip Sizes in Uganda

A considerable amount of data relating to both barrels was collected and it was therefore necessary to establish, early in the testing programme which barrel and tip combination gave the most reliable and useful results and thus concentrate on that data for future investigations.

Four sites with different types of material were chosen for the initial analysis:

- At 43.8km (Nebbi - Pakwach) Alwi Ghabi pit [reddish orange silty clayey gravel with very hard Lateritic nodules]
- At 34.0km (Nebbi - Pakwach) Pangieth pit [orangy brownish red, slightly silty slightly clayey gravel with Lateritic nodules].
- At 3.0km (Pakwach - Olwiyo) un-rutted section [red clayey silty very hard gravel (quartz and laterite)]
- At (Pakwach - Olwiyo) Uleppi pit [brownish yellowish orange slightly sandy silty clay with micaeceous fines and probably decomposing gneiss in very plastic clay].

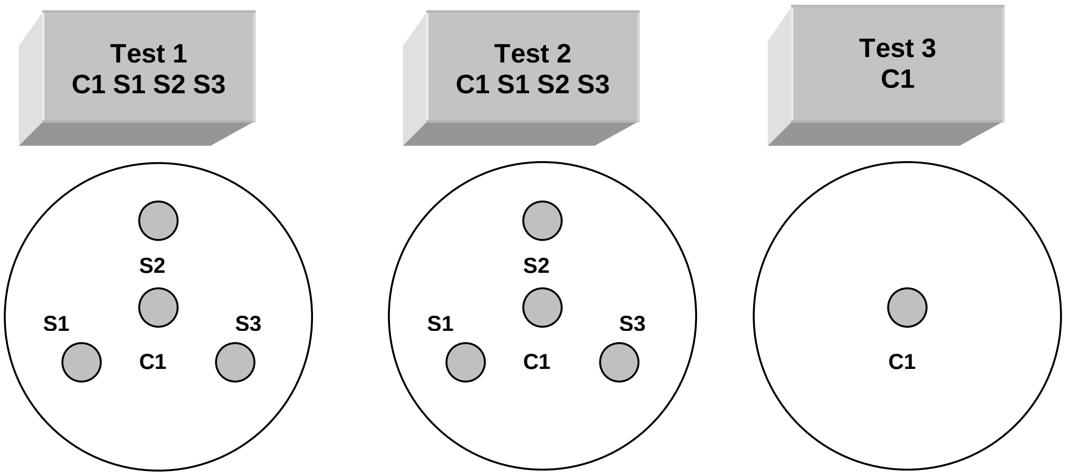
The laboratory data for the materials used in the barrel was as follows in Table 6-8:

Table 6-8 - CBR and MC Performed on Specimens taken in Uganda

	Un-Soaked CBR			Soaked CBR				
Sample location	top	bottom	mc	top	mc	bottom	mc	PI
Alwi ghabi	60%	53%	9%	45%	12%	24%	12%	25%
Pangieth	68%	58%	8%	38%	11%	23%	12%	24%
P-O 3km	37%	20%	7%	9%	10%	6%	11%	20%
Uleppi	35%	25%	11%	4%	21%	4%	20%	23%

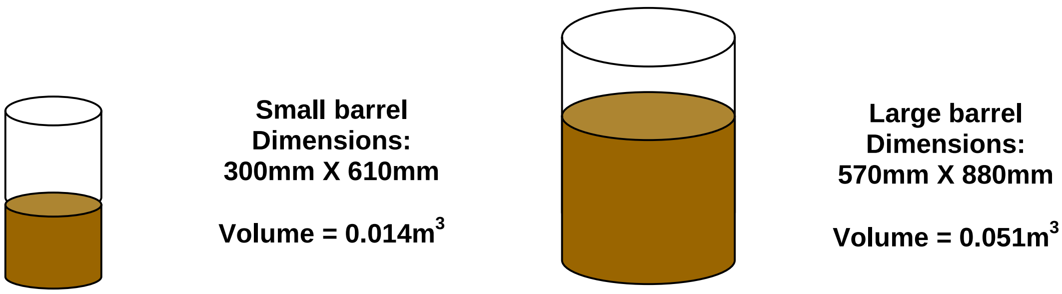
When using the barrels and flat tips, the barrels were filled with material and rolled to create the ‘standard’ loose condition. A test was then carried out in the centre of the barrel (C1) followed by three more tests around the outside of the barrel, but not too near the edge (S1, S2, S3), to see if a change in position of the tip affected the penetration results. The same procedure was then repeated, after the lid had been placed on the barrels and they had been rolled and tipped on end to loosen the material between tests. Finally a last test in the centre was carried out to finish after loosening again. Figure 6-2 shows the layout.

Figure 6-2 - Arrangement of Tests in Barrel



The small barrel previously used in the laboratory was not sufficiently large enough to allow more than one central test to be undertaken each time with the 75mm tip, and therefore the tests undertaken with the 75mm tip in the small barrel consisted of three tests with central assessments (C1) only.

Figure 6-3 - Small and Large Barrel (Sizes) used for Testing in Uganda



Comparisons were then made between the results from the small barrel with a 49mm tip and a 75mm tip and the large barrel with a 49mm tip and a 75mm tip to identify which gave the highest repeatability (as determined using an R^2 value using a 2^{nd} order polynomial equation).

When the testing was undertaken it had been hoped that eventually there would not be the need to undertake three sets of tests as detailed above, and it would only be necessary to undertake perhaps the three central tests, or even one central test. Thus keeping the procedure as simple as possible. Obviously, this would only be the case if the results obtained from the central tests [C] were repeatable and yielded comparable results with the tests undertaken at the sides of the barrel [S]. The repeatability was of concern as the supervision of the staff undertaking the test under research conditions was obviously more rigorous than what could be expected if the equipment were ever adopted in the field. Comparisons where therefore made between the graphs for the individual results, so that a decision on the choice of barrel and tip combination could be made and the repeatability could be assessed.

Photograph 6-4 DCP being used in Uganda

While further analysis of this type using all the available data (from 12 sites in Uganda) could possibly have given a slightly different outcome, it was felt that the time would be better utilised on other aspects of the analysis work. Therefore the sample 4 sites previously noted in Table 6-8 visited were chosen and analysed and a final choice of barrel and tip combinations made.

6.3.4.1 Outcome of the barrel test

The repeatability analysis considered plots of penetration in mm against blows for the [S1, S2, S3] and [C] positions in the barrel for the three tests, using both wet and dry samples. A 2nd order polynomial line was used to establish the value of R^2. The procedure was undertaken for four samples in Uganda. The results may be summarised as follows:

- When considering the wet tests in the small barrel, the 75mm tip gave higher R^2 values than with the 49mm tip in 3 out of the 4 cases looked at.
- When considering the dry tests the small barrel with the 75mm tip gave higher R^2 values than the 49mm tip in 3 out of the 4 cases, yet some of the R^2 values were rather low, and the reason for this is unknown.
- When considering the wet tests in the large barrel, the 75mm tip gave higher R^2 values than with the 49mm tip in 2 out of the 4 cases.
- When considering the dry tests in the large barrel the 75mm tip gave higher R^2 values than the 49mm tip in 1 out of 4 cases.
- In general most of the R^2 values were above 0.8, with many being above 0.9

After assessing the barrel tests using different tips, it was decided to consider only the central [C] results for the 75mm tip in the small barrel as there tended to be a higher value of R^2 for this tip than with the 49mm tip in the small barrel. The results with the small barrel were more consistent than those yielded by the larger barrel and so this combination was adopted for further testing and analysis, (it is probable that the small barrel gave more consistent results due to increased confinement over the larger barrel). A further consideration was the fact that the small barrel was easier to use in the field, due to its size, i.e. ease of transport, to carry, to fill, and to test in.

Photograph 6-5 The 75mm Tip was Chosen for further Testing and Analysis

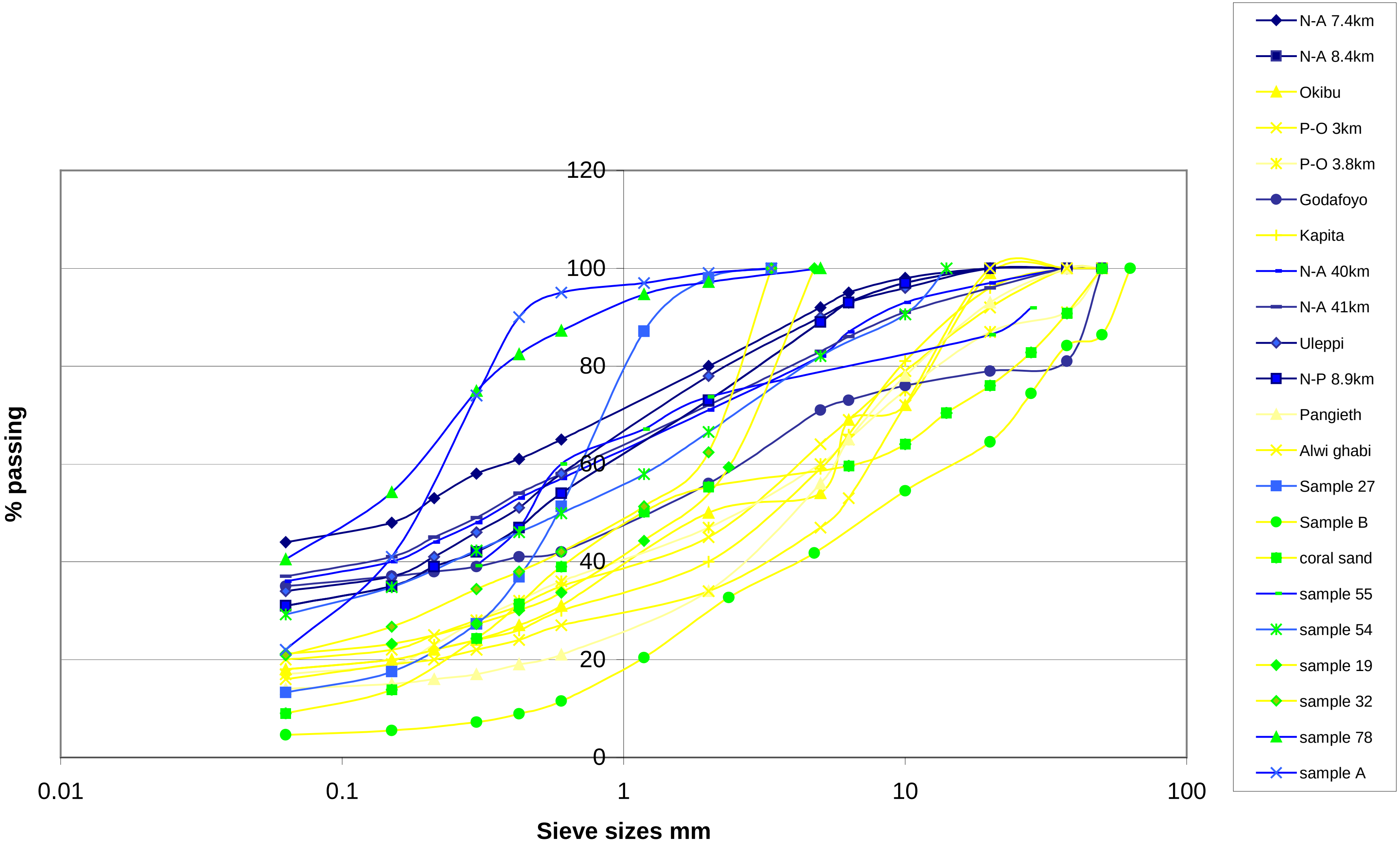
Once it had been decided on which barrel and tip combinations to concentrate, further investigations were undertaken to try and establish a correlation between the results from the barrel test and other laboratory tests. Characteristics such as CBR, Plasticity and Particle Size Distribution (PSD) were considered, to establish whether the barrel test could predict the quality of a material. The plasticity characteristics of the materials are given in Table 6-9 overleaf.

Table 6-9 – Plasticity Characteristics of the Materials

Location	LS	LL	PL	PI	PM
Godafoyo pit	9	46	20	26	1066
Uleppi pit	11	37	14	23	1173
N-A 8.4km	10	42	17	25	1175
N-A 41km	11	35	14	21	1134
N - A 7.4km	12	39	18	21	1281
N-A 40km	11	35	16	19	1007
Okibu pit	8	43	21	22	594
Kapita pit	10	43	16	27	702
Pangieth pit	10	40	16	24	456
Alwi ghabi	10	43	18	25	600
P - O 3km	9	31	11	20	620
P - O 3.8km	11	30	11	19	608

It is apparent from the characteristics of the materials that they fall, approximately, into two groups. One type was a material having a greater percentage passing the smaller sieves and thus a greater plastic modulus. The other type had a lower plastic modulus as a result of a lower percentage passing the smaller sieves. However, in most cases the PM was rather high. The PSD for the materials from Uganda, are plotted together with those from Fiji

Figure 6-4 - Particle Size Distribution for all Samples Split into Material with Higher PM (Blue) and Material with Lower PM (Yellow)



Various correlations were attempted, the results being summarised in Table 6-10. Clearly there are many unaccaptably low R^2 values and many of the better ones are specific to one type of material (higher or lower plasticity modulus). Other relationships were studies (e.g DCP 'v' CBR but these were even less highly correlated than as shown in Table 6-10. The relationship with plasticity modulus depends on splitting the materials into high and low groupings first, thus undermining the prediction ability of the DCP for this quantity. The only reasonable relationship detected was that between DCP and change in CBR upon soaking. Unfortunately there seems no good phenomenonological explanation to underwrite such a correlation, so this must be viewed with some scepticism. Of course the predictive ability of the DCP is certain to be somewhat hampered in that it is being performed on loose material at the pit. A realistic target is thus to see if it can offer a go/no-go categorisation of the pit material for use in the pavement. This goal is considered further in Section 6.4. It may be that this target is only attainable with a certain material type – i.e. some experience or more fundamental test must be available first and then the modified DCP in a barrel at the pit on loose material can assess that particular source against the existing knowledge database.

Table 6-10 - Correlation for Graphs Plotted

Graph Plotted	Value Of R^2 Obtained	
	Uganda Material With Higher PM	Uganda Material with lower PM
DCP 'V' PM	0.8146	0.9968
Change in top CBR 'V' DCP top	0.6204	0.6353
Change in bottom CBR 'V' DCP bottom	0.4241	0.2622
Unsoaked CBR 'V' change in CBR top	0.5639	0.2927
Unsoaked CBR 'V' change in CBR bottom	0.8513	0.2981
Unsoaked 'V' soaked CBR top	0.2476	0.9023
Unsoaked 'V' soaked CBR bottom	0.8712	0.4905
Unsoaked 'V' soaked CBR ave of top/bottom	0.7961	0.7142

Where top indicates the top of the CBR sample and bottom the base of the sample.

There was always some concern over using the average mm/blow from the barrel test as a basis of correlation with other tests. This was due to the fact that the mm/blow obtained reduced as the tip penetrated the material. This was partially due to a compacting effect and partly to end effects from the barrel.

6.3.5 Achievements

The testing undertaken in Uganda was successful for two reasons: the laboratory facility was well organised, and the testing programme set up had benefited from the lessons learnt in Malawi. The main problem with the data obtained in Uganda was that it lacked any resilient modulus measurements for comparison purposes, as it had not been possible to measure resilient modulus at the sites tested with the small flat tip. The data obtained for testing undertaken in the barrel allowed further refinements to be made to the test and the small barrel and large tip combination were chosen for further testing. However, it should be noted that the data for the large barrel test is available should further investigations be undertaken. The barrel data provided interesting results, which were analysed further in conjunction with the Fiji data. (see Section 6.4)

As in Malawi, it was possible to get new tips made, and a large barrel was easily found. It also appeared that it would be possible to get a small barrel made. Local labour was available and once training had been undertaken, there was no apparent problem in using the equipment. However, the smaller barrel was much easier and quicker to use than the larger barrel.

The testing of unrutted and rutted sections allowed the research to focus on whether or not there was a difference in the penetration rate between the sections. Unfortunately, no obvious difference in the results from a rutted and an unrutted section of road was found. It would probably have been more effective to test an un-trafficked section with the DCP and then allow the road to be trafficked to see how the rutting progressed, and thus whether or not the DCP could indicate any differences based on the materials.

6.4 Fiji Trial

Roughton International was undertaking sampling of road building as part of the present project for the Fiji Road Upgrading Project III (FRUP III) which allowed use to be made of materials and laboratory facilities in that country. This allowed a greater variety of materials including those with different characteristics to those found in the African countries, could be tested. It was also intended to ascertain whether or not any particular problems existed with the equipment due to the different climate. (The western side of Fiji is much wetter than the countries visited in Africa). It was intended to assess whether the natural moisture content of the materials to be assessed would affect the ease with which the barrel test could be undertaken and analysed. Therefore a selection of the materials gathered and delivered to the laboratory were tested in the barrel.

6.4.1 Purpose of Testing

- *Barrel tests* - investigation of the barrel test using potential road materials
- *Repeatability testing* - an investigation to assess the repeatability of the barrel test
- *Drop heights* - an investigation of the effect of drop height in the barrel.
- *Moisture content effect* - to see how the modified DCP penetrated at different moisture contents in different materials.

6.4.2 Barrel Test

Testing of different types of materials in the barrel at various moisture contents was undertaken to ascertain how the barrel test was affected by moisture, and whether any patterns were apparent. The testing which had been undertaken in Uganda, yielded moisture contents which were generally below 15% for the dry condition. However in Fiji, a far wetter country than those previously assessed the as-dug ‘dry’ condition was far wetter and varied considerably. A total of 9 samples were tested in the barrel (as detailed in Table 6-11) at four different moisture contents for each sample. The results show that the materials are very moisture susceptible, as expected, and therefore this affects the penetration recorded. Wetting of the samples took place by adding a measured quantity of water and mixing on a tray. Moisture content samples were taken. In the case of samples which already appeared to be very wet, the samples were dried in the sun to provide a range of moisture contents. Although the purpose of wetting and drying the materials was to obtain a wide range of moisture contents, a wide range was not always achieved.

The results obtained for the very clayey materials may have been affected by ‘lumps’ which may not have been at an even moisture content (for example wet on the outside but dry on the inside of the material). However attention was paid to mixing to ensure this was not a major problem.

Table 6-11 - Materials Tested in Fiji

Sample No.	Material type
Sample A	Grey and orange silty sandy clay
Sample B	Grey, river gravel (sandy and clayey with some cobbles)
Sample 19	Grey soapstone (crushed)
Sample 27	Red, clay
Sample 54	Grey clayey soapstone with weathered rock
Sample 55	Red clay
Sample 32	Orange clay and soapstone
Sample CS	Grey coral sand
Sample 78	Grey sandy silty clay

Anecdotal evidence from the laboratory staff, revealed that all of the materials tested in the Fiji barrel test, could feasibly be used in the road, either as selected fill or sub base. The laboratory staff pointed out that the river material would be used as it was found (without crushing to produce angular material) as a selected fill or sub base, or in a crushed condition as a base course.

Laboratory testing undertaken on each sample included a grading, Atterburg Limits and soaked and unsoaked CBR testing at 98% compaction. The results from each of the samples at the 'as dug' moisture content were firstly plotted in a similar way to the Uganda data, and then eventually the data was put together with the Uganda data to establish whether any general correlations existed. In the Uganda testing, the Particle Size Distribution (PSD) had been used to group together two types of materials which had a similar grading, and these groups were then used as a basis for further plots. The Fiji samples were therefore treated in the same way and the PSD was compared to the two material types already established. The samples were classified as either a material with a higher PM or as material with a lower PM as shown in Table 6-12.

Table 6-12 - Split of Material Types in Fiji.

	MC As Dug	OMC Result	Soaked (Top)CBR (98%)	Soaked (Bot)CBR (98%)	DCP Barrel Mm/Blow	Unsoak (Top)CBR (98%)	Unsoak (Bot)CBR (98%)	Liquid Limit	Plastic Limit	PI	PM
Materials with lower PM											
B	16.4%	5.80%	72	62	17.22	111	112	34	16	18	160.2
19	36%	27.60%	8	17	23.24	55	65	74	43.4	31	933.1
CS	32.2%	14.40%	54	65	26.28	88	75	45	22	23	722.2
32	80.2%	25%			30.67	74	70	75	50.1	24.9	946.2
Materials with higher PM											
55	54%	33%	15	8	25.73	18	16	95	53	42	2818.2
54	7.9%	21%	9	15	12.66	104	110	70	31	39	1794
78	50.2%	29.6%	8	12	21.29	52	50	61	39	22	1812.8
27	46.8%	25.60%	10	11	26.025	15	17	115	59	56	2066.4
A	40.5%	17%	10	6	38.81	57	50	70	36	34	3060

Photograph 6-6 Material Being Broken up to Pass a 20mm Sieve

When testing the “B” material, the large cobbles and pebbles had to be removed (as they would for a CBR test). It was therefore necessary to ensure that the material tested in the barrel passed a 20mm sieve. The soapstone materials were crushed by hand to pass a 20mm sieve, before being tested in the barrel. It was felt this method was appropriate as it would give a fairer comparison to the CBR test, but would also reduce the effect of large pieces of aggregate affecting the penetration of the tip in the barrel.

As with the Uganda data, plots were made between various parameters the resulting correlations are summarised in Table 6-13.

Table 6-13– Correlations Obtained in Fiji Testing

Graph plotted	Value of R^2 obtained		
	All Fiji samples	Fiji Material with higher PM	Fiji material with lower PM
DCP 'v' PM	0.2886	0.7076	0.8481
Change in CBR top 'v' DCP	0.5174	0.9306	0.5996
Change in CBR bottom 'v' DCP	0.739	0.9707	0.3343
Unsoak CBR 'v' change in CBR top	0.6247	0.997	0.2746
Unsoak CBR 'v' change in CBR bottom	0.5663	0.9956	0.789
Unsoaked 'v' soaked CBR top	0.3417	0.3075	0.8864
Unsoaked 'v' soaked CBR bottom	0.3421	0.3975	0.4521
Unsoaked 'v' soaked CBR ave of top/bottom	0.357	0.0342	0.7624

Summarising this data:

- None of the relationships are highly correlated for all materials
- The modified (flat-tipped) DCP penetration rates relate better to the CBR results from the less plastic materials than they do for the more plastic materials

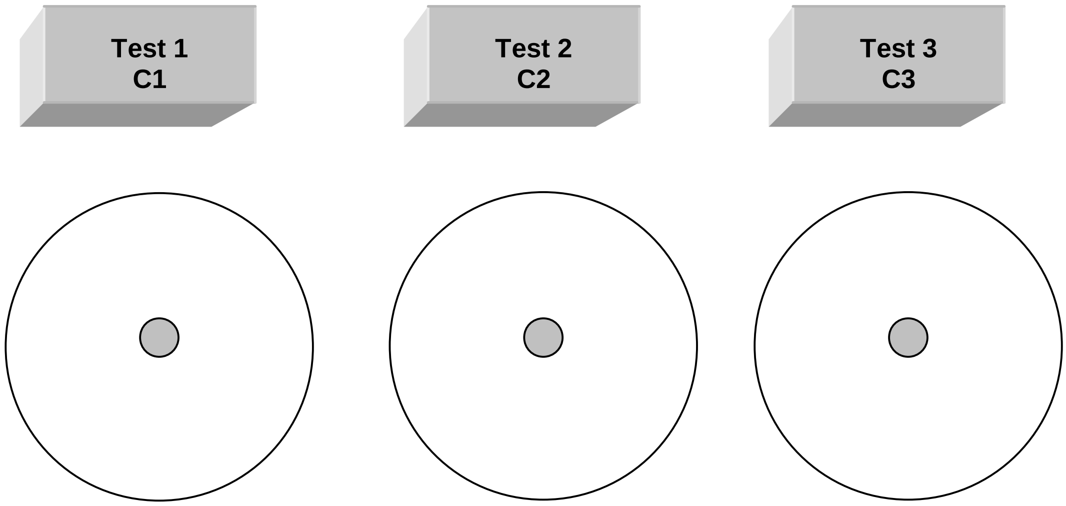
Once again, the need to split the materials into more and less plastic “families”, prior to deducing properties from the DCP results, hampers the free applicability of the DCP as a predictive tool.

Although the results were interesting, there was concern raised over using the average mm/blow from the barrel test and therefore an alternative way of dealing with the data was explored. The original aim of the barrel test was revisited and the Uganda/ Fiji data was examined together to try and develop a chart to be used to predict whether materials would be suitable for using in rural roads. The alternative method of treating the data was found to yield more consistent results and this is discussed in 7.4.

6.4.3 Repeatability Test

The testing undertaken in the barrel adopted the same methods as used in the other trials, in that a number of tests were undertaken on the same material to ascertain whether the results were repeatable. In this case an initial rolling of the barrel, to create the standard loose condition was followed by a test undertaken in the centre of the barrel [C1]. This was followed by the barrel being rolled with it’s lid on, turned upside down and having the bottom struck to ensure no material was ‘stuck’ on the bottom and then turned up the right way and retested with another central test [C2]. This variation was developed in the light of the rather more cohesive nature of some of the wetter materials encountered in Fiji. This process was repeated until three sets of readings [C1, C2, C3] were obtained. Refer to Figure 6-5.

Figure 6-5 - Arrangement of Tests in Barrel



The results were analysed from each of the nine samples, at 4 different moisture contents (MC1, MC2, MC3, MC4) using the [C1, C2, C3] readings (blows ‘v’ cumulative penetration). The R^2 values based on a linear trendline were then calculated and are summarised in Table 6-14. It can be seen that a reasonably high repeatability existed for the samples tested. However, this is partially to be expected due to the small number of data points for each test.

Table 6-14 - Correlations Based on Linear and Polynomial Lines for Fiji Samples

	R^2			
Sample	**MC 1**	**MC2**	**MC3**	**MC4**
Coral Sand	0.78	0.77	0.6682	0.7554
32	0.8032	0.7718	0.8956	0.744
19	0.6351	0.8335	0.8257	0.8352
27	0.7766	0.6898	0.8275	0.8478
54	0.8022	0.8822	0.8776	0.9129
55	0.6812	0.717	0.784	0.5965
78	0.7414	0.7532	0.5319	0.7738
A	0.8604	0.7951	0.7974	0.818
B	0.8118	0.8324	0.8635	0.8033

6.4.4 Drop Heights

Testing was undertaken to check the effect on the results of using different drop heights in the barrel. Early on in the project, it had been decided that a 1/3 drop height was most appropriate in the barrel due to the shallow depth of material in the barrel. All samples were checked using 1/3, ½ and full blows and the average penetration (mm/blows) yielded for the different moisture contents were compared and plotted. The penetration yielded by 1/3 blows was compared to the penetration yielded by full blows. It was found that 3 x 1/3 blows gave similar penetration to full blows. The average for the C1, C2, C3 results were compared and then the difference was noted. In most cases, a satisfactory linear relationship existed between different height blows and the penetration achieved in the barrel, although there were slight differences which were probably due to material type and the moisture content. The factors relating the penetration rates (achieved under the different drop heights) to each other generally ranged between 0.9 and 1.1.

It was decided that as it was intended to only use 1/3 blows in the barrel and not undertake direct comparisons with full blows as part of the test, it would be acceptable to consider only 1/3 blows as part of the barrel test. The supporting data is included in Appendix I.

6.4.5 Moisture Content Investigation

Each sample from Fiji was tested at a variety of moisture contents, the purpose of the investigation being to ascertain the effect of different moisture contents on the results obtained in the barrel – (the penetration).

Some of the samples had a very high 'as dug' moisture content and therefore these samples were dried in the sun, rather than wetted, to achieve a range of moisture contents. It was decided that only the samples tested in an 'as dug' condition would be used for comparison with the samples tested in Uganda. Some of the materials sampled in Fiji appeared to be 'dry' and yet when the moisture content test was undertaken, the moisture content was quite high. If the barrel test was only to be used on truly dry material, it would be necessary for the user of the equipment to be able to dry out the material and undertake a moisture content test to ensure it was in the correct range. Unfortunately this defeats the object of having a simple test, requiring no laboratory testing. Of further interest is the fact that in tropical countries, materials used for the construction and maintenance of roads often have to be used in a wetter condition than optimum. Thus compaction at the natural moisture content (as dug) has to be undertaken, as there are simply not the facilities to dry out large quantities of material, to a more suitable moisture content. Thus in such situations, the barrel test would have to be used on materials regardless of their moisture contents and it would have to give reasonable results. Therefore, using only the penetration rate results obtained in the barrel at 'as dug' moisture contents, seemed reasonable.

Photograph 6-7 Water being mixed into Sample for testing in Small Barrel

The C1, C2, C3 results for each of the moisture contents were plotted on a graph, so that it could be ascertained whether the penetration followed the expected pattern – that an increased moisture content would yield an increased penetration.

The pattern yielded was not entirely conclusive, as shown in the Table 6-15.

Table 6-15 - Results of Assessment of the Effect of Moisture Content on the Barrel Test

Sample And Description	Result
27 – Red clay	Higher MC = Greater penetration rate
B – Grey river gravel (sandy and clayey with some cobbles)	The graph yielded no obvious patternl
A - Grey and orange silty sandy clayey material	Higher MC = Greater penetration rate
19 – Grey soapstone	Higher MC = Greater penetration rate
54 – Grey clayey soapstone with weathered rock	Higher MC = Lower penetration rate
55 – Red clay	Higher MC = Lower penetration rate
78 - Grey sandy silty clay	Higher MC = Greater penetration rate
Grey coral sand	Higher MC = Greater penetration rate
32 – Orange clay and soapstone	Higher MC = Greater penetration rate

It may be seen that 6 out of 9 samples show a general pattern of an increased penetration with increasing moisture content. No reasonable explanation has been found to explain why the same pattern did not exist for all samples. However, what the results clearly show is that the moisture content has a significant effect on the penetration in the barrel and therefore this poses a significant problem to the work. In dry countries where the 'as dug' moisture content tends to be low, soils are tested in this "dry" condition, however in a wet tropical countries the 'as dug' moisture content can be very wet - this affects greatly the results. Despite this fact, the penetration results obtained from samples with a high 'as dug' moisture content were plotted with the materials from Uganda, to see if a relationship existed. This is discussed in 7.4.1.

In conclusion, the investigation into the effect of moisture content on the barrel results revealed that moisture did have a significant effect. The expected effect of moisture on the penetration means that any interpretation procedure for the barrel test will need to either account for moisture or not be susceptible to it. The probable explanation of the opposing changes in the penetration rate is that the moisture content is either above or below the optimum value. Where the material is on the dry side of optimum some additional moisture allows a stronger stiffer material to develop, meaning that a lower penetration rate is obtained. Beyond the optimum moisture value the reverse is true. This interpretation allows the modified DCP to be used to ascertain whether the optimum performance will be obtained by the tested material at a higher or lower moisture content that the current once yet without numerically determining what the moisture content value may be.

6.4.6 Achievements

The Fiji trial allowed a more detailed investigation using the barrel to be undertaken and it was found that:

1). In general the equipment showed a level of repeatability which was acceptable or better.

2). The relationship between drop height and penetration was generally linear indicating that it is the energy of the loading which controls the response of loose materials.

3). The moisture content had a significant effect on the penetration. It was therefore felt that further work would be needed in the future and any recommendations which were made with regard to the use of the equipment at this stage would relate only to the type of material tested and its moisture content.

4). The exercise to correlate the results from the barrel test to other standard tests was not conclusive although less plastic materials gave CBR values which were broadly predictable from the DCP. The large range of material types (with regard to PSD) and also the range of natural moisture contents limits applicability

5) More importantly, the average penetration rate had been used although the actual rate reduced at the application of each blow. Therefore the average was not necessarily representative of the whole test undertaken. For this reason an alternative method for analysing the data was adopted. See Section 7.4

6.5 Nepal Trial

6.5.1 Purpose of Testing

It was hoped to take advantage of a researcher going to Nepal, to undertake work on Report II of this Project (Stabilisation). While there he would also undertake some testing as part of the modified DCP trial. However, in the event, problems external to the project resulted in data from the barrel test not being obtained.

The researcher was able to get the required modified tips made in the country, although difficulties were experienced with trying to obtain a small barrel. Problems with the 75mm tip penetrating in the barrel were noted, due to the size of aggregate tested and similar problems were noted when trying to test road sections with the small flat tip. While Nepal represented an ideal country for application of the modified DCP as a simplified testing apparatus, reliability of laboratory and other supportive technical work was not of sufficient quality for the development of the new apparatus.

7. DISCUSSION

7.1 Choices Considered for Modifications to the DCP

As already discussed in previous sections, various choices of tip sizes and shapes, drop heights and barrel sizes were considered. The final decision on which combination was the most appropriate is discussed separately in relation to the sections on resilient modulus (7.2), rutting (7.3) and the barrel test – Road Material Assessment (7.4).

7.2 DCP: Resilient Modulus

The relationship between the small flat tip and resilient modulus from the GDP and TFT is worthy of further investigation. The relationship has so far only been investigated using the Bardon trial and a small amount of work in Malawi and therefore further investigations would be necessary using overseas soils. It was not possible to undertake many resilient modulus trials as part of this project, due to the limited availability of the GDP and TFT. Therefore in Malawi the resilient modulus was measured using deflection bowl measurements to compare with the DCP results obtained from the small flat tip.

It was noted when looking at some of the Bardon data, that the first 8 blows with the small flat tip, gave very different average penetration rates from the blows which followed. A fairly large scatter had been noted on the penetration blows on 0-100mm penetration range blows from the Bardon trial when compared against the penetration blows in the 100-200mm penetration range. The Malawi and Uganda testing with the small flat DCP tip was investigated similarly over the same penetration ranges. It was noted that the problem was not so obvious when the Malawi and Uganda data was considered. Based on the Bardon data, it had been suggested that the first eight blows should be ignored, however as the data from Malawi and Uganda did not show the same scatter, it was decided that the issue was not significant. In general, when the blows were plotted for 100mm layers, (0-100mm 'v' 100-200mm) etc., there was an improvement in the correlation to stiffness (resilient modulus)(R^2) with depth. However, the fact that the Malawi and Uganda data have less points may be significant. For the results, refer to Table 7-1.

Table 7-1 - Summary of Relationship between Stiffness Values and the Modified DCP Test

Location	DCP tip	R^2 0-100 & 100-200	R^2 100-200 & 200-300	R^2 200-300 & 300-400	R^2 300-400 & 400-500
Bardon Aggregate	Small flat	0.439	0.2923	0.4222	
Bardon Aggregate	Cone	0.0002	0.0716	0.5633	0.3573
Bardon subgrade	Small flat	0.0607	0.4376	0.6555	0.7711
Uganda	Small flat	0.8094	0.8512	0.7839	0.9651
Malawi	Small flat	0.5186	0.7548	0.5992	0.5954

From these studies the optimum relationship is

E = 1.2591 x DCP + 0.9961 for a linear fit, but where an equation of

$E = 1.096 \times DCP^{1.05532}$ is used, the R^2 value drops to 0.852, which is still comfortably the best correlation.

Where, DCP is the penetration rate in mm/blow over the distance 300mm – 500mm

7.3 DCP: Rutting

The data from the Bardon trial indicated that two mechanisms of failure may exist: the loss of subgrade support and excess shear in the aggregate causing failure. Unfortunately the Uganda trial did not reveal very much about rutted and unrutted sections, despite the fact that the penetration in 3 out of 4 sections yielded a slower penetration in the rutted sections than in the unrutted sections.

A simple piece of test kit which would be able to predict the susceptibility of rutting would obviously be a very useful tool. Permanent deformation can occur in any of the road layers or alternatively the subgrade, and this obviously complicates the issue somewhat. A tool which was required to predict the susceptibility of rutting (by testing either the material or the road) would have to be able to measure the potential for both types of failure.

This issue was explored by removing some of the material from an unrutted and a rutted section and testing it in the barrel. However no significant differences were found between the two. This may have been due to the difficulty with establishing why the failure had occurred in the first place. For example: poor compaction when the road was built could have played a significant role.

7.4 DCP: Barrel Test and Borrow Pit Testing – 'RMA'

7.4.1 Findings of the Overseas Testing

There were two avenues of investigation which could have been adopted with regard to the barrel test. Firstly, the test itself could have been calibrated against material performance on the ground. This would have involved testing potential sources of material for use in roads and then using the materials to build and/ or maintain roads and then monitoring how the materials performed in practice and relating this back to the results of the barrel test. However, such an investigation would have been costly in both time and resources and therefore an alternative method was adopted, which involved calibrating the test to existing characterisation tests. The method finally adopted involved a study of the sample characteristics, but this gave inconclusive results.

Table 7-2 - Characteristics of Uganda and Fiji Samples

Samples	LL	PL	PI	PM
Godafoyo pit	46	20	26	1066
Uleppi pit	37	14	23	1173
N - A 8.4km	42	17	25	1175
N-A 41km	35	14	21	1134
N - A 7.4km	39	18	21	1281
N-A 40km	35	16	19	1007
Fiji sample 55	70	31	42	2818.2
Fiji sample 54	95	53	39	1794
Fiji sample 78	61	39	22	1812.8
Fiji sample A	70	36	34	3060
Fiji sample 27	115	59	56	2066.4

Samples	LL	PL	PI	PM
Okibu pit	43	21	22	594
Kapita pit	43	16	27	702
Pangieth pit	40	16	24	456
Alwi ghabi	43	18	25	600
P - O 3km	31	11	20	620
P - O 3.8km	30	11	19	608
Fiji sample B as dug	34	16	18	160.2
Fiji sample 19	74	43.4	31	933.1
coral sand sample	45	22	23	722.2
Fiji sample 32	75	50.1	24.9	946.2

Table 7-3 - Correlations achieved when plotting Fiji and Uganda Data

Graph plotted	Value Of R2 Obtained	
	Fiji/ Uga Material With A Higher PM	**Fiji/ Uga Material With A Lower PM**
DCP 'V' PM	0.2671	0.5935
Change in CBR 'V' DCP top	0.6963	0.4946
Change in CBR 'V' DCP bottom	0.7542	0.1979
Unsoak CBR 'V' change in CBR top	0.9128	0.0311
Unsoak CBR 'V' change in CBR bottom	0.9372	0.267
Unsoaked 'V' soaked CBR top	0.0725	0.6887
Unsoaked 'V' soaked CBR bottom	0.3375	0.6275
Unsoaked 'V' soaked CBR ave of top/bottom	0.1834	0.7031

Table 6-10 and Table 6-13 have summarised some of the attempts at obtaining useful relationships between the various barrel and laboratory tests. The data from these has been combined in Table 7-3 for all the materials tested (as listed in Table 7-2). As stated earlier, these correlations may not always be very meaningful, either because there may be no clear phenomenolgical link between the results compared or because the relationship developed from mathematical perspective may not be one which be sustained from an engineering perspective. From Figure 6-4 and Table 7-3 it can be seen that the best fit of the DCP data to the higher plasticity modulus data gives a highly unlikely relationship. Some more likely straight line relationships are also given which suggests that, in essence, three different kinds of material were tested in the two countries.

Overall, the most positive relationship obtained from the study (which match the required objectives) was that DCP in the barrel can be correlated moderately well with the coarser, less plastic, materials. It is possible that this relationship is less sensitive to moisture content than the finer, more plastic, materials assessed. For these materials the relationship is

$$CBR = 32.37 \times DCP - 107.04 \qquad \text{Eqn. 19}$$

Where DCP is the penetration rate of the 75mm flat foot in the small barrel (300 mm) for the first (100mm) penetration.

Due to the fact that inconclusive results were coming from the correlations being tested it was decided to adopt an approach which aimed to classify the materials as “not suitable for” use or “worthy of further investigation”. For example: a chart which classified the materials as “POOR” or “POTENTIAL”. This involved plotting penetration (in mm) against blows obtained from the DCP for all samples in Uganda and Fiji and then looking to the Plastic Modulus, and the PI (initially) and then the Grading Modulus and the CBR for clues. This resulted in the development of a chart.

7.4.2 The Soils Assessment Chart (“Poor ‘V’ Potential”)

It had originally been hoped that it would be possible to produce a chart which would allow the user to undertake the simple barrel test and then plot his results on a simple chart and the position of the line would tell him whether or not the material was suitable for use in low cost roads or not. It has already been stated that the ideal way to undertake such an investigation would be to test materials and build sample sections of road or maintain sample sections of road and then see how the materials performed. However, as this was not possible, it was decided to look to the types of tests which are commonly used to specify materials: Soaked CBR, PI, PM and also GM.

Table 7-4 summarises the performance data available for all the materials tested in the barrel. The shaded entries indicate materials that are likely to perform poorly for the reasons given in the final column. The barrel DCP results were plotted for each of these materials as shown in Figure 7-1, it is apparent that, with the exception of the material from the Alwi-Ghabi Pit, which appears lows on Figure 7-1, all the well performing materials give DCP results which plot above the solid line indicated and all those having a poor performance plot beneath this line. Therefore the chart is proposed as an initial decision tool, to ascertain whether the material was worthy of laboratory testing. If a barrel test result appeared below the line the material would not be viable, if it appeared above the line it would be a potential road material but would have to undergo laboratory testing to decide.

The “rogue” result from the Alwi-Ghabi Pit, which appears in the ‘poor’only as good as the data which has been used to produce it. Testing of different types of materials, particularly good quality materials, would allow the results to be checked more fully. Unfortunately most of the samples had low CBR values and high PI. It would have been preferable to have a better mix of good and poor quality materials. The issue of the effect of moisture content would also need to be investigated further.

Figure 7-1 - Barrel V Penetration

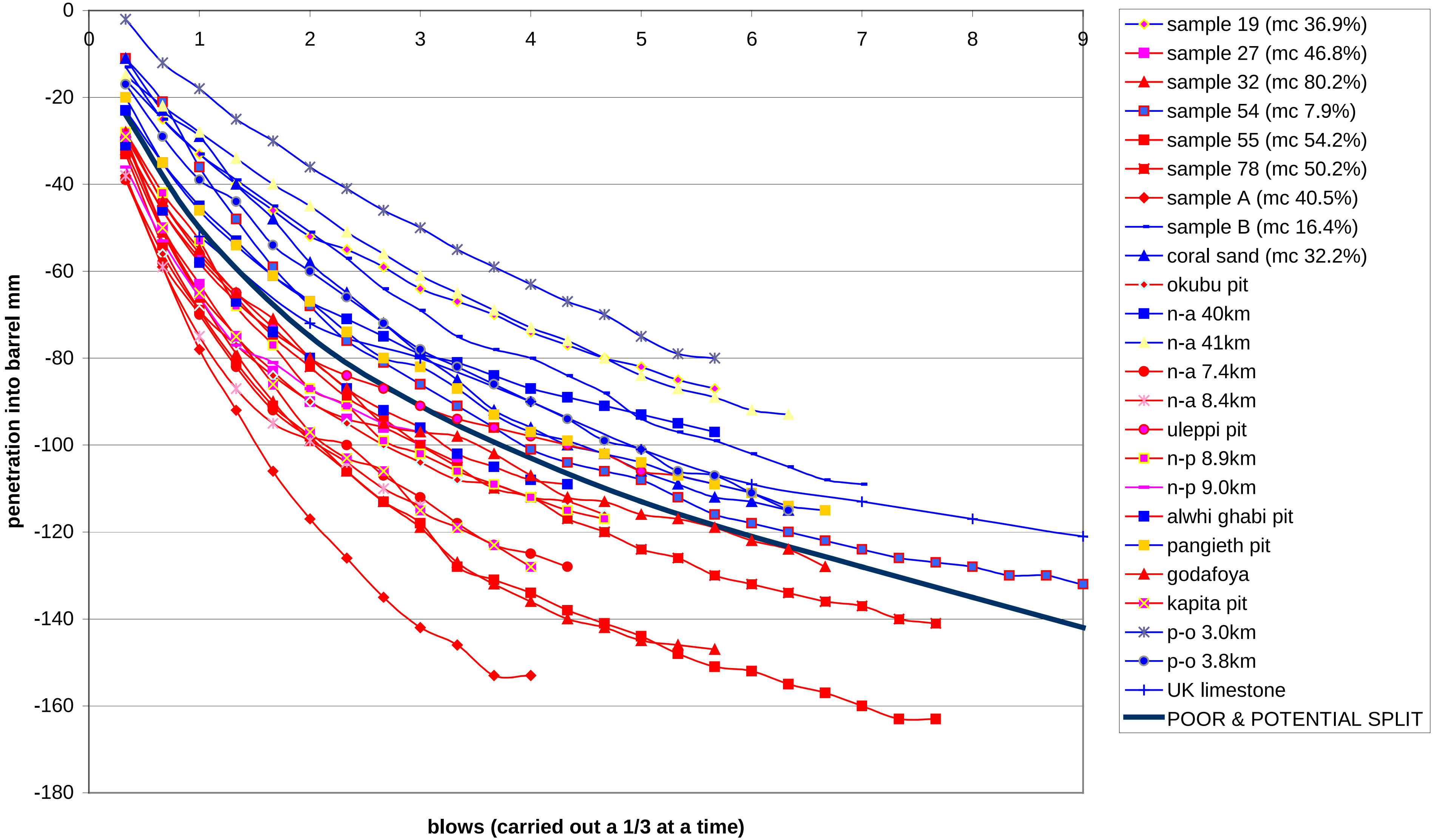


Table 7-4 - Summary of Laboratory Data for Fiji and Uganda Samples on "Poor/ Potential" Chart.

Sample	Approx GM	Soaked CBR Top/bottom	PM	PI	Comments
N-A 7.4km	1.15	6 and 4	1281	21	
N-A 8.4km	1.49	19 and 14	1175	25	
Okibu Pit	2.05	7 and 5	594	22	
P-O 3km	2.04	9 and 6	620	20	Lab testing would reveal the low CBR and high PI
P-O3.8km	2.04	53 and 20	608	19	Lab testing would reveal a high PI
Godafoyo Pit	1.68	3 and 3	1066	26	
Kapita Pit	2.16	32 and 17	702	27	
N-A 40km	1.4	23 and 23	1007	19	Lab testing would reveal high PI and low GM
N-A 41km	1.37	25 and 24	1134	21	Lab testing would reveal high PI and low GM
Uleppi Pit	1.37	4 and 4	1173	23	
N-P	1.67	13 and 15	Non P	-	
Pangieth Pit	2.33	38 and 23	456	24	Lab testing would reveal high PI
Alwi Ghabi Pit	2.26	45 and 40	600	25	
N-P 9.0km	1.73	Estimate 15	Non P		
A	0.65	10 and 6	3060	34	
B	2.50	72 and 62	160.2	18	Sample probably OK for use in road
19	1.828	8 and 17	933.1	31	Lab testing would reveal low (ish) CBR and high PI
27	1.33	10 and 11	2066	56	
32	1.69	12 and 25	946	31	
54	1.49	9 and 15	1794	39	Lab testing would reveal low (ish) CBR and high PI
55	0.875	15 and 8	2818	42	
78	0.614	8 and 12	1812	22	
Coral	1.92	54 and 65	722	23	Lab testing would reveal high PI

The advantage of the barrel test over conventional methods would therefore be that it could be used as an early indication of materials suitability, thus saving money on laboratory testing.

The results were therefore interesting, but more samples would need to be tested, as the chart currently only contains 24 samples from Uganda and Fiji. There is no guarantee that the chart would work for samples from other countries.

7.4.3 The Drop Height

It was decided to adopt a reduced drop height of a 1/3 of a blow, simply because it gave more readings, as the tip entered the material in the barrel more slowly. The relationship between drop height and blows was found to be approximately linear (from the Fiji data) and therefore it was acceptable to adopt a proportionate blow approach when testing in the barrel.

7.5 The Effect of Increased Moisture Content on the Test

The materials in Fiji were used when trying to find correlations between the DCP penetration rate from the small barrel with the 75mm tip. With regard to the Uganda materials, it was noted that the tests undertaken in the 'dry' condition gave much better correlations than the results of the tests undertaken on wetted samples. It was also noted that the samples tested in Uganda which appeared to be dry had a moisture content below 15%. In general the barrel test is very susceptible to the presence of water. The chance to investigate this issue fully was not available, apart from some testing undertaken at different moisture contents in Fiji, which gave the results as covered in 6.4.5.

Further work is obviously needed with regard to the effect of increased moisture content on the test and which moisture contents can be tested. In a tropical country, materials that appear to be dry can actually have a rather high moisture content and there are simply not the facilities to dry out large quantities of material. Thus in such situations, the barrel test would have to be used on materials regardless of their moisture contents and would have to give reasonable results which related to the charts regardless of the moisture content at which it was tested. With this in mind, further investigations into moisture content are suggested, should the work be taken further.

8. IMPLICATIONS AND FINDINGS

The following summary of findings has been split into those which relate to the investigation into resilient modulus, the investigation into rutting, the development of a barrel test and general observations:

8.1 Investigation into Resilient Modulus

The testing undertaken as part of the Bardon trial and the subsequent analysis showed that the small flat tip related to resilient modulus measurements using GDP and TFT better than the other the other trial tips (the standard cone and the dome tip). The use of a modified DCP tip improved the correlation between DCP readings (mm/blow) when plotted against the stiffness (resilient moduli) values from the GDP and TFT. The resilient modulus plotted against the flat tip gave the highest correlation.

The analysis of the resilient modulus against DCP value proved to be quite complex, but allowed an equation to be produced which linked the TFT to the modified DCP penetration rate. However, with future work (which would require testing with the small flat tip and corresponding data to be obtained from the TFT on overseas soils), the equation could be tested further to see if it was able to provide an approximate value for resilient modulus.

The ability to be able to predict layer stiffness has two uses in pavements. Firstly, for surfaced pavements a greater stiffness in a base material will mean a lower modular ratio with any overlying asphaltic (or other bound) layer, thereby reducing strains in that layer and, hence, extending its life to fatigue failure. This benefit will not be felt by most of the pavements which this project has sought to address as rural feeder roads are seldom surfaced with a bound layer. However, where the upper surface is comprised of a concretionary layer (e.g. of a lateritic material) and overlies an unbound layer then the modified DCP assessment of that lower layer will have value in this manner.

Secondly, the stiffness of the base material is an indication of its load spreading ability when considered against the stiffness of the underlying subgrade. Assuming a subgrade with a particular stiffness which is unaffected by the layer of base material which is placed upon it, then a base with a higher stiffness would be expected to deliver a longer lasting pavement as the stresses applied to the subgrade by the base will be reduced. This is likely to lead to slower development of rutting in the subgrade (experienced as a broad rut at the pavement surface) and to reduced occurrence of pot-holing due to soft spots in the subgrade.

For these two reasons, the relationship between modified DCP result and stiffness should be of value to those responsible for assessing pavement condition and construction achievements. At present, the only in-situ measurements of stiffness have been in the UK or overseas in very dry conditions. Variation of in-situ moisture content need further investigation in order that limits may be set for the conditions of testing.

8.2 Investigation into Rutting

The results yielded from the Bardon trial demonstrated two mechanisms of rutting, with the subgrade support being lost in one case and inadequate shear resistance in the aggregate being seen in the other. The two mechanisms of failure meant that the DCP values in different layers were relevant in particular cases and not others.

With regard to the rutting assessment undertaken in Uganda, little obvious difference was found between the DCP measurements with a modified tip on rutted and unrutted sections of road. As the cause of rutting was not clearly identifiable poor material properties may not have been present, rutting may have been caused by other factors (e.g. poor shape or drainage). In hindsight, a better experiment would have been to construct a section of road and then test it before trafficking with the modified DCP. It would then have been trafficked and monitored for rutting. Obviously, such an experiment would have been time consuming and costly.

8.3 Development of a Barrel Test for use at Potential Material Sources

It has been possible to develop a simple test that allows materials to be tested simply to see if they are at all suitable for use in road maintenance. It appears from the testing which has been undertaken to date, that the penetration rate from the barrel test may be plotted on a chart, which should allow the user to decide whether the material is unsuitable for use in road maintenance and thus not worthy of expensive laboratory testing or whether the material has a potential use in road maintenance and is therefore worthy of further laboratory investigation. This meets one of the project aims. It had been hoped that the chart might allow some quantification of material properties, but this has only been possible for a limited selection of materials and conditions.

Although, at present it is not possible for the test to be a suitable replacement for conventional laboratory testing, it has provided some interesting results and is, perhaps, worthy of further investigation. Further work would require a greater variety of materials to be investigated, along with different moisture contents.

The effect of the largest aggregate on the test must also be considered and also whether the tip actually breaks up the material when a test is undertaken and the effect of the bottom of the barrel on the test. Consideration might also be given to testing in-situ (rather than in the barrel) at the material source. This is likely only to be of use where there is no cementation between particles and no concretionary elements which would be broken up in excavation. In general, ad hoc trials in the present project suggested that this approach may be difficult to use successfully

Material variability obviously affected the results of the barrel test and therefore further testing should be undertaken on a wide variety of materials to assess it's applicability to different materials from different countries.

The moisture content has an effect on the plot of penetration in mm against blows and this is obviously an area that would benefit from further investigation. However the chart which was developed to allow analysis of the materials to take place seems to accommodate different materials at different moisture contents without there being a detrimental affect on the results. The materials were plotted on the graph at their natural moisture content. Wetting and drying of the samples did not appear to effect the results too much. In general the lines for each of the materials did not move from one section of the graph to the other when the moisture content was amended.

There are likely to be potential problems with certain types of material (with large aggregate) as seen in Nepal. The 75mm tip was unable to penetrate such materials properly. However the materials in Fiji and Uganda did not cause the same problem. Such an issue may ultimately limit, somewhat, the countries where the equipment may be used.

8.4 General

The relationship between 1/3 drop heights and full drop heights was approximately linear when used in the barrel test. However, the relationship was not found to be linear when the DCP was used in the road with a standard cone tip.

A principal aim of the project was to select a piece of equipment which could be simply used, maintained and analysed in a developing country, in a low cost rural road scenario. This has mostly been successful. The DCP is easy to use and is not expensive to purchase and maintain. The modified tips and feet can be made in developing countries. The barrel used in the test can be bought or made fairly readily. The analysis is generally not complicated. Training staff to use the equipment can be (and has been) easily undertaken. Providing interpretation of the results is not complicated, it is envisaged that training for this would also be relatively easy.

9. OUTSTANDING ISSUES

9.1 Implementability - with regard to Institutional Issues

'People are more willing to take a risk in trying a new approach if the potential benefits far outweigh the potential difficulties. This means the benefits must not only be to the agency but also to those persons who will be directly involved or who may prevent acceptance or full usage….'[122] Although the authors of this quote were talking about pavement management, the issues are the same for the implementation of new practices and methods. The people who will be directly involved with innovative material assessment methods, are not necessarily those who would benefit. The beneficiaries would probably be significantly higher up in the hierarchy than the users of any new technique. Therefore influence is needed at different levels to encourage the adoption of a new way of undertaking condition assessment and material selection (in this case). The influence is needed to persuade organisations that it is worthwhile adopting the new methods and practices, and will achieve savings. Influence is then also needed to ensure the users will adopt the new methods and understand the reasons for undertaking them. Interestingly the issue of turf protection comes up when trying to introduce equipment which tries to take the subjective element out of something like material selection. If a roads supervisor currently chooses his material sources based upon his experience and knowledge. Why would he want to lose his authority to make decisions to a piece of equipment, that he has not seen work and has not been involved in developing? Furthermore, the intention is to move the selection away from a regional roads supervisor to a more local, less expert, user. This introduces a further element against which the roads supervisor may rebel. These are real implementation issues to be overcome and there are probably many others.

9.1.1 Background to Institutional Issues

The investigations into the maintenance aspects of the KaR project undertook an investigation into a number of the issues which constrain effective and efficient maintenance, through interviews with experts and investigations of a variety of manuals. This work identified the fact that many of the problems surrounding maintenance often appear to be insurmountable. The report found that many years of research effort, funding and work by consultants (amongst others) has gone into trying to improve maintenance. However, such improvements, particularly those of an institutional nature, require years to take effect.

Institutional issues are complex and involve human relations and social factors which are difficult to measure in scientific terms. The number of factors are large and are likely to be different in each organisation, in each region, in each country. As a result, the issues which need to be tackled require in-depth investigations to be undertaken, and the resulting 'potential' solutions can take a significant time to implement with no guarantee that they will actually achieve the required result.

The research into the maintenance element of this work attempted a logical identification of the underlying causes of poorly executed and managed maintenance, by approaching the issues from an interface between 'social research' and 'engineering research' and using essentially qualitative data.

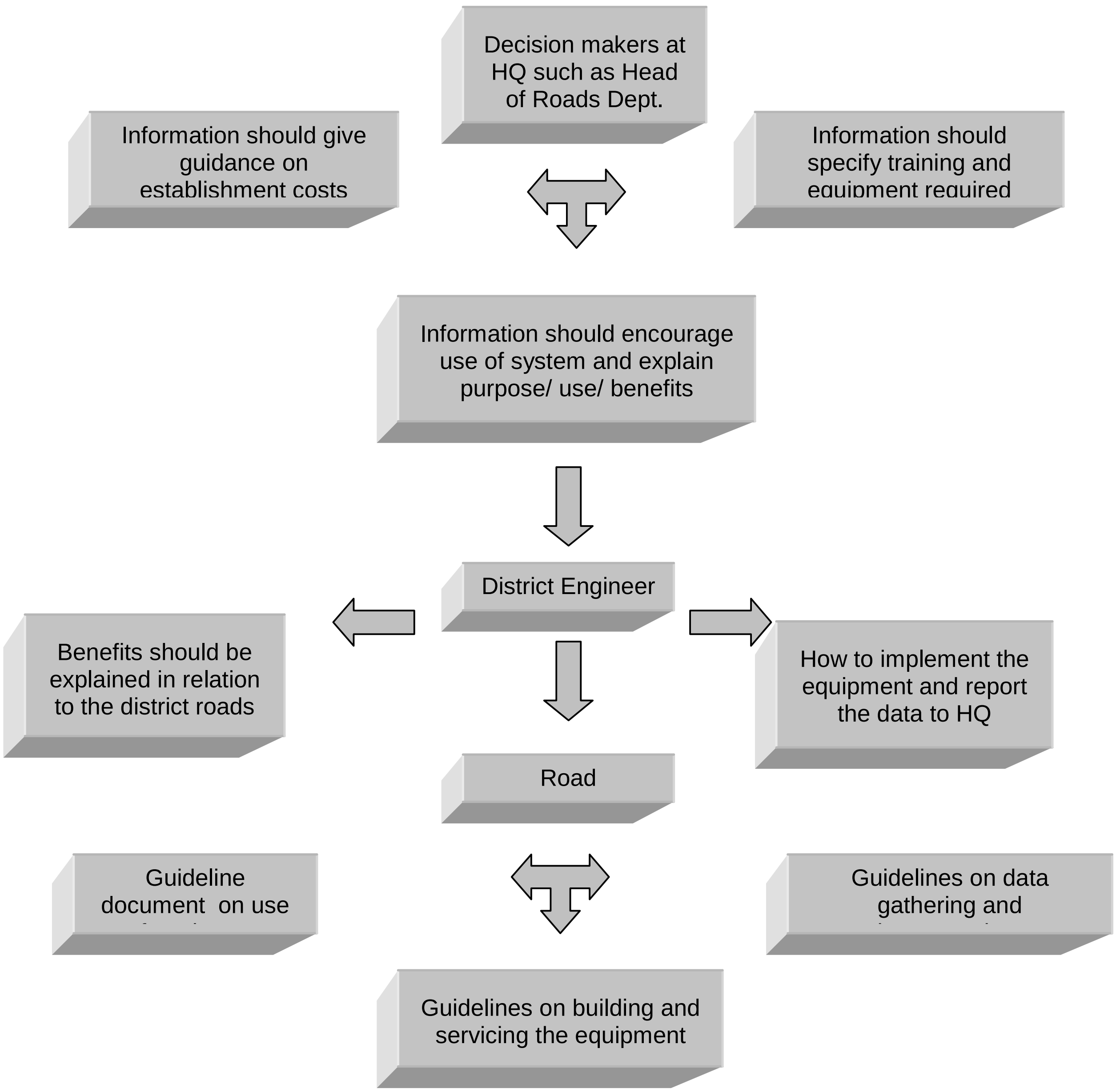
9.1.2 Institutional Issues Affecting the Introduction of the Equipment

By remaining mindful of the issues which may influence the efficient execution of maintenance in developing countries, one can give up and say that no piece of equipment will ever be adopted at the level of the organisation which is responsible for low cost rural roads. Alternatively one can accept that there are many problems, and expect that if some of them were solved, the equipment could be usefully adopted in some places. In the case of the work undertaken as part of this work, it is necessary to accept that a framework of constraints to effective maintenance exists and then work within those constraints.

[122] Unsealed roads manual – Guidelines to good practice, ARRB, 1993, p3.12.
[122] Overview of Institutional Issues in Pavement Management Implementation and Use, Roger E Smith and James P Hall, 3rd International Conference on managing pavements, May 22-26 1994, p59.

With that in mind the equipment and associated instructions and methods must be simple, cheap and effective. With regard to the modified DCP for the assessment of stiffness (resilient modulus) and rutting characteristics, one could argue that the equipment would never be used as the organisations have barely enough time and money to undertake visual condition surveys, let alone physical ones. We know that is the case, which means that the equipment is likely to be of use only on projects which are funded, and are undertaken in an artificial environment (such as that introduced by the presence of a consultant) or within those organisations which do not have such a resourcing problem (a rare occurrence in developing countries). As for the Road Materials Assessment (RMA) tool (the DCP in the barrel with the 75mm flat tip), its purpose is to assist local district engineers, supervisors etc in choosing suitable material sources for building and maintaining gravel roads. At present, the local person's knowledge is relied upon when a search for material is made, often in the absence of any laboratory data. According to consultants such as Roughton International, the local supervisor isn't always successful. This is why the work carried out in Materials Management as part of this study as well as this study need gel together for a more complete understanding. The guidance document has therefore been written in a very simple way. Whether or not the equipment will be adopted then depends upon the willingness of the proposed user group to adopt it and whether or not they have the resources to do so, or even whether policy exists to force them to use it. The following flow chart (Figure 9-1) illustrates the different levels of staff involved and the required type of information with regard to the new equipment. At present only a guidance document on the use of the equipment has been produced, and thought would need to be given to other levels of information required to demonstrate the benefits of using the equipment to the decision makers in an organisation.

Figure 9-1 - Levels of Staff and Information Required to Implement New Equipment



So which is the most appropriate method of ensuring the equipment is adopted in an organisation? Ultimately, whether or not a new piece of equipment is adopted will depend greatly on institutional issues, and such issues require a significant time scale to resolve.

9.2 Implementability – with regard to Technical Issues

The current work has not considered other defects typical to unpaved roads such as corrugations, ravelling, pot holes and edge deterioration. The materials used in the laboratory in the UK were standard road materials, this provided a starting point for the work. The site work then enabled overseas materials to be tested. The materials tested overseas were generally sampled from existing gravel roads or sources of materials typically used to build roads.

The pressure of moisture greatly affects the penetration rate achieved from the RMA (barrel test). Until further work can be undertaken, the RMA test must only be undertaken on materials at their natural moisture content or preferably in a ‘dry’ condition as some natural moisture contents are very high.

In addition to providing simple equipment it is also the institutional framework which needs to be provided. Lack of expertise in using equipment is not necessarily the only potential problem.

10. RECOMMENDATIONS FOR FURTHER WORK

In the previous sections the following suggestions have been made.

I. Regarding stiffness measurements:

- Check the observed relationship in other situations than the UK and (dry) Malawi
- In particular check at different moisture contents
- Aim to determine the limiting moisture regime at which DCP readings should be stopped

II. Regarding rutting

- Undertake comparisons of rutted and unrutted materials where drainage conditions are known to be uniform
- Either build purpose made trials or carefully select wheel tracks and between wheel track areas
- Investigate material and moisture impact

III. Regarding material source assessment

- Check the influence of particle breakdown in the barrel. A grading test before and after loading might be considered as part of the development.
- A greater depth of soil in the barrel could give more data, which might be helpful
- The effect of moisture requires some further investigation to enhance existing data.
- The limits of material and moisture applicability require careful definition.
- Further investigations to obtain quantative evaluation of material properties (perhaps in association) with some other test or tests) should be considered.
- Loosely compacted material could be assessed using a similar procedure but with DCP imposed pre-compaction

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In association with
THE UNIVERSITY OF BIRMINGHAM
and
THE UNIVERSITY OF NOTTINGHAM

Report III - Appendix I
Investigation of Material Assessment Apparatus

Appropriate and Efficient Maintenance of Low Cost Rural Roads

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Investigation of Material Assessment Apparatus

February 2000

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February 2000

Department for International Development
Knowledge and Research (KaR) Programme

Project Title:	Appropriate and Efficient Maintenance of Low Cost Rural Roads
DFID Project Reference:	R6852
Subsector:	*Transport*
Theme:	T2
Element A:	Review of Procedures, Standards and Methods
Date:	*February 2000*

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APPENDIX I – SUPPORTING DATA

This is a summary and collation of the test data obtained during the trial

Comparison between 1c1s1s2s3 and 2c1s1s2s3 tests in the barrel

	Alwi ghabi Small 49mm	Alwi ghabi Small 75mm	Pangieth Small 49mm	Pangieth Small 75mm	P-O 3km Small 49mm	P-O 3km Small 75mm	Uleppi Small 49mm	Uleppi Small 75mm
WET 1c1s1s2s3	0.7754	no	0.8888	no	0.8115	no	0.7227	no
WET 2c1s1s2s3	0.9641	no	0.9380	no	0.9114	no	0.7787	no
DRY 1c1s1s2s3	0.8701	no	0.7606	no	0.8724	no	0.8259	no
DRY 2c1s1s2s3	0.8456	no	0.6485	no	0.9380	no	0.8729	no

	Alwi ghabi Large 49mm	Alwi ghabi Large 75mm	Pangieth Large 49mm	Pangieth Large 75mm	P-O 3km Large 49mm	P-O 3km Large 75mm	Uleppi Large 49mm	Uleppi Large 75mm
WET 1c1s1s2s3	0.8164	0.8372	0.7960	0.9622	0.8034	0.3157*	0.6657	0.8997
WET 2c1s1s2s3	0.6817	0.9515	0.9760	0.7853	0.8858	0.8727	0.6799	0.8275
DRY 1c1s1s2s3	0.8811	0.9061	0.8201	0.7978	0.7404	0.8512	0.6765	0.953
DRY 2c1s1s2s3	0.8447	0.9291	0.8886	0.8876	0.9344	0.8016	0.7254	0.8513

*1c1 results are not consistent with the other results which effects the R^2 value.

No comparison was possible between the small barrel using a 49mm and 75mm tip as the small barrel was not sufficiently large enough to allow more than one central test to be undertaken with the 75mm tip. There appears to be no general pattern emerging between the first set of results and the second, yet the R^2 values are generally high, being above 7.5 in all but 8 out of the 48 cases.

Comparison of ALL tests in the barrel

	Alwi ghabi Small 49mm	Alwi ghabi Small 75mm	Pangieth Small 49mm	Pangieth Small 75mm	P-O 3km Small 49mm	P-O 3km Small 75mm	Uleppi Small 49mm	Uleppi Small 75mm
WET	0.8733	no	0.8554	no	0.8426	no	0.7431	no
DRY	0.727	no	0.6017	no	0.8616	no	0.8332	no

	Alwi ghabi Large 49mm	Alwi ghabi Large 75mm	Pangieth Large 49mm	Pangieth Large 75mm	P-O 3km Large 49mm	P-O 3km Large 75mm	Uleppi Large 49mm	Uleppi Large 75mm
WET	0.6855	0.8821	0.8086	0.7979	0.7230	0.5019*	0.6065	0.8318
DRY	0.8394	0.8851	0.8162	0.8374	0.8171	0.6383	0.6978	0.8475

*1c1 results are not consistent with the others.

It can be seen that the values of R^2 are generally high. The question that had to be answered was which of the combinations of barrel and tip gave the most reliable results. Apart from the odd exception, the values of R^2 obtained were generally high. As a final check, the results for the only two non-plastic (sand) samples were assessed and gave the following results:

	Akabi **Small 49mm**	Akabi **Small 75mm**	N - P 9.0km **Small 49mm**	N - P 9.0km **Small 75mm**
WET	0.8319	0.5946	0.9107	0.9024
MC of sample	23.40%	23.40%	12.80%	12.80%
DRY	0.8231	0.898	0.7642	0.9508
MC of sample	0.1% (!)	0.1% (!)	no value	no value

The dry results followed the same pattern as before with the wet results yielding higher R^2 values from the 49mm tip, but only just in one case.

Drop height data relating to investigation in Fiji. The data presented gives the name of the sample, the moisture content and the penetration achieved with 1/3 and full blows. For example: in the first sample A at a moisture content of 40.50%, 3 x 1/3 blows gave a penetration of 81mm and this corresponded to one full blow which yielded a penetration of 87mm.

Moisture %	**40.50%**	**40.50%**	**40.50%**	**37.40%**	**37.40%**	**37.40%**
Sample	**A MC1**	**A MC1**	**A MC1**	**A MC2**	**A MC2**	**A MC2**
BLOWS	**3x1/3 blows**	**full blow**	**factor**	**3x1/3 blows**	**full blow**	**factor**
0	0	0		0	0	
1	81	87	1.077	71.66	65	0.9069
2	116	129	1.108	96.33	74	0.7681
3	132	143	1.0833	107.00	101	0.9439
4				116.00	105	0.9051

Moisture %	**16.40%**	**16.40%**	**16.40%**	**14.40%**	**14.40%**	**14.40%**
Sample	**B MC1**	**B MC1**	**B MC1**	**B MC2**	**B MC2**	**B MC2**
BLOWS	**3x1/3 blows**	**full blow**	**factor**	**3x1/3 blows**	**full blow**	**factor**
0	0	0		0	0	
1	37.33	40	1.071	67.66	60	0.8866
2	64.33	68	1.056	94.33	87	0.9222
3	81.66	87	1.065	110.00	103	0.9363
4	94.00	102	1.085	120.33	114	0.9473
5	104.66	111	1.060	125.33	121	0.9654
6	113.00	121	1.070			

Moisture %	36.90%	36.90%	36.90%	38.90%	38.90%	38.90%
Sample	**19 MC1**	**19 MC1**	**19 MC1**	**19 MC2**	**19 MC2**	**19 MC2**
BLOWS	**3x1/3 blows**	**full blow**	**factor**	**3x1/3 blows**	**full blow**	**factor**
0	0	0		0	0	
1	52.33	57	1.0891	67.33	67	0.9950
2	77.00	82	1.0649	95.33	95	0.9965
3	95.66	104	1.0871	112.33	113	1.0059
4	110.00	119	1.0818	126.00	127	1.0079
5	121.66	131	1.0767	137.00	135	0.9854

Moisture %	46.80%	46.80%	46.80%	50.50%	50.50%	50.50%
Sample	**27 MC1**	**27 MC1**	**27 MC1**	**27 MC2**	**27 MC2**	**27 MC2**
BLOWS	**3x1/3 blows**	**full blow**	**factor**	**3x1/3 blows**	**full blow**	**factor**
0	0	0		0	0	
1	55.66	33	0.5928	40.33	39	0.9669
2	74.33	45	0.6053	52.66	56	1.0632
3				61.00	68	1.1147

Moisture %	7.9	7.9	7.9	10.7	10.7	10.7
Sample	**54 MC1**	**54 MC1**	**54 MC1**	**54 MC2**	**54 MC2**	**54 MC2**
BLOWS	**3x1/3 blows**	**full blow**	**factor**	**3x1/3 blows**	**full blow**	**factor**
0	0	0		0	0	
1	34.66	27	0.7788	40.33	36	0.8925
2	58.66	46	0.7840	62.33	54	0.8663
3	72.66	63	0.8669	77.33	65	0.8405
4	84.66	75	0.8858	89.00	75	0.8426
5	93.33	83	0.8892	97.66	85	0.8703
6	101.33	92	0.9078	103.33	90	0.8709
7	108.00	96	0.8888			
8	114.00	103	0.9035			
9	118.66	108	0.9101			

Moisture %	14.3	14.3	14.3	14.8	14.8	14.8
Sample	**54 MC3**	**54 MC3**	**54 MC3**	**54 MC4**	**54 MC4**	**54 MC4**
BLOWS	**3x1/3 blows**	**full blow**	**factor**	**3x1/3 blows**	**full blow**	**factor**
0	0	0		0	0	
1	38.66	39	1.008	31.00	31	1.000
2	56.00	61	1.089	46.33	50	1.079
3	69.66	74	1.062	57.66	62	1.075
4	81.66	84	1.028	67.66	70	1.034
5	90.66	94	1.036	75.66	76	1.004
6	96.33	100	1.038	82.66	83	1.004
7	102.33	106	1.035			

Moisture %	50.2	50.2	50.2	54.6	54.6	54.6
Sample	**78 MC1**	**78 MC1**	**78 MC1**	**78 MC2**	**78 MC2**	**78 MC2**
BLOWS	**3x1/3 blows**	**full blow**	**factor**	**3x1/3 blows**	**full blow**	**factor**
0	0	0		0	0	
1	70.66	85	1.2028	93.00	82	0.8817
2	97.66	111	1.1365	118.66	103	0.8679
3	113.33	127	1.1205	127.66	108	0.8459
4	125.33	139	1.1090			
5	134.66	147	1.0915			
6	142.33	154	1.0819			

Moisture %	57.1	46.4	46.4	46.4
Sample	**78 MC3**	**78 MC4**	**78 MC4**	**78 MC4**
BLOWS		**3x1/3 blows**	**full blow**	**factor**
0		0	0	
1		59.66	49	0.8212
2	not enough readings	73.00	63	0.8630
3		82.00	71	0.8658
4		88.00	78	0.8863
5		94.00	83	0.8829

Moisture %	54.2	54.2	54.2	65.9	65.9	65.9
Sample	**55 MC1**	**55 MC1**	**55 MC1**	**55 MC2**	**55 MC2**	**55 MC2**
BLOWS	**3x1/3 blows**	**full blow**	**factor**	**3x1/3 blows**	**full blow**	**factor**
0	0	0		0	0	
1	80.00	84	1.05	86.00	78	0.9069
2	125.00	105	0.84	110.33	116	1.0513
3	141.00	115	0.81	119.33	140	1.1731
4	152.33	123	0.80			

Moisture %	50.4	70.9	70.9	70.9
Sample	**55 MC3**	**55 MC4**	**55 MC4**	**55 MC4**
BLOWS		**3x1/3 blows**	**full blow**	**factor**
0		0	0	
1	not enough readings	75.33	60	0.79
2		100.00	82	0.82
3		110.66	95	0.85

Moisture %	32.2	32.2	32.2	32.8	32.8	32.8
Sample	coral sand MC1	coral sand MC1	coral sand MC1	coral sand MC2	coral sand MC2	coral sand MC2
BLOWS	3x1/3 blows	full blow	factor	3x1/3 blows	full blow	factor
0	0	0		0	0	
1	56.00	65	1.1607	82	75	0.9146
2	92.00	105	1.1413	103.66	96	0.9260
3	115.33	127	1.1011	116.33	109	0.9369
4	127.33	132	1.0366			

Moisture %	28.3	28.3	28.3	34.5	34.5	34.5
Sample	coral sand MC3	coral sand MC3	coral sand MC3	coral sand MC4	coral sand MC4	coral sand MC4
BLOWS	3x1/3 blows	full blow	factor	3x1/3 blows	full blow	factor
0	0	0		0	0	
1	68.33	80	1.1707	77.33	60	0.7758
2	88.00	98	1.1136	95.66	75	0.7839
3	97.66	110	1.1262			
4	107.00	116	1.0841			
7	113.33	120	1.0588			

Moisture %	80.2	80.2	80.2	70.7	70.7	70.7
Sample	32 MC1	32 MC1	32 MC1	32 MC2	32 MC2	32 MC2
BLOWS	3x1/3 blows	full blow	factor	3x1/3 blows	full blow	factor
0	0	0		0	0	
1	79.33	82	1.0336	86.66	89	1.0269
2	108.33	113	1.0430	111.33	114	1.0239
3	125.33	128	1.0212	121.66	127	1.0438

Moisture %	66.3	66.3	66.3	72.7
Sample	**32 MC3**	**32 MC3**	**32 MC3**	**32 MC4**
BLOWS	**0.333 ave**	**full blow**	**factor**	**0.333 ave**
0	0	0		0
1	78.00	106	1.3589	
2	97.33	123	1.2636	not enough readings
3	110.33	137	1.2416	