MAINTENANCE REQUIREMENTS FOR LIGHTLY TRAFFICKED ROADS

by

Ian C. Butler, B.Sc.

A thesis submitted to the Council for National Academic Awards in partial fulfilment of the requirements for the Degree of Doctor of Philosophy.

This work was undertaken within the Department of Civil Engineering at Plymouth Polytechnic in collaboration with

Transport and Road Research Laboratory
Devon County Council
Somerset County Council.

August 1982
ABSTRACT

MAINTENANCE REQUIREMENTS FOR LIGHTLY TRAFFICKED ROADS: I.C. BUTLER

A flexible pavement evaluation and overlay design procedure has been developed by the Transport and Road Research Laboratory (TRRL).

The majority of the information used to derive the relationships contained in this procedure was collected on medium and heavily trafficked roads.

Very little performance data existed prior to the start of this project for lightly trafficked roads, with the result that the relationships in the above procedure had to be extrapolated at the lower traffic levels.

This investigation has resulted in the setting up of a large database containing pavement condition, construction and strength information.

A new methodology has been developed for investigating the performance of flexible pavements based upon data gathered over a limited time scale. This methodology has been used to relate deflection to pavement condition and the subsequent relationships used to validate those published by TRRL.

The work has added confidence to the use of the evaluation and overlay design procedure for roads that have carried low volumes of traffic.

An investigation has also been undertaken into the use of deflected shape measurements, made with a Deflectograph, to characterise pavement construction.

Theoretical studies have been made using a finite element approach, together with practical studies based upon deflected shape measurements recorded on a number of lightly trafficked roads.

Resulting from these investigations, a method has been developed for estimating the thickness of bituminous material in a flexible pavement from measurements of deflected shape recorded with a Deflectograph.

Such an estimation procedure would in the long term remove a major obstacle to on-board real-time processing of deflection measurements recorded with a Deflectograph.

In the short term it would help eliminate a great deal of uncertainty that exists at present concerning the pavement construction between locations where cores have been removed.
DECLARATIONS

While registered as a candidate for the degree for which this submission is made, the author has not been a registered candidate for another award of the C.N.A.A. or of a University during the research programme.

All work described in this report is wholly original and was carried out by the author except where specifically noted by reference. Assistance received in the research work and preparation of this report is acknowledged.


Author


Supervisor
ACKNOWLEDGEMENTS

The author wishes to express his gratitude to Dr. C.K. Kennedy, Head of the Department of Civil Engineering, Plymouth Polytechnic, for his supervision and time spent in discussion.

Thanks are due to the staff of the department for their help, in particular to Mr. R. Pick, Mr. B. Pateman and Mr. J. Barker.

Thanks are due to Mr. B. Ferne and Mr. P. Fevre of the Pavement Design Division of TRRL for their technical assistance and to Miss G.K. Meider for typing the manuscript.

STATEMENT OF ADVANCED STUDIES

The author has, during his period of research, attended two residential courses. The first entitled, 'The Analytical Design of Flexible Pavements', was held at Glasgow University in April 1979 and the second entitled 'A Basic Introduction to the Finite Element Method', was held at Cardiff University in September 1979.

The author has written and presented a paper at an International Symposium on 'The Bearing Capacity of Roads and Airfields', held in June 1982 at Trondheim, Norway.

The author has also attended a number of seminars relevant to his areas of research organised by the Transport and Road Research Laboratory and the Department of Civil Engineering at Plymouth Polytechnic.

The author has participated in an annual, three day course, organised by the Department of Civil Engineering at Plymouth Polytechnic, for practising engineers, entitled 'The Deflectograph: use and analysis of results'.
1 FACTORS INFLUENCING FLEXIBLE PAVEMENT DETERIORATION AND THE REMEDIAL MEASURES AVAILABLE

1.1 INTRODUCTION 9

1.2 FACTORS CAUSING WEAR AND DAMAGE TO A FLEXIBLE PAVEMENT 10
1.2.1 Traffic 10
1.2.2 The Climate 18
1.2.3 Design and Construction 26

1.3 SURFACE MAINTENANCE 26
1.3.1 Day to Day Maintenance 27
1.3.2 Programmed Maintenance 27
1.3.3 Maintenance Techniques 28

1.4 PAVEMENT STRENGTHENING 32
1.4.1 Reasons for Pavement Strengthening 32
1.4.2 Types of Strengthening Measures 33

2 ASSESSING PAVEMENT CONDITION IN TERMS OF THE DETERIORATION EVIDENT ON THE PAVEMENT'S SURFACE 35

2.1 INTRODUCTION 35

2.2 AMERICAN FULL-SCALE EXPERIMENTS 37
2.2.1 The WASHO Road Test 37
2.2.2 The AASHO Road Test 38

2.3 BRITISH FULL-SCALE EXPERIMENTS 44
2.3.1 Assessment of Performance 45
2.3.2 Observations from the experiments conducted in the United Kingdom 46

3 ASSESSING PAVEMENT CONDITION IN TERMS OF THE MAXIMUM DEFLECTION OF THE PAVEMENT'S SURFACE 49

3.1 INTRODUCTION 49

3.2 MAXIMUM DEFLECTION 51

3.3 EQUIPMENT FOR MEASURING DEFLECTION 52
3.3.1 Rolling Wheel Techniques 52
3.3.2 Stationary Loading Techniques 59
3.4 FACTORS AFFECTING DEFLECTION

3.4.1 The Effect of Temperature
3.4.2 The Effect of Subgrade Strength
3.4.3 The Effect of Vehicle Loading and Test Speed on Static Deflection
3.4.4 The Effect of Magnitude of the Load and the Rate of Loading on the Dynamic Deflection

3.5 EVIDENCE OF AN EMPIRICAL RELATIONSHIP BETWEEN DEFLECTION AND PERFORMANCE

3.5.1 AASHO Road Test
3.5.2 British Full-Scale Experiments

3.6 THE USE OF DEFLECTION MEASUREMENTS IN ANALYTICAL PAVEMENT STRENGTHENING DESIGN METHODS

3.7 LIMITATIONS OF THE PARAMETER MAXIMUM DEFLECTION

4 ASSESSING PAVEMENT CONDITION IN TERMS OF THE CURVATURE OF THE PAVEMENT'S SURFACE

4.1 INTRODUCTION
4.2 DEFORMED SHAPE OR CURVATURE OF THE PAVEMENT'S SURFACE
4.3 EQUIPMENT USED TO MEASURE CURVATURE
4.3.1 Rolling Wheel Techniques
4.3.2 Stationary Loading Techniques
4.4 DEFINING CURVATURE
4.4.1 Single Value Curvature Measurements
4.4.2 Multi Value Curvature Measurements
4.5 RELATING CURVATURE, DEFINED AS A SINGLE VALUE MEASUREMENT, TO PAVEMENT PERFORMANCE
4.5.1 Practical Studies
4.5.2 Theoretical Studies
4.5.3 Flexible Pavement Evaluation and Strengthening Design Procedures
4.6 RELATING CURVATURE, DEFINED AS A MULTI VALUE MEASUREMENT, TO PAVEMENT PERFORMANCE
4.6.1 An Evaluation and Strengthening Design Procedure based upon Multi Value Curvature Measurements
4.6.2 Relating the Deflected Shape, as defined by a Multi Value Curvature Measurement, to the Properties of both the Pavement and the Subgrade
6.5 CONVERTING ABSOLUTE DEFLECTION MEASUREMENTS INTO EQUIVALENT DEFLECTOGRAPH DEFLECTION MEASUREMENTS

6.5.1 Estimating the Deflection Measured by a Deflectograph

6.5.2 Derivation of the Correlation Factors between the Deflection Beam and Deflectograph Deflections

6.6 THE DEVELOPMENT OF A BITUMINOUS LAYER THICKNESS ESTIMATION CHART FROM PREDICTED DEFLECTOGRAPH CURVATURE AND DEFLECTION MEASUREMENTS

6.7 ASSESSING THE ACCURACY OF THE ESTIMATE OF BITUMINOUS LAYER THICKNESS

6.7.1 Stiffness of the Bituminous Material

6.7.2 The Modulus and Thickness of the Granular Layers

6.7.3 The Modulus of the Subgrade

6.7.4 The Effect of Soft Spots in the Granular Sub-base on the Estimate of Bituminous Layer Thickness

6.7.5 The Effect of Single Vertical Cracks on the Estimate of Bituminous Layer Thickness

6.8 MEASURING CURVATURE WITH THE DEFLECTOGRAPH

6.8.1 Selecting the Position of the Ordinate Deflections to Define the Deflected Shape

6.8.2 Methods used to Record the Deflected Shape Measurements

6.8.3 Problems Associated with Recording Deflected Shape on a Time Basis

6.9 TEMPERATURE CORRECTION OF CURVATURE MEASUREMENTS

6.9.1 The Effect of Temperature Correction on the Magnitude of the Deflection Readings

6.10 SELECTING LENGTHS OF PAVEMENT WITH A RELATIVELY CONSTANT THICKNESS OF BITUMINOUS MATERIAL
6.11 DERIVING A RELATIONSHIP BETWEEN CURVATURE AND MAXIMUM DEFLECTION FOR PAVEMENTS WITH VARIOUS THICKNESSES OF BITUMINOUS MATERIAL

6.11.1 Defining a Curvature and Maximum Deflection Relationship

6.11.2 Analysis Techniques

6.11.3 Correlation Coefficient

6.12 COMPARISON OF THE PREDICTED AND EXPERIMENTAL CURVATURE AND MAXIMUM DEFLECTION RELATIONSHIPS FOR PAVEMENTS WITH VARIOUS THICKNESSES OF BITUMINOUS MATERIAL

6.12.1 Comparing the Predicted and Experimental Relationships

6.12.2 Adjusting the Structure of the Theoretical Model to more closely resemble that of the Pavements Surveyed with the Deflectograph

7 ASSESSING THE ACCURACY OF THE PREDICTION POSSIBLE WITH THE EMPIRICAL RELATIONSHIP BETWEEN CURVATURE AND MAXIMUM DEFLECTION

7.1 USE OF A NON-DESTRUCTIVE TECHNIQUE TO ESTIMATE CHANGES IN THE THICKNESS OF A BITUMINOUS LAYER OF A FLEXIBLE PAVEMENT AND VERIFICATION OF THESE CHANGES BY COMPARING THEM WITH THE ACTUAL THICKNESS OF EXTRACTED CORES

7.1.1 Theoretical Basis

7.1.2 Use of the Surface Wave Propagation Method at a Single Frequency

7.1.3 Application of the Single Frequency Technique to Determine Changes in the Thickness of the Bituminous Layer of a Flexible Pavement

7.1.4 Conclusions

7.2 USING TEMPERATURE CORRECTED DEFLECTED SHAPE MEASUREMENTS AND THE EMPIRICAL CURVATURE AND MAXIMUM DEFLECTION RELATIONSHIPS TO ESTIMATE THE THICKNESS OF BITUMINOUS MATERIAL

7.2.1 Temperature Correcting the Curvature Measurements

7.2.2 Estimating the Thickness of Bituminous Material

7.2.3 Conclusions
8 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER WORK

8.1 INTRODUCTION

8.1.1 Theoretical Investigations
8.1.2 Empirical Investigations
8.1.3 Comparison of the Theoretical and Empirical Relationships between Curvature and Maximum Deflection
8.1.4 Validation of the Empirical Curvature and Maximum Deflection Relationships

8.2 FACTORS INFLUENCING THE RELATIONSHIP BETWEEN CURVATURE AND MAXIMUM DEFORMATION

8.2.1 Plane Elasticity
8.2.2 Correlation Factors
8.2.3 Temperature Correction of the Deflected Shape Measurements
8.2.4 The Properties of the Lower Pavement Layers

8.3 CONCLUSIONS

8.4 A PROCEDURE FOR CONTINUOUSLY ASSESSING THE THICKNESS OF BITUMINOUS MATERIAL IN A FLEXIBLE PAVEMENT STRUCTURE

8.5 FURTHER WORK

9 THE BACKGROUND AND DEVELOPMENT OF THE UNITED KINGDOM FLEXIBLE PAVEMENT EVALUATION AND STRENGTHENING DESIGN METHOD

9.1 INTRODUCTION

9.2 DERIVATION OF DEFLECTION AND PERFORMANCE RELATIONSHIPS

9.2.1 Deflection History
9.2.2 Deflection Measurements
9.2.3 Pavement Condition
9.2.4 Traffic
9.2.5 Deflection Criteria Curve

9.3 EARLY FULL-SCALE DESIGN EXPERIMENTS

9.3.1 Granular Bases
9.3.2 Tarmacadam Bases
9.3.3 Sand Cement Bases
9.4 THE ALCHEMISTRY HILL EXPERIMENT

| 9.4.1 | Pavement Strength | 246 |
| 9.4.2 | Pavement Condition | 247 |
| 9.4.3 | Traffic | 247 |
| 9.4.4 | Deflection History | 249 |
| 9.4.5 | Application of the Deflection and Performance Data | 251 |
| 9.4.6 | Conclusions | 253 |

9.5 FULL-SCALE PAVEMENT DESIGN EXPERIMENTS - 1960 to 1965

| 10.1 | STEP ONE | 261 |
| 10.2 | STEP TWO | 261 |
| 10.3 | STEP THREE | 261 |
| 10.4 | STEP ONE - STANDARDISING THE STRUCTURAL STRENGTH MEASUREMENTS | 262 |
| 10.4.1 | Correlation of Deflection Measurements | 262 |
| 10.4.2 | Temperature Correction of Deflection Measurements | 263 |
| 10.4.3 | Assessing the Visual Condition of the Pavement's Surface | 269 |
| 10.5 | STEP TWO - ESTIMATING THE REMAINING LIFE OF A FLEXIBLE PAVEMENT | 269 |
| 10.6 | STEP THREE - DESIGNING AN OVERLAY TO STRENGTHEN A FLEXIBLE PAVEMENT | 273 |
| 10.7 | SUMMARY OF THE INFORMATION REQUIRED TO USE THE FLEXIBLE PAVEMENT EVALUATION AND STRENGTHENING DESIGN METHOD | 275 |

| 10.7.1 | Measurements | 275 |
| 10.7.2 | Traffic | 275 |
| 10.7.3 | The Pavement | 276 |
| 10.7.4 | The Subgrade | 276 |
| 10.7.5 | The Overlay | 276 |
11 VALIDATION OF THE FLEXIBLE PAVEMENT EVALUATION AND STRENGTHENING DESIGN METHOD FOR USE ON LIGHTLY TRAFFICKED ROADS

11.1 INTRODUCTION

11.1.1 The Aim of the Work and the Methodology adopted
11.1.2 Selecting Suitable Sites and Determining Roadbase Types and Bituminous Layer Thicknesses
11.1.3 Relating Deflection to the Condition of the Pavement's Surface
11.1.4 Estimating the Traffic carried by each Site
11.1.5 Developing Relationships between Deflection and Performance and Validating the Position of the Critical Condition Curve for Pavements with Granular Non-Cementing and Bituminous Roadbases

11.2 THE 'SLICE IN TIME' METHODOLOGY

11.3 SITE SELECTION

11.3.1 Introduction
11.3.2 Initial Site Selection
11.3.3 Soil Types and Strengths
11.3.4 Site Re-Selection
11.3.5 Sites Considered in Detail

11.4 PAVEMENT CONSTRUCTION

11.4.1 Introduction
11.4.2 Establishing Pavement Construction Data
11.4.3 Defining Roadbase Type
11.4.4 Defining the Thickness of the Bituminous Layer

11.5 THE USE OF THE DEFLECTOGRAPH ON LIGHTLY TRAFFICKED ROADS

11.5.1 Introduction
11.5.2 Problems associated with Operating the Deflectograph on Lightly Trafficked Roads
11.6 PROCESSING THE DEFLECTION DATA
11.6.1 Introduction
11.6.2 Use of the Mainframe Computer
11.6.3 Use of the Mini Computer

11.7 VISUAL CONDITION SURVEYS
11.7.1 The Procedure Adopted
11.7.2 Condition Survey Sheets
11.7.3 Classification of Pavement Condition

11.8 COMMERCIAL VEHICLE STUDIES
11.8.1 Introduction
11.8.2 Commercial Vehicle Traffic Counts
11.8.3 Portable Weighbridge Surveys
11.8.4 Estimation of Axle and Vehicle Damage Factors
11.8.5 An Improved Procedure for Determining Damage Factors from Traffic Counts and Portable Weighbridge Surveys
11.8.6 Calculating the 'Slice in Time' for each Test Site

11.9 RELATING DEFLECTION TO THE CONDITION OF THE PAVEMENT
11.9.1 Introduction
11.9.2 Matching Deflection and Visual Condition Recorded at 10 m Intervals
11.9.3 Matching Deflection and Visual Condition Recorded at 1 m Intervals
11.9.4 Use of the Deflection Beam to relate Deflection to Condition at the Exact Point of Measurement
11.9.5 Matching Deflectograph Deflection and Visual Condition
11.9.6 Using the Critical Deflection Range to Locate the Position of the Critical Condition Curve
11.9.7 Using the Top of the Sound Deflection Range to Locate the Position of the Critical Condition Curve

11.10 DISCUSSION

11.11 CONCLUSIONS
12 Extension of the Flexible Pavement Evaluation and Overlay Design Procedure to Include the Behavioural Data Gathered on the Lightly Trafficked Test Sections

12.1 Introduction
12.2 Deflection and Performance Relationships
12.3 Overlay Design Charts
INTRODUCTION

The deterioration of a flexible pavement even on well designed roads is generally a slow and seasonal process. The main cause of deterioration is the repetitive nature of the traffic loading and the damage caused has been shown to relate to approximately the fourth power of the axle load.

Inevitably a pavement requires strengthening if it is to continue to provide beyond its design life an adequate surface over which vehicles can travel safely.

In the United Kingdom the proportion of the national road network that is nearing the end of its design life and requires strengthening has increased significantly since the mid-seventies. The increase in fuel costs in the early seventies forced hauliers to make better use of their vehicles and as a result many were running full for a greater proportion of their working lives. This caused a general increase in the level of traffic loading bringing with it a dramatic increase in the rate of deterioration of many of the major roads and a reduction in their design lives when calculated in years.

A considerable amount of money is spent each year on road maintenance and to make it as effective as possible use should be made of a suitable strengthening design method.

Pavement strengthening in the past has only occurred when the materials within the pavement have deteriorated to such a condition that damage is evident on the pavement's surface. Once in this state its load-bearing capacity has been reduced to such an extent that at best a considerable thickness of overlay is required or, at worst, total reconstruction.

A great deal of money and materials could be saved if the pavement was strengthened before it reached a failed condition; ideally when it was in a critical condition.
A flexible pavement strengthening design method has been developed by the Transport and Road Research Laboratory (TRRL) for use in the United Kingdom and is outlined in Laboratory Report LR833. It is possible to use this strengthening design method to:

(i) predict the remaining life of a pavement so that strengthening by overlaying can be timed to coincide with the onset of critical conditions.

(ii) design the thickness of overlay required to extend the life of a road to carry the desired traffic and to indicate lengths of road which have deteriorated sufficiently to require partial or total reconstruction.

Essential to the structural design method is the characterisation of the present structural condition of the pavement in terms of the deflection as measured with a Deflection Beam or Deflectograph. These two pieces of equipment have been standardised for use in the United Kingdom and described in Laboratory Report LR834. The preferred operating procedures for both are given in Laboratory Report LR835.

The majority of the data used to define the relationships given in this method were collected from the full-scale pavement design experiments monitored by the TRRL. These experimental pavement sections consisted of construction thicknesses appropriate to heavy, medium and lightly trafficked roads. During the period for which these sections were monitored various performance relationships were established for the deeper construction thicknesses associated with heavily and medium trafficked roads. Unfortunately, little data was collected from the sections of thinner construction, appropriate to lightly trafficked roads, because these tended to fail prematurely.

The rapid rate of deterioration of these sections could have influenced the parameters used to measure their behaviour and for this reason the data collected was possibly not a true reflection of the performance of these construction thicknesses under normal traffic loading.
In an effort to determine more information about the performance of lightly trafficked roads, a contract was awarded to the Department of Civil Engineering at Plymouth Polytechnic by the TRRL to investigate the behaviour of these roads over a three year period.

This project, as part of the main contract, has two main areas of interest:

(i) validation of the existing evaluation and strengthening design method for use on lightly trafficked roads;

(ii) an investigation into the use of measurements of the deflected shape of the pavement's surface to estimate the thickness of bituminous material in the pavement structure.

A lightly trafficked road was defined as one having a traffic flow that would not produce more than about one million cumulative standard axles in a 10 to 15 year maintenance period. (The value adopted for a standard axle in the United Kingdom is 8160 kg and axle loads are generally expressed as an equivalent number of standard axles).

On the basis of this definition, sections of existing roads in Devon and Somerset have been selected as test lengths. The condition of these test lengths has been assessed from periodic measurement of deflection, made with the Deflectograph and Deflection Beam, and the data obtained from the visual condition survey of the pavement's surface.

This information has been analysed in relation to the pavement material types and thicknesses, and the weights and number of commercial vehicles carried, to define the deflection and performance relationships for lightly trafficked roads.

These relationships have been used to extend the published performance and overlay design charts to include the behaviour of the more lightly trafficked roads.
Validation of existing evaluation and strengthening design method for use on lightly trafficked roads is particularly relevant because of the large proportion of a county's road network that falls into the lightly trafficked category.

The information collected from these lightly trafficked pavement sections has allowed measurements of the deflected shape of the pavement's surface recorded by the Deflectograph to be used as an additional indicator of performance to investigate a relationship between curvature and thickness of the bituminous material in a flexible pavement structure.

The aim was to develop a procedure for continuously assessing the thickness of the bituminous material in a flexible pavement structure and is important for two reasons. Firstly, it would remove a large obstacle to on-board real-time processing of deflection measurements obtained with a Deflectograph. Secondly, it would greatly reduce the degree of uncertainty that presently exists concerning the thickness of bituminous material present in the pavement structure between the positions where cores have been extracted.

Both theoretical and empirical studies have been undertaken.

A finite element model has been developed that can be used to accurately predict the response of pavements with granular and bituminous roadbases to a given load.

Use has been made of the theoretical model to develop relationships between curvature, defined as a differential deflection 200 mm from the point of maximum deflection, and maximum deflection for pavements with various thicknesses of bituminous material. These relationships could be used together with measurements of the deflected shape recorded by a Deflectograph to estimate the thickness of bituminous material in a pavement structure.

Empirical relationships between curvature and maximum deflection have been developed from deflected shape measurements recorded on actual pavement structures. These relationships were similar to those derived
theoretically, but differed in numerical value.

The difference has been shown to be largely due to the differences in the structure of the pavements modelled theoretically and those actually surveyed with the Deflectograph. The accuracy of estimate possible with the empirical curvature and maximum deflection relationships and deflected shape measurements has been assessed by comparing the estimates of bituminous layer thickness made in this way with the actual thickness of extracted cores.

This thesis is composed of two parts.

Part 1 contains a literature search that includes the factors influencing flexible pavement deterioration, the remedial measures available, and the methods used to assess the present condition of flexible pavement structures, together with details of the investigation into the use of deflected shape measurements to estimate the thickness of bituminous material in a flexible pavement structure.

Part 2 gives details of the background and development of the existing flexible pavement evaluation and strengthening design procedure for use in the United Kingdom, together with an in depth account of the work necessary to validate this procedure for use on lightly trafficked roads.

Chapters 1 to 8 inclusive constitute Part 1 of this thesis and Chapters 9 to 12 inclusive constitute Part 2.

Chapter 1 discusses the factors that contribute to flexible pavement deterioration and the remedial measures associated with structural and non-structural deterioration.

Chapter 2 describes how measurements of pavement surface deterioration can be used to assess the present condition of a flexible pavement. Details are given of the major full-scale experiments undertaken in the U.K. and the U.S.A. to empirically related axle loads to the performance in terms of the degree of rutting and the extent of cracking of the pavement's surface. The important concepts of
'Present Serviceability Index', standard axle and equivalence factors, are introduced. Observations from the behaviour of the full-scale experiments in the U.K. are mentioned.

Chapter 3 introduces a structural approach to the problem of assessing the present condition of a flexible pavement. The parameter maximum deflection is defined and its use as a measure of the response of a pavement structure to a given load is outlined.

Details are given of the equipment available to measure maximum deflection and the factors that can effect its measured value. Information is given about the empirical maximum deflection and performance relationships that have been developed in the U.K. and included into an evaluation and overlay design procedure. Brief details are given of analytically based flexible pavement evaluation and overlay design procedures.

The main advantages and disadvantages of the parameter maximum deflection as a measure of pavement condition are discussed.

Chapter 4 describes the advantages of deflected shape measurements as indicators of pavement condition. The equipment that has been developed to record the deflected shape is described, together with the various methods of expressing curvature. The relative merits of single and multi-value deflected shape measurements are discussed. Details are given of some of the practical and theoretical studies undertaken to relate single and multivalue curvature measurements to the properties of one or more of the pavement layers.

Chapter 5 outlines the reasons for wanting to be able to continuously assess the thickness of bituminous material in a flexible pavement structure.

A measure of the response of a pavement structure to given load is considered as a possible indicator of bituminous layer thickness.

Details are given of a modified Deflectograph that is capable of measuring the deflected shape of the pavement's surface as a series
of ordinate deflections. A procedure is suggested for expressing these deflected shape measurements in terms of a differential deflection at a specific distance from the position of maximum deflection. The parameters likely to control any estimate of bituminous layer thickness made using measurements of deflected shape are considered.

Chapter 6 describes the investigations into the use of deflected shape measurements to estimate the thickness of bituminous material present in a flexible pavement structure. Details are presented of theoretical studies, using a finite element approach, together with practical studies, using in situ measurements of maximum deflection and deflected shape recorded with a Deflectograph on a wide range of lightly trafficked roads. Resulting from these investigations a method of estimating the thickness of bituminous material in a flexible pavement is presented.

Chapter 7 assesses the accuracy of prediction possible with the empirical curvature and maximum deflection relationships, by comparing the estimates of bituminous layer thickness with those deduced from destructive and non-destructive techniques.

Chapter 8 provides a summary of the more important aspects of the investigations detailed in Chapters 6 and 7, together with a discussion of the factors most likely to have influenced the relationships derived in these chapters. A number of conclusions are drawn that collectively suggest that a method can be developed for continuously assessing the thickness of bituminous material from the analysis of deflected shape measurements made with a Deflectograph. A procedure for continuously assessing bituminous layer thickness is presented and the areas requiring further work detailed.

Chapter 9 marks the beginning of Part 2 of this thesis and gives details of the background and development of the present evaluation and strengthening design procedure for use in the U.K. The important concept of the deflection history is introduced and information is given on its development and use in the derivation of
maximum deflection and performance relationships for pavements with a wide range of roadbase types.

Chapter 10 gives a detailed breakdown of the use of the present more comprehensive evaluation and strengthening design method. Particular reference is made to areas where the format has changed from that originally presented in the initial evaluation and strengthening design procedure.

Chapter 11 outlines the work involved with the validation of the existing evaluation and strengthening design procedure for use on lightly trafficked roads. Details are given of the use of the Deflectograph, use of a portable weighbridge for measuring vehicle weights, the method adopted for recording visual condition, analysis of the results, derivation of relationships, conclusions etc.

Chapter 12 contains the deflection performance and overlay design charts that have been extended to include the behavioural information gathered during this project from the lightly trafficked roads. The range of roads for which these charts are applicable has been considerably increased at the lower traffic level. The approach used in this investigation can also be used to calibrate the existing evaluation and strengthening design procedure to suit local conditions and existing local maintenance standards.
1 FACTORS INFLUENCING FLEXIBLE PAVEMENT DETERIORATION AND THE REMEDIAL MEASURES AVAILABLE

1.1 INTRODUCTION

This chapter gives an overview of the factors most likely to cause wear and damage to a flexible pavement together with details of the measures available to maintain or strengthen a flexible pavement.

Commercial traffic loading and climatic variations are the two factors most likely to influence the rate of deterioration of a flexible pavement.

Details are given of the methods available for converting commercial vehicle flows into estimates of their damaging power in terms of a number of standard axles.

Variations in the climate generally affect the subgrade moisture conditions and the stiffness of the bituminous layers of a pavement.

A review of previous work into the effect of climatic changes on the individual pavement layers and subgrade is given with particular emphasis on the likely effect of extremes of temperature on the pavement materials and their performance.

The objectives of flexible pavement maintenance are specified and the available maintenance techniques outlined.

Flexible pavement strengthening is defined and the reasons for such action detailed.

The two major alternatives for strengthening a flexible pavement, overlaying with a bituminous material and total or partial reconstruction, are considered.
1.2 FACTORS CAUSING WEAR AND DAMAGE TO A FLEXIBLE PAVEMENT

1.2.1 Traffic

The initiation and subsequent development of pavement deterioration caused by traffic are related to the numbers and types of vehicles and to their axle loads.

Defects caused by traffic include fatigue cracking, deformation, wear by loss or polishing of aggregate and excessive embedment of chippings. Cumulative traffic causing structural damage to a pavement is expressed in terms of an equivalent number of standard axles and the development of this concept is detailed in this chapter.

1.2.1.1 Traffic loading

In designing highway pavements and in assessing the effects of traffic on their structural performance, it is essential to quantify the contribution made to structural damage by loads of different sizes.

A most comprehensive investigation of this relation was undertaken during the late 1950s in Illinois, USA, at the AASHO road test\(^{(1)}\) (details given in Chapter 2).

Test data indicated that the damaging power of any load can be expressed in terms of the equivalent axle load factor related to the damaging power of a standard axle \(L_s\).

\[
F = \left( \frac{L}{L_s} \right)^a
\]

The value of \(a\) was found to be dependent upon the thickness of the pavement and the strength of the subgrade. The value obtained by most investigations, including the results of the AASHO test\(^{(1)}\), indicate an average value for \(a\) of 4.0; this has given rise to the so-called fourth power law which states that the ratio of the structural damaging effects of two loads is proportional to the fourth power of the ratio of the loads.
Recent work undertaken in South Africa\(^{(2)}\) with the heavy vehicle simulator has shown that the value of 'a' can vary depending upon the pavement type and the criterion of distress. If the conditions which can be reasonably expected to occur in practice are simulated, then the approximate equivalences obtained for different pavement types are given in Table 1.1.

<table>
<thead>
<tr>
<th>PAVEMENT BASE TYPE</th>
<th>VALUE OF 'a'</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural gravel</td>
<td>2 - 3</td>
</tr>
<tr>
<td>Crushed stone</td>
<td>3 - 4</td>
</tr>
<tr>
<td>Asphalt</td>
<td>4</td>
</tr>
<tr>
<td>Cement treated</td>
<td>6</td>
</tr>
</tbody>
</table>

Table 1.1 Recommended Standard Axle Factors for use in South Africa. (after Freeme, et al)\(^{(2)}\)

On the basis of the data shown above, a value of 4.2 was recommended for use in South Africa if an average value for all pavement types was required. In the United Kingdom the value chosen for the standard axle was 8160 kg.

Equivalence factors determined from the AASHO test data for two pavement thicknesses are shown in Table 1.2 together with those recommended for use in the United Kingdom in Road Note No. 29\(^{(3)}\). There are small variations in the equivalence factors for pavements of different thickness and these have been taken into account by recommending the use of average equivalence factors in Road Note No. 29\(^{(3)}\).

The inference from this table is that one pass of a 'standard' 8160 kg axle causes as much damage as 5000 passes of a 910 kg axle and that one pass of a 18140 kg axle as much damage as 23 standard axles.

It is evident from Table 1.2 that the damage caused to pavements increases very steeply with the axle loading and that the axle loads from private cars contribute very little to structural deterioration. For this reason commercial traffic only is considered for both pavement design
<table>
<thead>
<tr>
<th>Axle Load (lb)</th>
<th>Axle Load (kg)</th>
<th>AASHO Sections (1) total thickness</th>
<th>Road Note 29(3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2000</td>
<td>910</td>
<td>0.0002</td>
<td>0.0002</td>
</tr>
<tr>
<td>4000</td>
<td>1810</td>
<td>0.003</td>
<td>0.0025</td>
</tr>
<tr>
<td>6000</td>
<td>2720</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>8000</td>
<td>3630</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>10000</td>
<td>4540</td>
<td>0.09</td>
<td>0.09</td>
</tr>
<tr>
<td>12000</td>
<td>5440</td>
<td>0.19</td>
<td>0.19</td>
</tr>
<tr>
<td>14000</td>
<td>6350</td>
<td>0.36</td>
<td>0.35</td>
</tr>
<tr>
<td>16000</td>
<td>7260</td>
<td>0.62</td>
<td>0.61</td>
</tr>
<tr>
<td>18000</td>
<td>8160</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>20000</td>
<td>9070</td>
<td>1.51</td>
<td>1.5</td>
</tr>
<tr>
<td>22000</td>
<td>9980</td>
<td>2.18</td>
<td>2.3</td>
</tr>
<tr>
<td>24000</td>
<td>10890</td>
<td>3.03</td>
<td>3.2</td>
</tr>
<tr>
<td>26000</td>
<td>11790</td>
<td>4.09</td>
<td>4.4</td>
</tr>
<tr>
<td>28000</td>
<td>12700</td>
<td>5.39</td>
<td>5.8</td>
</tr>
<tr>
<td>30000</td>
<td>13610</td>
<td>6.97</td>
<td>7.6</td>
</tr>
<tr>
<td>32000</td>
<td>14520</td>
<td>8.88</td>
<td>9.7</td>
</tr>
<tr>
<td>34000</td>
<td>15420</td>
<td>11.18</td>
<td>12.1</td>
</tr>
<tr>
<td>36000</td>
<td>16320</td>
<td>13.93</td>
<td>15.0</td>
</tr>
<tr>
<td>38000</td>
<td>17230</td>
<td>17.20</td>
<td>18.6</td>
</tr>
<tr>
<td>40000</td>
<td>18140</td>
<td>21.09</td>
<td>22.8</td>
</tr>
</tbody>
</table>

Table 1.2 Equivalent Load Factors
and pavement performance purposes. To use the information given in Table 1.2 to predict either past or future traffic loading requires that the distribution of existing axle loads be determined.

1.2.1.2 Assessing the Damaging Effect of Commercial Traffic

Wheel load spectra for mixed traffic can be obtained from dynamic weighbridges placed in the road. These weighbridges count each axle that passes over them and places it into one of a predetermined number of load categories.

Whiffin, and Grainger (4) and Currer (5) have presented data collected in this way on a number of major roads in the United Kingdom.

The number of standard axles for each category is calculated from dynamic weighbridge data by multiplying the number of axles in that category by the appropriate equivalence factor (Table 1.2).

Where load spectra data is available, therefore, the use of the equivalence data in Table 1.2 leads to a relatively simple technique for estimating the damaging effect of mixed traffic.

Where load spectra data is not available an alternative technique must be used.

The Transport and Road Research Laboratory has monitored the results from dynamic weighbridges, installed in their full-scale pavement experiments and elsewhere, and has determined average values of the number of standard axles per commercial axle for different types of road. Also determined was the average number of axles per commercial vehicle.

The product of these two values gives the number of standard axles per commercial vehicle. This allows the damaging effect of a traffic flow on a particular road to be estimated solely from the number of commercial vehicles it carries. This method of assessing the number of standard axles from a traffic flow is presented in the third edition of Road Note No. 29 (3) and is reproduced in Table 1.3.
<table>
<thead>
<tr>
<th>Type of Road</th>
<th>Number of axles per commercial vehicle</th>
<th>Number of standard axles per commercial axle</th>
<th>Number of standard axles per commercial vehicle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Motorway and truck roads designed to carry over 1000 commercial vehicles per day in each direction at the time of construction</td>
<td>2.7</td>
<td>0.4</td>
<td>1.08</td>
</tr>
<tr>
<td>Roads designed to carry between 250 and 1000 commercial vehicles per day in each direction at the time of construction</td>
<td>2.4</td>
<td>0.3</td>
<td>0.72</td>
</tr>
<tr>
<td>All Other Public Roads</td>
<td>2.25</td>
<td>0.2</td>
<td>0.45</td>
</tr>
</tbody>
</table>

Table 1.3 Standard Axle Factors specified in Road Note No. 29.(3)

<table>
<thead>
<tr>
<th>Traffic Loading</th>
<th>Equivalent number of standard axles per commercial vehicle</th>
<th>% Increase in original Road Note 29 Figures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roads designed to carry over 2000 commercial vehicles per day in each direction at the time of construction</td>
<td>2.75</td>
<td>—</td>
</tr>
<tr>
<td>Roads designed to carry between 1000 and 2000 commercial vehicles per day in each direction at the time of construction</td>
<td>2.25</td>
<td>108</td>
</tr>
<tr>
<td>Roads designed to carry between 250 and 1000 commercial vehicles per day in each direction at the time of construction</td>
<td>1.25</td>
<td>74</td>
</tr>
<tr>
<td>All Other Public Roads</td>
<td>0.75</td>
<td>30</td>
</tr>
</tbody>
</table>

Table 1.4 Standard Axle Factors specified in Technical Memorandum H6/78.(6)
There appears to have been a dramatic change in the axle load spectra for all categories of road in the United Kingdom after the oil crisis of 1972-73. This was reflected some years later by the publication of Technical Memorandum/H6/78 indicating revised standard axle factors for four different categories of road, as shown in Table 1.4.

Reference to Table 1.4 shows very large percentage increases in the figures given originally in Road Note No. 29(3) and these revised figures had a significant effect on design lives. The more lightly trafficked roads will now have actual lives of about 60-70 per cent of their expected design lives, while for the most heavily trafficked roads the actual life will be of the order of 30-40 per cent of their design lives.

The most likely reasons for the increase in the damage factors were: that the number of multi-axled vehicles had increased, vehicles were tending to be operated in a more fully loaded condition and certain types of vehicle were covering larger annual distances.

Currer and O'Connor(7) have published a report giving details of the data collected from 30 dynamic weighbridges installed at a number of full-scale road experiments and on three motorways. The data is presented as a series of charts to show the estimated changes in the

(i) number of axles per commercial vehicle;
(ii) number of standard axles per commercial axle;
(iii) number of standard axles per commercial vehicle;

for four traffic levels during the period 1945 to 2005.

Where axle load spectra data is not available the information in this report represents the next best estimate and as such should be used for the calculation of both past and future traffic in terms of numbers of standard axles.

1.2.1.3 Directional and Other Effects

The average values of standard axles per commercial vehicle published in H6/78(6) implicitly assumes that these values can be determined solely
from the total number of commercial vehicles using a road. Where weighbridges have been installed in both carriageways of a road, it has been demonstrated that this can be an unrealistic assumption. Currer\(^{(5)}\) has reported the following values for two sites in which traffic in both directions was classified:

<table>
<thead>
<tr>
<th>Road</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>M6 (Birmingham)</td>
<td>0.43</td>
<td>0.52</td>
</tr>
<tr>
<td>A1 (Alconbury)</td>
<td>0.26</td>
<td>0.59</td>
</tr>
</tbody>
</table>

Details were also given of variations both along a road and in each direction:

<table>
<thead>
<tr>
<th>Road</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>A40 (east end)</td>
<td>0.31</td>
<td>0.36</td>
</tr>
<tr>
<td>A40 (west end)</td>
<td>0.21</td>
<td>0.26</td>
</tr>
</tbody>
</table>

The effect of direction is a significant problem and its importance has been shown from the work undertaken on traffic loading as part of this project and is commented upon later in the report.

On multi-lane highways it is necessary to apportion commercial traffic to each of the traffic lanes. Currer and O'Conner\(^{(7)}\) have presented evidence on this, collected from a number of sites in the United Kingdom. This information is given in Figure 1.1.

1.2.1.4 Expressing Commercial Traffic in terms of numbers of Standard Axles

Damage to flexible pavement results from an accumulation of very small irreversible changes brought about by the passage of commercial vehicles. For this reason the cumulative traffic carried by a road is important in determining its present condition and its life expectancy. Cumulative traffic expressed as the total number of commercial vehicles carried can only be an approximate criterion since it has been seen that the constitution of commercial traffic, in terms of vehicle types and their degree of loading, varies from road to road. The concept of expressing traffic in cumulative standard axles appears to be the best method currently available, although it does depend heavily on the fourth power relationship.
Figure 1.1 Total Commercial Vehicles per day in one direction. (After Currer and O'Connor). (7)
In the full-scale design experiments on which the current British design standards are based, the axle load spectrum at each site has been used to relate performance with the cumulative number of standard axles carried. To use these standards the design engineer has to estimate as best he can the cumulative number of standard axles which the pavement he is designing will need to carry during its design life. If axle load spectra data is not available use should be made of the data given in LR910.(7)

1.2.2 The Climate

The climate can have a dramatic influence on the performance of a flexible pavement, particularly due to its effect on subgrade moisture conditions and stiffness of upper bituminous layers.

The strength of the subgrade supporting a pavement is dependent upon the moisture content of the soil especially in the first few metres below formation level.

The amount of water present is defined by the position of the water table which varies from season to season. In the United Kingdom the water table is generally at its highest during the Winter and Spring and at its lowest during the Summer. It is when the water table is at its highest that the subgrade is at its weakest, and in this weakened condition the subgrade is most susceptible to permanent deformation.

The elastic, deformation and fatigue properties of bituminous materials are all temperature dependent. The elastic modulus or stiffness decreases with increasing temperature resulting in an increase in the traffic imposed stresses on the materials below the bituminous layers. This effect means that permanent deformation of a flexible pavement is most likely to occur in the Summer when the temperatures are at their highest. At lower temperatures bituminous materials become increasingly brittle and are more liable to fatigue failures under repeated stress. At very low temperatures thermal cracking due to tensile failure is possible. This type of pavement distress occurs in cold climates where it has been reported that such cracking is transverse in character with spacing ranging from
1.5 m to 9 m. Very low pavement temperatures can result in pavement damage\(^{(15)}\) due to heave in the subgrade or sub-base. This heave arises not from the expansion of the water on freezing, but from a continuous migration of moisture into the freezing zone from the unfrozen material below. On certain types of foundation heave as great as 80 mm may occur followed by a temporary but often considerable loss of strength during the thaw. In the absence of noticeable heave, crazing may be evident in some forms of flexible pavement.

1.2.2.1 Effect of Climate on the Subgrade

The supporting power of the subgrade soil is dependent upon its moisture condition. Under idealised conditions the variation in subgrade moisture depends primarily upon the relative position of the ground water table if it is within a certain depth below the pavement. Variations of the position of the ground water table and hence variations of subgrade strength are dependent upon the climatic conditions experienced. Croney and Bulman\(^{(8)}\) presented a review of the research work into the effect of climate on the pavement layers and subgrade, and decided that subgrade moisture conditions could be classified in three main categories detailed below, where reference is also made to the methods for estimating subgrade strength from the equilibrium moisture condition for each category.

1.2.2.1.1 Category 1

Under conditions of rainfall and evaporation, such that the water table forms in the soil within 5 m from the surface, an equilibrium moisture condition will develop under an impervious pavement. Reference was made to a method\(^{(9)}\) for estimating the equilibrium moisture condition from absorption and compressibility characteristics of the soil, and the position of the water table.

1.2.2.1.2 Category 2

Where there is no water table within 5 m of the surface, but the climate is such that for several months of the year rainfall exceeds
moisture loss, a relatively stable moisture condition will be achieved under an impervious surfacing by a process of moisture exchange to and from the verges. Reference was made to a method (11) relating the equilibrium condition to soil type, annual rainfall, evaporation rate and meteorological factors. The average moisture content and the strength of the soil could be estimated using this method.

1.2.2.1.3 Category 3

In dry arid climates such rainfall as there may be has no influence on the moisture conditions under the road, and the moisture content of the subgrade will be close to the average value observed in the surrounding uncovered soil of the same type (12).

Croney and Bulman (8) produced a table to show the effect of climate on pavements from data given in the various references quoted; this table has been reproduced in this report as Table 1.5. Table 1.5 gives details of the probable effect of six different climatic conditions on four broad soil types and its inclusion in this report serves merely to show the likely magnitude of the change in the material strengths as a result of changes in the climate.

1.2.2.2 Effect of climate on sub-base and base materials

As a result of the review of available information, Croney and Bulman (8) stated that unless the water table in a road structure was very high there was no evidence of long term moisture exchange with the subgrade sufficient to affect the strength of the sub-base/base layers. The point was also made that crushed stone base materials normally had a very low moisture content and that there was no evidence to suggest that the strength of such materials was influenced by climate.

1.2.2.3 Effect of climate on the bituminous bound materials

Changes in the properties of the bituminous bound materials are not as a consequence of moisture but temperature. The elastic, deformation and fatigue properties of bituminous materials are all temperature dependent.
Table 1.5 The Effect of Climate on Pavements
(after Croney and Bulman) (8)
1.2.2.3.1 High Temperatures

The elastic modulus or stiffness decreases with increasing temperature resulting in an increase in the traffic imposed stresses on the materials below the bituminous layers. This means that permanent deformation of flexible pavements is most likely to occur in the Summer when the temperatures are at their highest.

Lister (13) shows this to be the case for flexible pavements with rolled asphalt and crushed stone bases. Figure 1.2 shows the development of deformation in a rolled asphalt pavement and its subgrade related to monthly temperature durations with the pavement.

Figure 1.3 shows the development of permanent deformation in the pavement layers and subgrade in a pavement with a crushed limestone base. The results in Figures 1.2 & 1.3 were obtained from the same road experiment and were therefore subjected to the same temperature conditions.

Reference to Figure 1.2 indicates that during the three hottest Summer months when the rate of deformation in both asphalt pavement and subgrade is greatest, a small proportion of temperatures 40 mm below the surface are very much higher than the mean value for these months or the mean annual temperature, whereas during the five coldest months minimum temperatures are much closer to mean temperatures.

Figure 1.3 indicates that in a pavement with a crushed stone base deformation takes place in all layers of the pavement and in the subgrade. For both pavement structures deformation was confined to the Summer months and was greater in the Summer of 1969 which was known to be warmer than the succeeding one.

1.2.2.3.2 Low Temperatures

The stiffness of the bituminous material is temperature dependent; the lower the temperature the stiffer the material. At low temperatures the bituminous material of relatively high stiffness is least able to resist the horizontal tensile strains caused by the repeat load application of the traffic and under these conditions fatigue cracking
Figure 1.2 Development of deformation in rolled asphalt pavement and its subgrade related to monthly temperature durations within the pavement (after Lister (13)).

Figure 1.3 Development of permanent deformation in the pavement layers and subgrade in a pavement with a crushed limestone base (after Lister (13)).
is most likely to occur.

Laboratory investigations of bituminous mixtures in flexure \(^{(14)}\) shows that the number of load repetitions to cause fatigue failure depends upon the magnitude of the tensile strain and that for a given tensile strain there exists a corresponding number of load repetitions below which fatigue will not occur.

Fatigue failure of bituminous material does not just occur at low pavement temperatures but can occur at any typical temperature under the action of traffic. Although laboratory results show shorter lives at low temperatures, in the pavement this will depend upon the pavement structure, the layer stiffness at the low temperature and the strain level generated.

The fatigue life of a particular material includes the length of time necessary for the crack to propagate, after its initiation, sufficiently to cause failure. Under low temperature conditions and correspondingly high bituminous stiffnesses the propagation times will be short with the result that the material will fail quicker at low temperatures.

1.2.2.3.3 Very Low Temperatures

At very low temperatures thermal cracking of the bituminous layer due to tensile failure is possible. This type of pavement distress occurs in areas which experience extremely cold conditions. Details of an investigation into low temperature pavement cracking have been reported by the Soils and Materials Committee of the Roads and Transportation Association of Canada.\(^{(15)}\)

The reason suggested for this type of failure was that the decreases in temperature caused the bituminous material to become stiffer and to contract. Because the bituminous layer is restrained by the underlying layers, the contraction induced stresses often exceeded the tensile strength of the material and when this occurs fracture results. Analysis of the full-scale experiments revealed that
generally fracture initiated at the surface when surface temperatures reached a minimum. Laboratory and field investigations concluded that the major variable involved with low temperature cracking was the nature of the asphalt used, and recommendations were suggested to limit the asphalt specifications and mix stiffness.

1.2.2.3.4 Frost Susceptibility

Very low pavement temperatures can result in pavement damage due to heave in the subgrade or sub-base. This heave arises not from the expansion of the water on freezing, but from a continuous migration of moisture into the freezing zone from the unfrozen material below. On certain types of foundation, heave as great as 80 mm may occur, followed by a temporary but considerable loss of strength during the thaw. In the absence of heave, crazing may be evident in some forms of flexible pavement. Croney\(^{(16)}\) gives details of a freezing test adopted by the TRRL to determine the frost-susceptibility of different materials. Results show that most road materials heave to some extent when subjected to the TRRL freezing test. Criteria have been developed to classify material in terms of their frost susceptibility. However, these criteria do not take into account such factors as the long term effect of the increase in stresses in the upper pavement layers, as a result of the weakened frost susceptible foundations, and the effect of differential heave where pavements join structures etc.

In countries where frost heave is a major problem investigations have been undertaken to determine the effectiveness of various frost retarding layers. Andersson\(^{(17)}\) has reported the details of such an investigation in Sweden where extruded polystyrene foam, granulated expanded clay, mineral wool and pulp mill waste were used as the materials for the frost retarding layer. The frost retarding layer was positioned between the roadbase and the sub-base. The main conclusion from the experiments was that all the frost retarding materials tested caused a considerable reduction in the frost heave, but most of them were detrimental to the bearing capacity of the pavement construction.
1.2.3 Design and Construction

Both the actual design and the construction techniques adopted have a great influence on pavement performance.

1.2.3.1 Design

Underdesigned pavements deteriorate at an increased rate resulting in premature failure. Underdesign can be due to an incorrect design, but it is more likely to be as a consequence of an error in either the estimate of traffic or the strength of the subgrade and/or pavement layers.

Traffic and subgrade strength are the two criteria used in the UK as the basis for designing a pavement structure and errors can arise due to unexpected changes in traffic loading, similar to the increase in 1972/73 or inadequate soil surveys.

Underdesigned pavements which fail prematurely will have to be strengthened either by the application of an overlay or by reconstruction to restore the required level of service.

1.2.3.2 Construction

Poor quality materials and/or construction techniques will have the same effect as underdesign, since the pavement will be weaker than the designer intended.

1.3 Surface Maintenance

Pavement maintenance consists of remedying and/or preventing road surface deterioration so as to maintain initial surface qualities in order to keep a pavement at a given level of service.

The primary objectives of maintenance activities are:

(i) to restore skid resistance.
(ii) to restore evenness (to remedy deformed, rutted, disintegrated or worn surfaces).
(iii) to maintain or restore impermeability (repair of cracks, sealing of joints).

Typical maintenance techniques include: patching, surface dressings, thin overlays and planing.

Maintenance can be classified on several grounds and distinction can be made between 'day to day maintenance' and 'programmed maintenance'.

1.3.1 Day-to-Day Maintenance

Some day-to-day remedial maintenance (patching is a main example) will always be necessary because deficiencies will occur and emergencies arise for a variety of reasons in even the best networks. To maintain acceptable safety conditions will always require a level of expenditure on day-to-day maintenance which cannot be cut back, and which may see a sharp rise following a hard winter. Excessive day-to-day maintenance presupposes the acceptance of a relatively low level of service and this also may be indicative of an inadequate level of programmed maintenance.

1.3.2 Programmed Maintenance

Programmed maintenance aims at ensuring an acceptable level of service at all times with an absolute minimum of day-to-day maintenance. Programmed maintenance presupposes that roads are adequate for the traffic they carry. If this is not the case a policy of programmed maintenance cannot be implemented until sub-standard pavements have been strengthened.

Programmed maintenance prevents the deterioration of the structural characteristics of pavements beyond some defined thresholds of defectiveness by ensuring that future traffic requirements are met. It also prevents the surface qualities of the pavement (particularly evenness and skid resistance) from falling below specified thresholds.
1.3.3 Maintenance Techniques

1.3.3.1 Patching

With the progressive introduction of programmed maintenance, patching operations will be considerably less frequent at least on major roads. However it is essential that, in the case of localised damage of a surface which is otherwise in good condition, patching be done rapidly and be of a high quality.

Patching can be carried out to maintain minimum service levels over a short period on a damaged pavement, for which major repairs have been scheduled.

Patching can be either temporary or permanent, depending upon the working conditions and durability required. Patching may be used to temporarily remedy the undesirable effects of localised deficiencies such as pot-holes, depressions, crazing and single cracks.

Patching methods vary widely from country to country and according to the type of road. Pot-holes can be remedied either by filling in the hole as it is, or by cutting and removing the pavement immediately surrounding it, so that the edges of the repair are in contact with good quality materials over a sufficient depth for it to be supported by pavement courses in good condition.

Comparative tests have shown that high quality patching, although initially more expensive, is cheaper in the long run as it is more durable and does not require frequent attention.

1.3.3.2 Surface Dressing

One of the principal tasks of a maintenance engineer is to preserve skid resistance so as to ensure a high level of road safety. Resistance to skidding at all speeds depends upon the surface texture (micro-texture) of the aggregate particles exposed in the road surface, and at high speeds additionally upon the texture of the road surface as a whole (macro-texture).
Microtexture is an inherent property of the stone and its scale is such that it cannot normally be distinguished with the naked eye; macrotexture is provided by appropriate construction techniques, Figure 1.4.

Microtexture provides frictional resistance between aggregate and tyre. Macrotexture aids rapid drainage of water from the surface of the road in contact with the tyre.

Microtexture is lost by the polishing effect of traffic and some aggregates polish more readily than others. The quality of stones with respect to resistance to skidding is measured by their Polished Stone Value (PSV)\(^{(18)}\), a high PSV denoting high quality.

On roads with bituminous surfacings macrotexture is lost for various reasons—physical wear of the aggregates, aggregates being pushed into the surface, the bleeding of surface dressings.

A commonly used method to restore skid resistance is by the application of a surface dressing. The method involves spraying the surface of the pavement with a suitable binder, followed by hand or machined spreading of the chippings. A rubber tyred roller is often used to seat the chippings into the binder. The use of surface dressings has been criticised for a number of reasons:

(i) they are affected by climatic conditions.
(ii) loose chippings can be thrown up, causing damage to vehicle windscreens.
(iii) their contribution to the structural qualities of the pavement is nil.
(iv) there is no improvement in the evenness of the pavement.
Nevertheless, surface dressings are used because the advantages outweigh the disadvantages. Among the advantages of surface dressings are:

(i) speed and ease of laying;
(ii) low cost;
(iii) good waterproofing characteristics of the surface obtained;
(iv) significantly improved skid resistance.

Because of the reduction in coal tar production in most countries, research has focused on the use of bituminous binders and synthetic binders and aggregates.

Binders used include tar bitumens, penetration bitumens, bitumen emulsions, cutbacks, fluxed bitumens, bitumens containing synthetic material (e.g. elastomers) synthetic binders with epoxy resins.

Choice of chippings will depend upon the choice of surface dressing to be laid and the class of road concerned. Chippings should be hard, polish resistant in relation to traffic, clean and as near regular in shape as possible. Correct choice of binder and aggregate is vital if the dressing is to be successful.

1.3.3.3 Thin Overlays (10 - 40 mm)

Thin overlays of bituminous material are used for damaged pavements which do not require general strengthening or for pavements which, following the necessary repairs, are generally sound but show localised weaknesses.

Thin overlays improve riding quality, remedy rutting, maintain or enhance water resistance, restore skid resistance and can generally improve the appearance of the road. Achieving the required compaction of a very thin overlay remains a major problem and, in connection with this, provision need be made for the application of a tack coat to ensure adhesion to the existing support.

Hot rolled asphalt is used in the United Kingdom as the wearing
course for the maintenance of all major roads and is progressively being adopted for other roads.

1.3.3.4 Planing

The technique of planing damaged or deformed bituminous surfacings has been used for many years as a pre-surfacing treatment, particularly in urban areas where it is necessary to preserve existing levels by removing a layer equal in thickness to that being laid. During the last few years, however, its use has become widespread in a number of countries as a technique in its own right for correcting defect surfacings. The machines are fitted with diamond-studded discs and fall into the following three categories:

(i) Hot Scrapers: These machines heat the surface by diesel oil or propane gas burners and then scrape it with oscillating or non-oscillating steel blades.

(ii) Hot Planers: The heating principle is the same as for scrapers, but the surface is milled by a series of tungsten carbide tools rotating about a horizontal or vertical axis.

(iii) Cold Planers: These machines mill the surface, without heating, using tools rotating about a horizontal axis.

The application of planing techniques include:

(a) removal, in one or several passes, of the surfacing presenting unsatisfactory characteristics which may or may not be followed by the application of a new surface course.

(b) planing of rut edges, as applicable, followed by the application of a new surfacing.

(c) planing off a thin layer of material showing bleeding.

(d) planing localised surface irregularities.
1.4 PAVEMENT STRENGTHENING

Pavement strengthening remedies a structural fault or deficiency, either by supplying an additional structural element or by removing the cause of the deficiency. It provides the extra structural capacity needed to withstand future traffic, particularly commercial vehicles.

Pavement strengthening is usually accomplished by the addition of one or more structural layers to an existing pavement. Strengthening can also be achieved by replacing one or several of the pavement layers with new higher quality material.

1.4.1 Reasons for Pavement Strengthening

Sections of the existing pavement network may require strengthening for one or more of the following reasons:

(1) Inadequate structural capacity.
(2) Unacceptable rate or level of structural distress or deterioration.
(3) Unacceptable maintenance costs.
(4) Level of service.

1.4.1.1 Structural Capacity

The structural capacity of a pavement will progressively deteriorate with time, under the action of the commercial traffic, to a level where it can no longer carry any further traffic.

Inadequate structural capacity will result at the end of a design life when the cumulative traffic loading exceeds that originally estimated for design.

1.4.1.2 Unacceptable rate of Deterioration

Structural distress or deterioration may increase with age and traffic applications to some predetermined \textit{critical} condition indicating the need for pavement strengthening.
1.4.1.3 Unacceptable Maintenance Costs

Clearly, strengthening of a pavement becomes imperative once remedial maintenance becomes prohibitively expensive, or when excessive resources are involved. The definition of an unacceptable level of maintenance costs depends upon several factors including available budget and class of road.

1.4.1.4 Level of Service

To a much lesser extent the level of service given by a pavement, defined by riding quality and/or safety, may also be a reason for pavement strengthening. Factors affecting the level of service include: skid resistance, evenness, rut depth, alignment etc.

1.4.2 Types of Strengthening Measures

The methods used to strengthen flexible pavements can be classified under the two main headings of 'overlay with a bituminous material' and 'total or partial reconstruction'.

1.4.2.1 Overlaying with a bituminous material

The most commonly adopted alternative is that of overlaying because not only is the structural strength increased but improvements are also generally made in the riding quality and/or skid resistance.

Overlays are often used as a preventive measure, i.e. to strengthen an existing pavement prior to the application of a known increase in traffic. Overlays applied to flexible pavements are generally of the bituminous type and the thickness required is dependent upon the condition of the pavement to be strengthened and the proposed volume of traffic. Other factors possibly influencing the thickness of the overlay are the available budget and previous experience.

The number and thickness of the courses constituting the overlay are determined by the design thickness and to some extent the irregularity of the existing road surface.
In general strengthening is only carried out in one course if the total thickness of the overlay is less than about 100 mm and providing that the existing surface is either even or has been regulated beforehand.

Two layers are most widely used in practice; a base course of variable thickness serving both to regulate and to strengthen beneath a wearing course of high quality.

1.4.2.2 Total and Partial Reconstruction

If a pavement is showing signs of serious damage, the existing structure is sometimes removed and replaced with new and generally higher quality materials. This approach is adopted frequently for multi-lane highways where severe structural damage is often confined to the lane carrying the majority of the heavy commercial traffic. Reconstruction of this lane only can be cheaper than the cost associated with the application of a thick overlay which must be applied over all the lanes to preserve the necessary crossfall. Reconstruction also avoids the problems of bridge clearance and level in relation to other roads.

The extent of the reconstruction will depend upon overall structural condition of the pavement and the origin of the weakness. Pavement failure due to deterioration of the pavement materials is remedied by the removal of the surfacing and base and replacement to the original total thickness with materials of higher quality. Pavement failure due to deterioration of the subgrade requires a more complete reconstruction, including the removal of any frost susceptible materials.

For a particular length of damaged pavement it is often economic to reconstruct the comparatively short lengths exhibiting excessive deterioration, the limiting length being dictated by the need to efficiently operate conventional laying and compaction equipment. The adjacent lengths are likely to require some strengthening by overlaying and the most satisfactory strengthening solution would be to extend the overlay across the reconstructed lengths, thus avoiding unnecessary changes in level.
2. ASSESSING PAVEMENT CONDITION IN TERMS OF THE DETERIORATION EVIDENT ON THE PAVEMENT’S SURFACE

2.1 INTRODUCTION

Chapter 1 gave details of the factors influencing pavement deterioration and the methods available for remedying both structural and non-structural weaknesses.

To be able to decide upon the correct remedial treatment it is necessary at first to assess the present condition of the pavement structure.

One method of doing this is in terms of the deterioration evident on the pavement’s surface.

The two main criteria used to classify the condition of the pavement’s surface are the degree of rutting under a straightedge laid transversely across the carriageway and the extent of cracking. Rutting and cracking is usually more predominant in the wheelpaths.

The concept of visually assessing pavement condition, in terms of the degree of rutting and extent of cracking, is a simple one; the deeper the ruts and greater the extent of the cracking, the weaker the assumed condition of the pavement.

The repetitive loading action of the commercial vehicles is the factor most influencing pavement deterioration and several full-scale experiments in the USA (WASHO & AASHO)\(^{(19,1)}\) and UK\(^{(21,22)}\) have been undertaken in an attempt to empirically relate axle loads to the damage evident on the pavement’s surface.

Of these full-scale experiments the AASHO Road Test\(^{(1)}\) was the most comprehensive covering a wide range of materials and layer thicknesses and having the distinction of trafficking the pavement sections with known single axle loads.

The concept of Present Serviceability Index (PSI) was introduced for the AASHO Road Test to express pavement performance as a measure of
the pavement's ability to provide a comfortable and safe ride. Four parameters defined PSI; longitudinal surface irregularity degree of cracking, extent of patching and depth of rutting. The performance of the pavement sections was not related to the total construction thickness but to an 'equivalent thickness' which was a function of the strength of the pavement layers. The relative performance of the different pavement sections was expressed as a series of charts relating performance in terms of a PSI value to the number of applications of the known axle load. From such charts 'equivalence factors' were obtained relating the damage caused by one axle load to the corresponding damage caused by applications of a 'standard axle' load.

The concepts of 'standard axle' and 'equivalence factors' are of extreme importance because they can be used to convert traffic flows in terms of a mixed load spectra into an equivalent number of standard axles; a damage criterion.  

British full-scale experiments were incorporated into major realignments of existing highways, resulting in the pavement sections being trafficked by a range of axle loads. The performance of those sections was measured in terms of the depth of rutting and extent of cracking evident on the surface of the pavement.

These full-scale experiments showed that:-

(i) deformation was the principal indication of deterioration of flexible pavements' constructions used in the UK.

(ii) deformation was greatest in the nearside wheelpath.

(iii) deformation as measured on the pavement's surface is a result of component deformations in the constituent layers.

(iv) Severe deformation was usually accompanied by cracking.

Because it was not always possible to measure deformation in relation to the original level of the pavement's surface, it is recommended in the UK that deformation be measured under a 2 m straightedge placed
across the nearside wheelpath.

2.2 AMERICAN FULL-SCALE EXPERIMENTS

Two large scale experiments, namely the WASHO\textsuperscript{(19)} and the AASHO\textsuperscript{(1)} road tests, were undertaken in the USA during the 1950's to study the performance of experimental pavements constructed to a wide range of overall thicknesses, when trafficked by repetitions of known axle loads.

2.2.1 The WASHO Road Test

The WASHO\textsuperscript{(19)} road test was started in 1953, ran until 1954 and consisted of two identical pavement loops, built in Idaho. The performance of the pavements was assessed mainly in terms of the permanent deformation which occurred under the action of traffic. Measurements were made in and between the wheel tracks at monthly intervals. Observations were also made of the amount of cracking which occurred and the elastic deformation measured by the Benkleman Beam.

Results showed that initial rapid deformation under the traffic coincided with the highest Summer temperature conditions, when the elastic modulus of the asphalt was low and the stresses transmitted to the lower pavement layers correspondingly higher.

Much of the initial deformation was due to compaction of the materials by the traffic. This made it difficult to assess the true effect of the seasonal variations, because the duration of the test was such that the only Summer included was that at the beginning of the test.

The performance of the sections with 100 mm asphalt surfacings was much superior to that of the sections of the same total thickness, but with a 50 mm surfacing.

The amount of permanent deformation was reduced by increasing the thickness of the sub-base in all cases. However, the influence of the sub-base was less under the 100 mm surfacing than under the 50 mm surfacing.
Deformation increased with axle load for the sections with 100 mm surfacings. The performance of the sections with 50 mm surfacings was more random, suggesting that small differences in the quality of the roadbase, sub-base and subgrade were more important than the axle load, where the surfacing was thin.

2.2.2 The AASHO Road Test

The second large scale experiment was the AASHO road test\(^{(1)}\) which was started in October 1958 and finished in November 1960. As with the WASHO road test, experimental pavements were laid in loops consisting of straight lengths of normal two lane dual carriageway connected at the ends by circular turnabouts. The material used for the pavements were similar to those of the earlier test, but the thicknesses and axle loads of the test vehicles covered a wider range. Continuation of the test for two years allowed the influence of climatic factors to be studied more completely than the WASHO test.

2.2.2.1 Servicability Index

The concept of 'present serviceability' was introduced to assess the performance of the AASHO pavements under the action of traffic. The adoption of such a concept effectively ruled out any direct comparison between the results obtained from both tests.

Performance expressed in terms of present serviceability was not directly related to the structural condition defined by rutting and cracking. The concept was based upon the principle that the road user was not directly interested in the amount of deformation or cracking present in the pavement over which he drives; he was primarily interested in the ability of the road to provide a comfortable and safe ride.

A panel of drivers were used to assess the acceptability of a number of test lengths in different parts of the USA. Their mean opinion was used to define the Present Serviceability Rating (PSR) of each section.

The conclusion reached was that a PSR = 2.5 represented the critical
condition likely to require an overlay and PSR = 1.5 represented the condition of a pavement unfit to carry further traffic. The PSR = 2.5 value for heavy duty highways was reduced to PSR = 2.0 for secondary low traffic highways.

The following observations were made on the sections to give the Present Serviceability Index (PSI) on a scale in close agreement with the subjective PSR.

The four observations made were:

(i) longitudinal surface irregularity \( \overline{SV} \times 10^6 \)
(ii) degree of cracking, measured over an area of 100 ft\(^2\) \( C \)
(iii) extent of patching, measured over an area of 1000 ft\(^2\) \( P \)
(iv) depth of rutting, average of both wheel tracks \( \overline{RD} \)

The equation used to evaluate PSI was

\[
PSI = 5.03 - 1.91 \log (1 + \overline{SV}) - 1.38 \overline{RD}^2 - 0.01 \sqrt{C + P}
\]

Immediately after construction a new road should have a PSI = 5. In fact, for the flexible pavements at the AASHTO test the initial PSI was equal to 4.2.

2.2.2.2 Structural Number and Equivalent Thickness

To express the relation between pavement behaviour and its structure, a structural number, or thickness index, was defined in terms of the thicknesses and types of material used for the surfacing, the roadbase and the sub-base.

The relation was defined as

\[
D = a_1 D_1 + a_2 D_2 + a_3 D_3
\]

where \( a_1 \), \( a_2 \) and \( a_3 \) are coefficients of equivalence of different materials and \( D_1 \), \( D_2 \) and \( D_3 \) are the thickness of the surfacing, roadbase and sub-base respectively.
For the materials used for the AASHO road test, asphalt concrete surfacing, crushed stone roadbase and sandy gravel sub-base, the relation found was:

\[ D = 0.44D_1 + 0.14D_2 + 0.11D_3. \]

This relationship suggests that unit thicknesses of surfacing was three times as effective as the same thickness of roadbase and four times as effective as the same thickness of sub-base material.

The concept of 'equivalent thickness' related to the granular roadbase material can be obtained by taking \( a_2 = 1 \) and by maintaining the same ratios of \( a_1/a_2 \) and \( a_3/a_2 \).

For the AASHO experiments this gives

\[ T = 3.14D_1 + D_2 + 0.78D_3. \]

Thus it can be concluded that for a given traffic intensity and a given subgrade, the behaviour of a surfaced road depends not upon the total pavement thickness but on the 'equivalent thickness' obtained by adding the total individual thicknesses of the layers, each thickness being weighted by a coefficient of equivalence, itself a function of the rigidity of the material forming this layer.

\subsection{2.2.2.3 Assessment of Performance}

Pavement performance was assessed in terms of the Present Serviceability Index, measured at 14 days intervals, and the relationship between axle load applications and PSI obtained.

To take account of the weakening of the subgrade after the winter thaw and other seasonal effects, a concept of 'weighted' load applications was developed to enable deterioration to be assessed in terms of 'average' load applications.

The shape of the curves relating PSI and the number of weighted applications of axle load \( W \) for all sections were analysed statistically and a model to fit the data was determined.
The model arrived at was:

$$\text{PSI} = 4.27 - 2.7 \left( \frac{W}{\rho} \right)^{\beta}$$

where $\beta$ and $\rho$ are functions of the structural number $D$ and the axle loads.

Therefore, for a particular subgrade and traffic intensity the appropriate value of $W$ can be found for any PSI between 1 and 5. This relationship was used to determine, for any value of PSI, the curve relating $D$ and $W$ for each of the 10 axle loading conditions used for the AASHO road test. Figure 2.1 shows the relationship between thickness index (structural number) and number of load applications for PSI values of 1.5 and 2.5.

From such curves 'equivalence factors' were obtained relating the damage caused by applications of one axle load to the corresponding damage caused by applications of a 'standard' axle. The standard axle load adopted was 18,000 lb (8160 kg), so that the equivalence factor is the ratio of the number of applications of an 18,000 lb axle to the number of applications of the test axle to give the same terminal PSI value. Definitions and details of the terms 'equivalence factor' and 'standard axle' are given in Chapter 1 of this report.

Equivalence factors derived from the AASHO test data were first published by Liddle (20) and are reproduced as Table 2.1.

The structural capacity of each pavement section was expressed in terms of the structural number; the higher the structural number the greater the structural capacity.

Table 2.1 clearly shows the superior performance of sections with a higher structural number by the correspondingly lower equivalence factor, indicating that the stronger the pavement the greater the number of required applications of a standard axle to result in the same amount of damage.

The relative performance of the materials in each pavement section was determined in this way.
Figure 2.1 Relationship between Design and Axle load applications at $p = 1.5$ and 2.5 (1)
<table>
<thead>
<tr>
<th>Class</th>
<th>All LOAD</th>
<th>Equivalent Factors</th>
<th>Flexural Moment</th>
<th>p = 5.3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Right Side</td>
<td>Right Side</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0.0003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>1</td>
<td>0.0007</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>2</td>
<td>0.001</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>3</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>4</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>5</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>6</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>7</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>8</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>9</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>10</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>11</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>12</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>13</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>14</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>15</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>16</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>17</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>18</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>19</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>20</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>21</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>22</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>23</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>24</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>25</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>26</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>27</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>28</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>29</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>30</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>31</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>32</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>33</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>34</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>35</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>36</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>37</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>38</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>39</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>40</td>
<td>0.003</td>
<td>0.0002</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
</tbody>
</table>

Table 2.1 Equivalence Factors derived from the AASHO Test Data. (After Liddle)
2.2.2.4 Observations from the experiments conducted in the USA

Performance expressed in terms of the concept of present serviceability (PSI) is not directly related to the pavement condition defined by rutting and cracking.

The reason being that the PSI concept is based upon the principle that the road user is not directly interested in the amount of deformation or cracking present in the pavement over which he drives; he is primarily interested in the ability of the road to provide a safe and comfortable ride. An assessment of the longitudinal surface irregularity (SV) is used as the riding quality input into the present serviceability equation.

Croney (16) describes how the value of the SV assumes a much greater importance than the other variables in determining the level of PSI.

Consider two pavements with the same value of SV, one of which showed no cracking or patching and the other with a surface covered with cracking or patching. Substitution of the relevant values into the PSI equation would show a difference of only 0.3. Similarly, a difference of rut depth between 0 and 10 mm would affect the PSI by only 0.4.

Because of the statistical nature of the equations this does not mean that cracking and rutting are not important, since they will affect the value of SV. It does mean, however, that the PSI equation cannot be directly used in comparing the AASHO road test results with those from other experiments, in which rutting and cracking have been used as the criterion of structural performance.

The important concept of expressing the damaging power of a traffic flow of varying axle loads in terms of a number of standard axles, derived from these performance studies, is commented upon in Chapter 1.

2.3 BRITISH FULL SCALE EXPERIMENTS

Similar full scale design experiments were also undertaken in the United Kingdom. These experiments were normally carried out on
heavily trafficked roads with advantage being taken of road improvement schemes in order that the sections were laid on newly prepared subgrade and not on foundations which had already been subjected to traffic stresses.

Each site was used to investigate the performance of one or more base materials under various types and thicknesses of bituminous surfacings.

The objective of these experiments was different to that of the WASHO\(^{(19)}\) and AASHO\(^{(1)}\) road tests, in that the performance of each section was not related to a single known axle load but to a combination of axle loads which made up the daily traffic flow. Frequent traffic counts were made and on certain sections these were accompanied by regular axle weighings.

2.3.1 Assessment of Performance

The performance of the sections was judged primarily on the deformation caused by the traffic and the degree of cracking which developed. Permanent deformation of about 25 mm was adopted as one criterion for failure, fatigue cracking arising from heavy axle loads on thin pavements was the other. The failure criterion for permanent deformation of 25 mm was chosen because experience in the United Kingdom indicated that highway authorities normally undertook repair work on roads when the permanent deformation was of this order.

Permanent deformation was measured by observing periodically the levels of rows of studs located in the surface of the road; the individual studs in each row being set at 300 mm intervals transversely across the carriageway. Three or more rows of studs were used in each experimental section making it possible to determine the total deformation of the pavement and where, with respect to the width of the carriageway the maximum deformation had occurred.

Results indicated that the deformation was much greater in the nearside slow lane, carrying the bulk of the heavy commercial traffic, than in the fast lane. Tracking in the wheelpaths was also apparent in both lanes, but the maximum deformation occurred in the nearside wheelpath of the slow lane. This was reported as being a noticeable feature
with all forms of construction used and appeared at all of the experimental sites.

Performance data was expressed in terms of the permanent deformation and the corresponding number of days for which the sections had been trafficked. Figure 2.2 illustrates the effect of (a) thickness and type of base and (b) type and thickness of surfacing on the deformation in the nearside wheelpath.

The most important conclusion reached from these experiments was that the type of surfacing and base was more important in determining the performance of flexible pavement than the overall thickness of the pavement itself. This, and other conclusions, formed the basis of the recommendations for flexible pavement design in the United Kingdom.

2.3.2 Observations from the experiments conducted in the United Kingdom

Deformation or rutting provides the principal indication of deterioration in flexible pavements. It develops chiefly in the wheel tracks of the left hand lane, where the commercial traffic is concentrated. The deformation is greatest in the nearside wheelpath due probably to the tendency of drivers to keep the nearside wheels of commercial vehicles at a constant distance from the verge. The offside wheelpath is less canalised because of the different widths of vehicles.

Deformation increases with the cumulative amount of traffic carried by the pavement.

The total deformation measured at the surface of the pavement is the result of the component deformations in the constituent layers and the subgrade.

Shear, viscous flow, compaction and consolidation can all cause deformation.

Flexible pavements once opened to traffic will exhibit an initial rapid increase in deformation of a few millimetres due to compaction of the pavement layers. A well designed pavement will thereafter show only a
Figure 2.2 Deformation in Nearside Wheel Tracks  
(After Lee and Croney) (21)
small increase in deformation with time (1-2 mm per year), although
the rate depends upon the volume of commercial traffic carried.
Underdesigned pavements will show a larger increase in deformation
with time.

Severe deformation is generally accompanied by cracking in the wheel-
paths. For a road in this condition rapid deterioration is likely to
follow because the cracking allows moisture to enter the road
structure.
3. ASSESSING PAVEMENT CONDITION IN TERMS OF THE MAXIMUM DEFLECTION OF THE PAVEMENT'S SURFACE

3.1 INTRODUCTION

A more fundamental approach to the problem of assessing the present condition of a flexible pavement structure would require some measure of pavement strength.

Ideally the strength would be measured as the response of the pavement structure under a given load, expressed in terms of the stresses and strains developed in each layer.

People became increasingly aware of the need for such information in the 1950's, but the lack of knowledge about the real stress-strain behaviour of roads at that time meant that an alternative approach was required.

The method adopted was the simple in situ measurement of the maximum deflection of the pavement's surface under a standard load.

The maximum deflection of the pavement's surface was selected as an indicator of pavement condition because investigations\(^{(23)}\) had shown that it was dependent upon the stress and strain characteristics of the pavement materials and underlying subgrade and, as such, was expected to correlate in some broad sense with pavement strength and performance.

Several different pieces of equipment were developed to measure maximum deflection\(^{(24-34)}\) and these are briefly commented upon in this chapter. Maximum deflection, as a measure of the response of the pavement materials and subgrade to a given load, can be influenced by changes in material strength\(^{(26,29)}\) and load application\(^{(26,29,37)}\). The possible influence of these changes on maximum deflection are considered, together with the methods derived to compensate for their effect.

Empirical relationships have been developed in the United Kingdom\(^{(36)}\) and elsewhere\(^{(1)}\) between maximum deflection and pavement performance.
Brief details are given of the work by TRRL that has resulted in different deflection and performance relationships being developed for pavements incorporating the four main roadbase types used in the United Kingdom.

These relationships have been included into a flexible pavement evaluation and overlay design procedure\(^{(35)}\) that can be used, together with a knowledge of the present deflection and traffic levels, to estimate the remaining life of a pavement and, where necessary, design an overlay to extend the life of a pavement.

Analytically based flexible pavement evaluation and overlay design procedures have also been developed\(^{(32,40,41)}\) and are commented upon.

These procedures identify the vertical compressive strain at the top of the subgrade and the horizontal tensile strain at the bottom of the bituminous bound layer as the two criteria for assessing the behaviour of the pavement structure.

These parameters were selected because the vertical compressive strain controls the permanent deformation of the subgrade and horizontal tensile strain controls the cracking of the bituminous material.

To use these procedures for assessing pavement performance requires a measure of the existing values of these strains. However the measurement of these strains in the field would be difficult and, for this reason, a measure of the deflected shape is used to characterise pavement condition. Deflected shape measurements are used, together with theoretically based relationships between deflection and performance, to assess the remaining life and, where necessary, design an overlay to extend the remaining life.

The main advantages of using maximum deflection as an indicator of structural condition are the relative simplicity with which the measurements can be made, the large amount of experimental data which now exists, and the strong correlation that has been found between the level of deflection and the overall pavement performance.

The main disadvantages are that as a single parameter it does not
reflect the overall deflected shape and, as such, cannot differentiate between changes in the pavement properties and change in the subgrade properties.

An alternative parameter, more closely related to the condition of the pavement, is therefore required; the use of the deflected shape of the pavement's surface for this purpose is explored in Chapter 4.

3.2 MAXIMUM DEFLECTION

Maximum deflection is a measure of the response of the pavement and subgrade to a given load, and can be recorded as either a static or a dynamic deflection depending upon the loading technique associated with the measuring equipment.

Static deflection is defined as the vertical displacement of a point on the surface of a pavement when a load passes over it, Figure 3.1.

![Figure 3.1 Maximum Deflection of the Pavement's Surface](image)

Static deflection is recorded as being either a total deflection or a rebound deflection depending upon the technique adopted. Total deflection is measured as the wheel approaches the measuring device, and rebound deflection is measured as the wheel moves away from the measuring device.

Dynamic deflection is the vertical displacement of a point on the pavement under the action of a time varying applied load. Unlike
static deflection, which is represented by a point, dynamic deflection is represented by a curve with the amplitude expressed in terms of the frequency of application of the load. A particular deflection value for a given frequency can be determined from the curve.

3.3 EQUIPMENT FOR MEASURING DEFLECTION

Several different pieces of equipment have been developed to measure deflection of a pavement's surface and these generally fall into two categories.

Those which measure the displacement of the road's surface under the action of a rolling wheel load are known as ROLLING WHEEL TECHNIQUES.

Those which measure the displacement under a single or repeat pulse load are known as STATIONARY LOADING TECHNIQUES. Considering each of these techniques in more detail:

3.3.1 Rolling Wheel Techniques

Use of rolling wheel techniques involves the measurement of the maximum deflection or displacement of a point on the pavement under the action of a rolling wheel load. The main criticism levelled at this particular technique is that normal traffic does not move at the creep speed adopted for the test and that the dynamic response of the bituminous materials, whose stiffness is frequency dependent, is therefore not properly characterised.

The strength of this criticism depends upon the conditions in which pavement damage takes place. In the United Kingdom much of the damage is caused by deformation of the pavement layers and underlying subgrade. This takes place in warm and hot weather when the moduli of the bituminous layers are low, because the stiffness of the bituminous material is also temperature dependent.

Research has shown that similar moduli are obtained when deflections are measured at creep speeds under the moderate pavement temperatures used for the sake of practical convenience, to those obtained under normal vehicle loading speeds at higher temperatures when the majority
of the damage occurs.

Another characteristic of such techniques is that the displacements measured with the rolling wheel techniques tends, due to the geometry of such systems, to be related but not equal to the absolute deflection of the road's surface.

3.3.1.1 Deflection Beam

The design of the Deflection Beam\(^{[24]}\) based upon that of the Benkelman Beam and developed at the Transport and Road Research Laboratory, is illustrated in Figure 3.2 and is the one recommended for use in the United Kingdom.

The equipment consists of an aluminium alloy beam sufficiently slender to pass between the dual rear wheels of a loaded truck. It is 3.66 m in length and is pivoted at a point 2.44 m from the tip giving a 1:2 length ratio. The pivot is carried on a frame made of aluminium angle supported by three adjustable legs. The frame also carries a dial gauge arranged to measure the movement of the free end of the beam. Vibration of the beam to minimise sticking at the pivot during recording is achieved by an electric 'buzzer' mounted on the frame close to the pivot. To ensure that the beam is moving freely it is desirable to calibrate it before use.

This manual method of measuring deflection is relatively slow; taking measurements at 10 m intervals, only 1 kilometre of road can be surveyed in a working day. Details of the Deflection Beam are given in the Transport and Road Research Laboratory's Report 834\(^{[25]}\) and the operating procedures for its use in the United Kingdom presented in Report 835.\(^{[26]}\)

The main drawbacks with this equipment are:

(i) the effect on the measured deflection caused by the movement of the beam supports resting in the deflection bowl;

(ii) low speed of operation reduced its feasibility as a routine evaluation tool.
Figure 3.2
Diagrammatic Representation of the Deflection Beam

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Metres</th>
<th>Feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>2.44</td>
<td>8</td>
</tr>
<tr>
<td>b</td>
<td>1.22</td>
<td>4</td>
</tr>
<tr>
<td>c</td>
<td>0.30</td>
<td>1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Metres</th>
<th>Feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>d</td>
<td>0.92</td>
<td>3</td>
</tr>
<tr>
<td>e</td>
<td>0.61</td>
<td>2</td>
</tr>
<tr>
<td>f</td>
<td>1.30</td>
<td>4.25</td>
</tr>
</tbody>
</table>

ELEVATION

PLAN

1. Tip
2. Beam
3. Initial position of wheels
4. Pivot
5. Twin feet
6. Recording dial gauge
7. Base frame
8. Single foot

Road surface

Initial position of wheels

Pivot

Twin feet

Recording dial gauge

Base frame

Single foot
3.3.1.2 Travelling Deflectometer

The Travelling Deflectometer, shown in Figure 3.3, has two deflection beams mounted on a travelling frame, which can be so placed that the deflections resulting from the load carrying dual wheels can be measured automatically.

The dual wheel loading is 33 kN, the speed 1 km/h and a measured road length of 3-4 km per day (between 1500 and 2000 individual deflection measurements at 6 m intervals) is reported.

The whole influence line determined with this equipment is recorded on chart paper as the wheel approaches the beam tip.

Equipment of this type has also been constructed by the National Road Laboratory in Denmark and is known as the Travelling Deflectograph.

Figure 3.4 shows the measuring procedure (one cycle) used for the Danish Travelling Deflectograph. The total length of the truck-semi-trailer is about 15 m. The Deflectograph travels at a constant speed of 1.5 km/h, whereas the reference frame moves discontinuously. During a measurement the reference frame stands on the road on four wheels and is completely independent of the semi-trailer. Measurements are made in both wheel tracks every 11 m usually with a 39 kN wheel load, although a 64 kN wheel load is also possible. The total influence line is recorded on magnetic tape and a computer programme is available which provides the output data, firstly as maximum deflection and then as a measure of the radius of curvature. This system is capable of about 2000 measurements per day.

3.3.1.3 The Deflectograph

The Deflectograph consists essentially of two automated Deflection Beams, one operating with each pair of loaded dual rear wheels of a two axle lorry. The version shown in Figure 3.5 is based upon the original Deflectograph, designed by M. Lacroix in 1956 at the Laboratoire Central des Ponts et Chaussées in France.

This equipment operates within the wheel base of a standard rigid
Figure 3.4 Measuring procedure of the Danish Deflectograph (29)
Figure 3.5 The United Kingdom Deflectograph (after Kennedy) (25)
wheel base lorry and is thus shorter and more manoeuvrable than the deflectometers.

The beam mechanisms are mounted on a common frame to form the beam assembly located directly beneath the lorry and the deflection of the road is measured as the rear wheels approach the tips of the beams, which during this period are at rest in contact with the road surface. After the maximum deflection has been recorded by electrical transducers near the beam pivots, the beam assembly is pulled forward on steel skids, at about twice the speed of the vehicle, by an electromagnetic clutch and winch system, to the initial position appropriate for the next cycle. An arrangement of guides ensures that the beams are 'aimed' at the centre of the space between the rear twin tyres even when the vehicle is negotiating bends.

The working speed of the Deflectograph is about 2 km/h giving a maximum surveyed length of about 10 - 16 km, representing 2500 - 4000 readings per day in each wheel path, although outputs of up to 22 km per day have been achieved by commercial firms. The spacing between each measurement is fixed at 4 m unlike many other types of equipment which can test at any selected spacing.

The main drawback with this equipment is the effect on the measured deflection caused by movement of the beam and reference frame supports, brought about by the loaded front and rear axles. The non-absolute nature of the measurements is only a problem when comparing deflections measured with other pieces of equipment; the problem does not exist if the deflection measurements are used in conjunction with strengthening design procedures developed on the basis of Deflectograph deflection measurements.

3.3.2 Stationary Loading Techniques

Use of stationary loading techniques involves the measurement of dynamic deflection, which is essentially the vertical displacement of a point on the pavement under the action of a varying applied load. The load is transmitted to the pavement through one or two bearing points and is generally much below the maximum permitted axle load. The load is applied at one or several frequencies.
Unlike static deflection, which is represented by a point, dynamic deflection is represented by a curve, indicating pavement reaction, with the amplitude of the deflection expressed in terms of the frequency of application of the load. A particular deflection value can be determined for a given frequency from the curve. The pulse loads associated with these techniques generate stresses and strains within the pavement which differ in several aspects from those generated by the rolling wheels of traffic and the measured deflections are consequently affected.

3.3.2.1 Plate Bearing Tests

Plate Bearing Tests \(^{(31)}\) consist of an almost static load applied to a circular loading plate by a hydraulic jack. During the test the vertical deformation of the pavement is measured. There are several test methods and the result obtained depends upon the equipment used, load, loading time, duration of load application, stiffness and diameter of the plate etc.

Repeated static plate bearing tests, to determine pavement layer properties and deflections, are commonly used in Denmark and Finland. The plate diameter is about 0.3 m; loading is usually up to 59 kN and the number of repetitions between 3 and 10. The deflection is measured through the centre of the plate. The repeated rebound deflection is used for calculation of stiffness values. The testing capacity of this equipment is limited, especially with the more complicated procedure and is of the order of 2 to 10 measurements per day.

3.3.2.2 Falling Weight Deflectometer

With the Falling Weight Deflectometer \(^{(32)}\) a pulse load is applied by a mass falling on a set of springs which are mounted on a rigid circular plate resting on the pavement's surface. Reference to Figure 3.6 shows the Falling Weight Deflectometer mounted on a small trailer containing the power supply and towed by a vehicle carrying the recording equipment.

The falling mass weighs 150 kg and the maximum force developed is 60 kN.
Figure 3.6  The Falling Weight Deflectometer
at a drop of 0.4 m with a pulse width of 28 ms. The load pulse is transmitted to the road by means of a circular plate 0.3 m in diameter. The deflection of the pavement is measured by means of velocity transducers, one in the centre of the loaded area and \( \frac{\text{at a fixed distance from the load.}}{\text{at a fixed distance from the load.}} \)

The response of the pavement to the load is characterised not only by the maximum deflection but also by the shape of the deflection bowl.

A parameter \( Q_r \) is used to define the ratio of the deflection at a distance \( r \) from the load to the deflection under the centre of the test load. Claessen, et al.\(^{32}\) state that the parameter \( Q_r \) was chosen instead of the radius of curvature, because \( Q_r \) can be measured more easily with this equipment.

A total number of 80 measurements per day is reported.

3.3.2.3 The Dynaflect

The Dynaflect equipment was developed in 1965 by the Lane-Wells Highway Products Company of Houston, Texas.\(^{33}\) The Dynaflect is an electromechanical system for measuring the dynamic deflection of the road's surface caused by an oscillatory load. Measurements are independent of a fixed surface reference. The extent of the deflection basin of a flexible structure can also be defined.

Figure 3.7 shows the Dynaflect system. It comprises a dynamic force generator, a set of motion sensing devices mounted on a small trailer, and a motion measuring system which is normally carried in the towing vehicle. The generator produces a vertical force which varies sinusoidally at the rate of 8 Hz. The total force applied to the road surface consists of the static weight of the apparatus plus the dynamic force which alternately adds to and subtracts from this weight. The peak to peak excursion of the dynamic force is 1000 lb. This force is applied to the road through a pair of rigid wheels, which causes it to deflect downwards and upwards in phase with the repetitive force. The amplitude of this motion is sensed by geophones which are lowered into contact with the surface at various distances along the trailer tow bar from the point of application of the force.
Figure 3.7 The Dynafleet in the Test Position with Force Wheels and Geophones Down.
An adjustable measuring range of between 200 mm and 2300 mm along the tow bar permits the shape of the deflection basin to be determined.

A remote controlled lift mechanism in the trailer moves the force generator, with its steel wheels, in or out of contact with the ground. When out of contact, the trailer is supported on rubber tyres for travel at normal vehicle speeds. With the steel wheels down and rubber tyres lifted, the trailer may be moved short distances from one measuring point to another, at speeds up to 6 miles per hour. The sensors are raised and lowered by remote control to enable such moves to be made quickly.

The deflection data is recorded on a data sheet by the operator in the tow vehicle. The visual condition of the road surface adjacent to the point of measurement is recorded, together with information relating the point to the edge of the carriageway and nearest identifiable event. It has been reported that 150 different locations on an airport runway were obtained in an eight-hour day. Measurement on highways varied from 54 to 71 different locations in five hours, depending upon the traffic volume and degree of personnel protection required.

The main drawbacks of the system are:

(a) the low force level which does not represent heavy traffic,

(b) a fixed frequency of 8 Hz sometimes produces resonance.

3.3.2.4 The Road Rater

The Road Rater\(^{(34)}\) is powered hydraulically and controlled electronically. It attaches to the front of a van or a truck and serves as a means of measuring a road's strength by the application of a dynamic load superimposed over a static load. In the working mode, the unit is hydraulically lowered onto the road's surface and adjustable static pressure is applied. Dynamic load is then applied through a steel mass at either 10, 20, 25, 30 or 40 Hz upon actuation of the hydraulic vibrator.
Two upper air springs are designed to minimise vibratory feedback into the vehicle and the six lower air springs centre and transfer the dynamic load onto the surface through two $180 \times 100$ mm steel pads.

A static load of 500 kg and frequency 25 Hz are normally used. The road's deflection profile, measured by four velocity sensors housed in a boom and spaced 0.31 m apart, is visually displayed on instrumentation located inside the vehicle.

Once deflection readings are noted either manually or automatically, the unit is raised off the pavement's surface and moved to the next testing spot.

About 300 measurements at a single frequency or, alternatively, 100 at three frequencies are possible in a working day, the number being dependent upon the spacing of the points of measurement.

3.4 FACTORS AFFECTING DEFLECTION

Deflection is a measure of the response of the pavement materials and the subgrade to the applied load and, as such, can be influenced by changes in material strength or load application. Material strength is affected by environmental conditions; temperature will affect the stiffness of bituminous bound materials and the moisture conditions will affect the strength of the subgrade. The magnitude of the applied load, rate of loading and the system of loading will all influence the measured value of deflection.

3.4.1 The Effect of Temperature

The temperature of the bituminous layers of a road influence greatly its stiffness and therefore the magnitude of the measured pavement deflection. Figure 3.8, after Kennedy, shows the change of deflection with temperature measured 40 mm below the road surface for different types of flexible pavements. Pavements with uncracked cement bound bases appear to be least affected, whereas pavements with crushed stone bases show the greatest rate of change of deflection with temperature, or temperature susceptibility; the two bases containing bituminous material, whose temperatures susceptibilities were
Figure 3.8 Change in Deflection with Temperature on 4 types of Road Pavement heated by Infra-red Heaters. (After Kennedy) [26]
intermediate, had the greatest proportional change of deflection with temperature, as would be expected of pavements whose stiffness was almost entirely derived from temperature-susceptible materials.

It is clear from Figure 3.8 that a correction procedure was needed to allow the comparison of deflection measurements made on pavements at different temperatures. Extensive studies carried out by the TRRL on their full scale pavement design experiments has led to the formulation of charts suitable for correcting deflections to a standard temperature of 20°C. These charts are included in the flexible pavement evaluation and strengthening design method \(^{(35)}\) and selection is dependent upon the thickness and type of bituminous materials present in the pavement.

Kennedy \(^{(26)}\) makes the point that at temperatures greater than about 30°C the pushing up of the softened surfacing material between the dual rear wheels can mask the normal linear increase in deflection with temperature. The temperature correlation charts detailed above cannot be extrapolated beyond 30°C and therefore this temperature should be regarded as the maximum operating temperature in the U.K. Kennedy \(^{(26)}\) also states that at low temperatures the relations between deflection and temperature converge in such a way as to reduce greatly the accuracy with which deflections measured at low temperatures can be corrected to equivalent values at the standard temperature at 20°C.

3.4.2 The Effect of Subgrade Strength

Experience from the TRRL full scale pavement experiments has shown that the subgrade moisture conditions will vary throughout the year and the relative movement of the water table will be reflected in changes in the strength of the subgrade and hence in the measured value of deflection. Evidence suggested that the Spring and Autumn periods were the most satisfactory periods of the year for carrying out deflection surveys. During these two periods the yearly variation in subgrade moisture conditions and hence subgrade stiffness was generally small and temperatures normally lie within a satisfactory range. Similar conditions in respect of subgrade condition have been reported elsewhere \(^{(29)}\) and Figure 3.9 shows the effect of the season on the measured value of the dynamic deflection.
Figure 3.9 Seasonal Variation in Dynamic Deflection
3.4.3 Effect of Vehicle Loading and Test Speed on Static Deflection

Static deflections are normally measured under the action of loaded dual wheels and variation in either the loading on these wheels or the loading of the front wheels will affect the magnitude of both the pavement deflection and the recorded deflection. A simple correction factor can be employed to adjust the recorded deflection if the wheel load differs by only a small amount from that specified.

The crossfalls normally used in modern road construction in the U.K. of 1 in 40 does not produce a significant change in loading. However, redistribution of load on superelevated curves could become substantial and a guide to the changes in wheel loadings which can take place is given by Kennedy. The effect of larger variations in load on measured deflection is less well defined.

Lister and Jones have shown that the elastic modulus of some pavement materials and subgrades depend upon the rate at which they are loaded; the modulus increasing with increased rates of loading. This is reflected in a decrease in the deflection measured with increasing vehicle speed.

It follows that it is necessary to standardise the magnitude of the applied load and rate of loading, and recommendations are usually specified for the test vehicle concerning the loading of the front and rear wheels, wheel base, tyre pressure and size, contact area and vehicle speed.

3.4.4 The Effect of the Magnitude of the Load and the Rate of Loading on the Dynamic Deflection

The magnitude of the applied load will affect the measured value of dynamic deflection (Figure 3.10) and a standard value is adopted for most pieces of equipment designed to measure dynamic deflection, e.g. the falling weight deflectometer applies a mass of 150 kg from a height of 400 mm.

Dynamic deflection also depends upon the frequency with which the load is applied to the pavement's surface. Operation of the equipment at
Figure 3.10 For the same structure the value of the dynamic deflection depends upon the applied load and the amplitude depends upon the frequency. (29)

Figure 3.11 The same apparatus gives different curves for different structures, hence it is difficult to work with one frequency. (29)
a single frequency on all pavement structures is therefore not recommended (Figure 3.11).

3.5 EVIDENCE OF AN EMPIRICAL RELATIONSHIP BETWEEN DEFLECTION AND PERFORMANCE

The use of the parameter maximum deflection as an indicator of structural strength was based upon the apparently valid assumption that it was closely related to the level of stresses and strains developed in a pavement structure. On the basis of this assumption work was undertaken in the United Kingdom (36) and elsewhere (1) to develop empirical relationships between deflection and pavement strength.

3.5.1 AASHO Road Test

The fifth objective of the AASHO Road Test (1) called for relationships that would link measured deflection values with the prediction of future pavement performance. Deflections of flexible pavements proved to be highly effective.

The conclusions arrived at were:

(i) Deflections taken during the Spring when the sub-surface conditions were adverse gave a better prediction of pavement life than those taken in the Autumn.

(ii) There was a high degree of association between deflection and rutting.

All of the deflection data were analysed and a model found by which the life of a pavement to a given level of serviceability could be estimated satisfactorily, provided that both load and deflection were included in the function.

3.5.2 British Full Scale Experiments

Details of the studies to investigate the relationship between deflection and performance by the TRRL in the United Kingdom have been
given by Lister and are commented upon at length in Chapter 9 of this report.

Evidence was given by Lister that the long term performance of a flexible pavement was related to the magnitude of the transient deflections which occur under a rolling wheel load moving at creep speed. This relationship was determined from data collected from the full scale pavement design experiments over a period of about 15 years and expressed in the form of deflection histories (Figure 3.12).

3.5.2.1 Deflection History

The deflection history of a pavement relates the deflection measured under a standard wheel load at standard conditions of pavement temperature to the life of the road (in terms of standard axles) and its condition, determined principally by the permanent deformation and cracking in the wheel paths. The deflection histories thus derived enabled the development of critical conditions within the pavement, when strengthening of the road was most appropriate, to be related to the deflection behaviour under traffic. Critical conditions were defined by the depth of rutting and the extent of the cracking evident on the pavement's surface.

3.5.2.2 Deflection and Traffic Relationships

Figure 3.13 shows the relationship between deflection and traffic up to the onset of critical conditions in sections with rolled asphalt bases. Use of this figure enables the unexpired life of a pavement to be estimated, at any stage before critical conditions develop, from a knowledge of the deflection and the cumulative traffic carried by the road to date.

3.5.2.3 A Flexible Pavement Evaluation and Strengthening Design Method for Use in the United Kingdom

Further work has led to up-dated deflection and traffic relationships being developed for the four main road base types used in the United Kingdom.
Figure 3.13 The relation between Deflection and Traffic up to the onset of Critical Conditions in Sections with Rolled Asphalt Bases at Alconbury Hill. (After Lister)
Figure 3.14 shows the deflection and traffic relationships for pavements with non-cementing granular road bases.

These deflection and traffic relationships have been incorporated into a flexible pavement evaluation and strengthening design procedure\(^{35}\) and can be used to predict the remaining life of a pavement under traffic so that strengthening can be timed to coincide with the onset of critical conditions.

Use can also be made of these deflection and traffic relationships, in conjunction with empirically derived overlay design charts, to design the thickness of overlay required to extend the life of a road to carry any given traffic and to indicate lengths of road which have deteriorated sufficiently to require partial or total reconstruction. The background to and the development of the flexible pavement evaluation and the strengthening design procedure is given in more detail in Chapter 9.

3.6 THE USE OF DEFLECTION MEASUREMENTS IN ANALYTICAL PAVEMENT STRENGTHENING DESIGN METHODS

Deformation of the subgrade\(^{39}\) and fatigue cracking of the bituminous layer have been identified as the main causes of pavement failure.

A method of precluding such failures would be to consider the pavement as an elastic layered system and limit the vertical compressive strain on the subgrade and horizontal tensile strain on the bottom of the asphalt bound layer.

However, the measurement of these strains in the field would be difficult and, as an alternative approach, analytically based strengthening design procedures\(^{32,40,41}\) use a measure of the deflected shape of the pavement's surface to indicate the present condition of the pavement structure.

An example is the SHELL pavement evaluation and overlay design procedure\(^{32}\) based upon pavement condition measurements made with a Falling Weight Deflectometer.
Figure 3.14 Relation between Standard Deflection and Life for Pavements with Granular Non-Cementing Road Bases
(after Kennedy and Lister) (35)
A Deflection Interpretation Chart, Figure 3.15 has been developed to show the theoretical relationships between the modulus of the upper bound layer $E_1$, the maximum deflection $d_0$, the curvature ratio $Q_r$ (details given in next chapter) and thickness of the bound layer $h_1$ for predetermined values of the intermediate unbound layer $E_2$, the thickness of the unbound layer $h_2$, the modulus of the subgrade $E_3$ and the distance $r$ of the second deflection measurement from the load point for a given test load.

The use of this chart, together with the measured values of maximum deflection $d_0$ and curvature ratio $Q_r$ results in an estimate of the thickness of the upper bound layer $h_1$ and the modulus of the subgrade $E_3$, if the other variables are known or can be calculated.

The values of $h_1$ and $E_3$ can then be used to estimate the remaining life and, if necessary, design an overlay to extend the remaining life.

3.7 LIMITATIONS OF THE PARAMETER MAXIMUM DEFLECTION

The derivation of empirical deflection and performance relationships clearly indicates that maximum deflection can be used as a measure of the structural condition of a flexible pavement. Indeed, an evaluation and strengthening design procedure has been developed on the basis of these relationships and in most cases a pavement is regarded as having sufficient strength if the deflection measured under a test load exceeds an empirically determined value related to the traffic expected.

There is, however, a limit to the amount of information that can be determined from a single parameter such as maximum deflection.

Consider the simple example illustrated in Figure 3.16.
Figure 3.15 Deflection Interpretation Chart
(after Claessen, et al) (32)
It can be seen from this figure that the same value of maximum deflection will be measured for two quite different shapes of deflection bowl.

Investigations have shown that the overall deflected shape of the pavement's surface is dependent upon the material properties and thicknesses of the layers and properties of the subgrade. Clearly the two deflection bowls are associated with different material properties, a feature that cannot be reflected by the value of the maximum deflection.

It follows that because maximum deflection is a single parameter, it cannot be used to identify whether a change in its value is due to a change in the pavement material properties or the subgrade properties, i.e. a reduction in either would cause an increase in the measured value of maximum deflection.

Therefore, any assessment of pavement condition using maximum deflection can only be made in terms of the relative magnitude of the deflection measurements; high deflections indicating weak areas and low deflections indicating strong areas. No indication can be given as to where the weakness lies.

A further complication arises with the use of maximum deflection as a measure of pavement strength, in that although it is responsive to changes in pavement strength, it is also most sensitive to changes in subgrade strength.

In conclusion, maximum deflection has the advantages of being relatively simple to measure, a large amount of experimental data now exists, and that a strong correlation has been found between the level of deflection and overall pavement performance. (36)

Its disadvantages are that as a single parameter it does not reflect the overall deflected shape and, as such, cannot differentiate between changes in the pavement properties and changes in the subgrade properties.

An alternative parameter is therefore required which is more closely
related to the performance of the pavement and less influenced by the subgrade.

The following chapter explores the possibility of the use of the deflected shape of the pavement's surface for this purpose.
ASSESSING PAVEMENT CONDITION IN TERMS OF THE CURVATURE OF THE PAVEMENT'S SURFACE

4.1 INTRODUCTION

Curvature has been adopted as an indicator of pavement condition because, as a measure of the overall deflected shape of the pavement's surface, it is a more accurate representation of the true response of the pavement layers and the subgrade to the applied load.

Curvature is not a replacement of maximum deflection as an indicator of pavement condition, but is essentially an extension of it.

Details are given of the various pieces of equipment developed to measure the deflected shape. The equipment ranges from that capable of defining the shape in the vicinity of the maximum deflection to that capable of defining the total deflection dish.

Two distinct methods have been used to represent the curvature of the pavement's surface; the method adopted depending upon the equipment used. Curvature has either been expressed as a single value based upon the maximum deflection, or as a series of deflection measurements including the maximum deflection.

The advantage of the multi-value curvature measurement is that it can be used to separately estimate change in the properties of either the pavement layers or the subgrade.

The single value curvature measurement cannot distinguish between the two, because change in the deflected shape is expressed only in terms of the relative magnitude of the measurements, i.e. a large radius of curvature would indicate a strong pavement/subgrade combination and a small radius of curvature would indicate a weak pavement/subgrade combination.

Details are given of the practical studies which have related curvature, expressed as a single value, to the properties of the upper bituminous bound layer, and theoretical studies, which have confirmed this relationship by showing that curvature, unlike
maximum deflection, is more influenced by the properties of the 
pavement layer than the properties of the subgrade.

Flexible pavement evaluation and overlay design procedures (47,48,49) are also outlined, based upon theoretically derived relationships, between two pavement condition measurements (maximum deflection and single value curvature measurements) and pavement performance. These procedures can be used to separately estimate the properties of the pavement and the subgrade only if measurements are made of both the maximum deflection and curvature (expressed as a single value).

Information is also given of an alternative method for separately estimating the properties of the pavement and the subgrade from 'multi-value' curvature measurements recorded with a Dynaflect. In this method (49) the shape in the vicinity of the maximum deflection is linked to the properties of the upper bound layer, and the shape at distance from the maximum deflection is linked to the properties of the subgrade.

The potential of the curvature measurement to accurately define the overall deflected shape, in such a way as to make it possible to separately estimate the condition of the pavement and the subgrade, is of particular importance because not only can weakness be detected, but an indication can also be given of where the problem lies.

The ability to identify the position of weakness from a measure of the deflected shape of the pavement's surface will ultimately lead to a more effective use of the resources available for pavement strengthening.

4.2 DEFLECTED SHAPE OR CURVATURE OF THE PAVEMENT'S SURFACE

To overcome the limitations of the measurement of maximum deflection it is necessary to adopt a parameter:

(i) capable of defining accurately the deflected shape of the pavement's surface;
of a form that makes it possible to detect the effect of changes in either of the properties of the pavement or the subgrade.

The obvious choice to satisfy condition (i) is a direct measure of the deflected shape of the pavement's surface.

![Figure 4.1 The Deflected Shape or Curvature of the Pavement's Surface](image)

Various pieces of equipment have been designed to measure the deflected shape of the pavement's surface and a review of these follows. Full descriptions are only given for the equipment not previously mentioned in the section detailing the deflection measuring equipment. The same distinction between loading techniques is made by separating the curvature measuring equipment into the two categories of Rolling Wheel and Stationary Loading Techniques.

A number of different methods have been suggested, by the various authors, for defining curvature related to the method adopted and equipment used to measure the deflection dish.

4.3 EQUIPMENT USED TO MEASURE THE DEFLECTED SHAPE

4.3.1 Rolling Wheel Techniques

4.3.1.1 The Deflection Beam

An early review of the possible use of curvature as an indicator of
A pavement condition was presented by Kung in 1962, who discussed the ideas of several investigators and also suggested a 'slope of deflection' method for assessing pavement performance from deflection measurements made with a Benkelman Beam.

The 'slope of deflection' was defined as the tangent of the angle made by the original pavement surface and the extension of the straight line connecting the point of inflection of the deflection curve and the point of maximum deflection, i.e., \( \tan \theta = \frac{cd}{bc} \) - Figure 4.2.

![Figure 4.2 Defining the 'Slope of Deflection'](image)

Reference was made to:

(i) the Bending Index (proposed by Carey and Benkelman) 
\[ b = \frac{cd}{ac} \] where 'cd' was the deflection and 'ac' was one half of the total deflected length;

and

(ii) a similar idea by Ford and Bisselt\(^{(43)}\), who calculated the ratio 'cd' to 'ac' defining 'ac' as the 'radius of influence', a horizontal line from the point of maximum deflection to the point where the curve became tangential to the horizontal.

Kung quotes Ford and Bisselt\(^{(43)}\) as stating that "the deflection of a pavement alone was not sufficient information to indicate pavement performance. The ratio of the 'radius of influence' to deflection can be used as a criteria for overall pavement performance".
The two concepts 'one half of the deflected length' and the 'radius of influence' were adjudged to be identical by Kung and were, in his opinion too long in length because it was observed that deflection increased slowly until the wheel was within a certain distance from the measuring tip of the Benkelman Beam and then increased rapidly to the maximum point. Kung concluded from this that his idea of 'Slope of Deflection', expressed as the ratio $\frac{cd}{bc}$ was therefore more appropriate.

The deflected shape was measured by a Benkelman Beam and was characterised by the maximum deflection and the deflection at a point two feet away from the point of maximum deflection. It was suggested that a point one and a half feet away from the point of maximum deflection was more appropriate for lighter test loads.

A Benkelman Beam\(^{(44)}\) has also been used in South Africa to measure the deflected shape. The technique was also used to measure the deflection recorded as the truck stopped briefly every six inches of its travel for a distance of four feet on either side of the point of measurement. The resulting deflection was plotted and the radius of curvature determined graphically by fitting a curve to the point of maximum deflection. This method of measuring curvature was in theory very accurate, but in practice proved difficult because of the need to stop the truck at exactly six inch intervals. Both the field measurements and the analysis of the observations took a considerable time.

The Benkelman Beam has also been used in the United Kingdom and France to measure the deflected shape. In both instances the equipment was used in conjunction with velocity transducers to measure the curvature, and a chart recorder to plot the deflected shape. This was achieved by fitting the Benkelman Beam with an electronic transducer connected to the Y axis of a recorder with cartesian co-ordinates, and connecting the X axis to a rotary potentiometer activated by the movements of the wheels of the truck.

More recently Leger and Autret\(^{(45)}\) have recorded the deflected shape of the pavement's surface using a Benkelman Beam linked up to a chart...
The method adopted for analysing the deflected shape was to fit to the recorded curve an analytical curve, which approached the shape of the deformation curve more closely than a parabola. It was suggested this method of analysis resulted in the two peaks being fitted closer together, and the radius of curvature was calculated from the analytical curve.

The curve adopted was of the form

\[ d(x) = \frac{d_0}{1 + ax^2} \]

The authors remarked that the matchings obtained with this curve were in general excellent up to some 400 or 500 mm from the peak of the deformation curve.

The matchings were generally performed numerically by a regression calculation in the axes \( Y = \frac{1}{d} \) and \( X = x^2 \), in which the matching curve was a straight line. Once 'a' and 'd_0' were determined the radius of curvature was obtained using the expression

\[ 2Rd_0 = \frac{1}{a} \]

4.3.1.2 Curvature Meter

A Curvature Meter\(^4\) was used in South Africa to measure deflected shape and was based on the principle that the differential deflection occurring over a short distance is related to the curvature of the surface over that distance.

The equipment consisted of a short bar resting on the road surface at both ends, with a centrally mounted dial gauge, the spindle of which was also in contact with the road surface. (Figure 4.3)
The instrument was placed five feet ahead of the dual rear wheels of a loaded truck and aligned, using pointers permanently attached to the truck body. The truck was then driven slowly forward and the maximum reading recorded as the wheels passed astride of the instrument. The Curvature Meter recorded both positive and negative maximum deflections as the dual wheels were driven up to and passed it. (Figure 4.4).
The relation between curvature and differential deflection was deduced by fitting an appropriate curve to the three points defined by the instrument, (the maximum deflection and the two points of inflection).

Dehlen\(^{(44)}\) suggested that in practice the relation was almost identical to that for a circle. However, previous work undertaken in South Africa with the Benkelman Beam indicated that in the vicinity of the points of maximum deflection, the curves were typically of a sine form. The relation between curvature and differential deflection in the case of a sine curve is

\[
R = \frac{L^2}{Fd}
\]

where 
- \(d\) is the differential deflection
- \(L\) is the distance between the dial gauge and each support
- \(F\) is the factor which varies between 2.0 and 2.47 as the ratio \(L/S\) varies between 0 and 1. (See Figure 4.4).

It was reported\(^{(44)}\) that satisfactory results were obtained based on a sine curve analysis of the results. It was shown that for accurate results, the instrument should measure differential deflections within a distance of 12 inches between the points of inflection. With this in mind an instrument length of 10 inches was adopted.

Dehlen\(^{(44)}\) dismisses the possibility of defining a curvature or deflection index on the basis that the deflection at the suggested distance away from the maximum deflection was outside the range over which the sharpest curvature occurs. The main disadvantage with the Curvature Meter was the need to use either a telescope or a pair of binoculars to read the dial gauge while sitting on the road behind the vehicle.

Use of this equipment in the United Kingdom\(^{(46)}\) has indicated that \(d\) has a range of 0.001 inches to 0.002 inches on motorways, and up to 0.015 inches on roads of light construction.
Assuming the road surface approximates to a sine curve, the radius of curvature range for the motorways would be 900 to 450 feet and a limiting value of 60 feet on light construction.

4.3.2 Stationary Loading Techniques

Detailed descriptions of the Dynaflect, the Falling Weight Deflectometer and the Road Rater have been given earlier in Chapter 3.

4.3.2.1 The Dynaflect

Interpretation of the deflected shape measurements from the 5 geophones mounted on the tow bar of the Dynaflect is in terms of the spreadability of the deflected basin,\(^{(48, 49)}\) Figure 4.5.

![Figure 4.5 Recording Curvature with a Dynaflect](image)

Spreadability is the average deflection expressed as a percentage of the maximum deflection.

\[
\text{Spreadability} = \frac{d_{\text{max}} + d_1 + d_2 + d_3 + d_4}{5 d_{\text{max}}} \times 100\%
\]

where \(d_{\text{max}}\) is the maximum deflection of the pavement, \(d_1, d_2, d_3\) and \(d_4\) are the deflections at 1, 2, 3 and 4 feet from the centre of the applied load.
4.3.2.2 The Falling Weight Deflectometer

The Falling Weight Deflectometer measures the deflected shape of the pavement's surface by means of geophones, one in the centre of the loaded area \((d_0)\) and one at a fixed distance \(r\) from the load \((d_r)\). The curvature of the pavement's surface was represented by the ratio \(Q_r\) of the deflection at a distance \(r\) from the load \((d_r)\) to the deflection under the centre of the load \((d_0)\). Claessen, et al.\(^{(32)}\) states that the ratio \(Q_r\) was chosen instead of the radius of curvature because \(Q_r\) can be measured more easily and provides equivalent information.

For typical structures, \(r\) has been fixed at 600 mm and the curvature expressed as the \(Q_{600}\) ratio.

4.3.2.3 The Road Rater

The Road Rater serves as a means of measuring a road's strength by the application of a dynamic load superimposed over a static load.

The road's deflection profile, measured by four velocity sensors located on a boom and spaced 0.31 m apart, is visually displayed on instrumentation located inside the vehicle.

The deflection readings are noted either manually or automatically at each particular test point.

4.4 DEFINING CURVATURE

Two distinct methods have been used to define the deflected shape of the pavement's surface:

(i) expressing curvature as a single value, such as the 'slope of deflection', radius of curvature, spreadability and curvature ratio, based on the value of the maximum deflection.

(ii) expressing curvature as a series of deflection measurements
consisting of the maximum deflection and several other deflection readings.

4.4.1 Single Value Curvature Measurements

The slope of deflection and radius of curvature measurements have the disadvantage that they do not always define the total shape of the deflection bowl. An exception is the radius of curvature measurement derived from the readings made with the Curvature Meter, which is based upon the points of inflection as well as the value of the maximum deflection.

(48,49) Although the Dynaflect can characterise the overall deflected shape by a series of five deflection readings, curvature is usually expressed in terms of the spreadability \( S_p \).

\[
S_p = \frac{d_{\text{max}} + d_1 + d_2 + d_3 + d_4}{5d_{\text{max}}}
\]

The spreadability concept defines the shape as an average deflection expressed as a percentage of the maximum deflection.

The two deflection measurements made with the Falling Weight Deflectometer are expressed as a Curvature ratio \( Q_r \). \( Q_r \) is the ratio of the deflection \( (d_r) \) at a distance \( r \) from the load to the deflection \( (d_0) \) under the centre of the load.

All of these methods define curvature as a single value with the result that change in the overall shape of the pavement's surface is reflected in the relative magnitude of these single values (c.f. max. defl.).

The interpretation of the relative magnitudes of the single value curvature measurements depends to a large extent upon the way in which they define the deflected shape. However, in general a high curvature value would be interpreted as indicating a strong pavement/subgrade combination and a low value as indicating a weak pavement/subgrade combination. The deflected shapes involved with these two extremes are shown in Table 4.1.
Table 4.1 Interpreting the Curvature Measurements

<table>
<thead>
<tr>
<th></th>
<th>Strong Pavement/Subgrade Combination</th>
<th>Weak Pavement/Subgrade Combination</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristics:</td>
<td>wide shallow dish.</td>
<td>narrow deep dish.</td>
</tr>
<tr>
<td>Curvature Meter</td>
<td>Large radius of curvature.</td>
<td>Small radius of curvature.</td>
</tr>
<tr>
<td>Deflection Beam</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynaflect</td>
<td>High Spreadability value.</td>
<td>Low Spreadability value.</td>
</tr>
<tr>
<td>Falling Weight</td>
<td>High Curvature Ratio value.</td>
<td>Low Curvature Ratio value.</td>
</tr>
<tr>
<td>Deflectometer</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

These generalisations are based upon the assumption that the deflection sensing devices are positioned at the same distance apart for both measurements.

4.4.2 Multi Value Curvature Measurements

Both the Dynaflect and a modified Deflectograph (details given later in this report) can record the deflected shape as a series of deflection measurements. In this case reliance is not placed on the relative magnitude of a single value to give an indication of the deflected shape.

The advantage of this type of curvature measurement is that it can be analysed to give information on the strength of the pavement and subgrade separately, in addition to an indication of the overall strength of the pavement/subgrade combination.
4.5 RELATING CURVATURE, DEFINED AS A SINGLE VALUE, TO PAVEMENT PERFORMANCE

Several investigators \( (42, 44, 48, 32) \) have expressed the curvature of the pavement's surface as a single value; Kung \( (42) \) in terms of the slope of deflection, Dehlen \( (44) \) in terms of the radius of curvature, Vaswani \( (48) \) in terms of the spreadability and Claessen \( (32) \) in terms of the curvature ratio.

Practical studies \( (42, 44) \) have shown that curvature, as a single value, can be related to the performance of the upper bound layer of a flexible pavement. Theoretical studies \( (51) \) have indicated that curvature, unlike maximum deflection is more related to the properties of the pavement layers than the properties of the subgrade.

Curvature, again as a single value, but this time in conjunction with maximum deflection (also a single value) has been theoretically related to the performance of both the pavement and the subgrade.

These relationships have been incorporated into flexible pavement evaluation and strengthening design procedures \( (47, 48, 32) \) capable of being used to separately estimate the properties of both the pavement and the subgrade from measurements of maximum deflection and curvature.

4.5.1 Practical Studies

Both Dehlen \( (44) \) and Kung \( (42) \) have shown that curvature, defined as a single value, can be related to the performance of the upper bound layer, from observations on actual pavement structures.

Dehlen \( (44) \) used both maximum deflection and the radius of curvature at the point of maximum deflection to investigate the problem of 'chicken net' cracking, and concluded that a correlation existed between the condition of the surfacing and both the maximum deflection and the radius of curvature. The existence of these relationships were taken by Dehlen \( (44) \) as an indication that 'chicken net' cracking was due to excessive flexure of the bituminous surfacing.
Kung\textsuperscript{(42)} also used a measure of curvature to investigate the incidence of cracking in flexible pavements. Curvature, expressed in terms of the 'Slope of Deflection' was defined as the tangent of the angle made by the original pavement's surface, and the extension of the straight line connecting the point of inflection and the maximum deflection, Figure 4.6.

\[ \text{Slope of Deflection} = \frac{ac}{ba} \] where \( \text{ac} \) is the maximum deflection.

Figure 4.6 Defining the 'Slope of Deflection'

Kung concluded from the field tests undertaken that the 'Slope of Deflection' method showed a high degree of accuracy for indicating pavement cracking.

4.5.2 Theoretical Studies

Dehlen\textsuperscript{(44)} used Odemark's theory\textsuperscript{(50)} to investigate the effect of a change in Young's modulus of any of the pavement layers on the deflection and curvature measured at the pavement's surface. The results, presented as Table 4.2, shows that while the value of maximum deflection was dependent to a large degree on the Young's moduli of the materials at depth, in addition to those of the upper layers, the radius of curvature was dependent mainly on the moduli of the upper layers of construction and very little on those of the materials at depth. Dehlen\textsuperscript{(44)} used these findings to substantiate the use of curvature as an indicator of excessive stresses in the upper bound layer giving rise to flexure cracking.

Huang\textsuperscript{(51)} identified deformation of the subgrade and fatigue cracking
Percentage change in Deflection or Radius of Curvature at surface resulting from a 50 per cent change in the Young's modulus of the layer indicated.

<table>
<thead>
<tr>
<th>LAYER</th>
<th>THICKNESS</th>
<th>CASE 1b</th>
<th>CASE 2b</th>
<th>CASE 3b</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Δ</td>
<td>Δ</td>
<td>Δ</td>
</tr>
<tr>
<td>Surface and base</td>
<td>200 mm</td>
<td>16</td>
<td>14</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td></td>
<td>38</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>Sub-base</td>
<td>200 mm</td>
<td>11</td>
<td>9</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>13</td>
<td>14</td>
</tr>
<tr>
<td>Subgrade</td>
<td>Semi infinite</td>
<td>25</td>
<td>28</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>5</td>
<td>4</td>
</tr>
</tbody>
</table>

b computed for the following Young's moduli of the layers (base, sub-base, and sub-grade): CASE 1 - 10:2:1; CASE 2 - 10:5:1; CASE 3 - 4:2:1.

TABLE 4.2 Dependence of Deflections and Curvatures on Stiffnesses of Materials in Upper and Lower Layers of Foundations

(after Dehlon)(44)
as the two main causes of pavement failure. A method of precluding such failures was to consider the pavement as a three layer elastic system and limit the vertical compressive strain on the subgrade and the horizontal tensile strain at the bottom of the asphalt bound layer. Measurement of these strains in the field was difficult and as an alternative, Huang determined relationships between deflection, curvature, compressive strain on the subgrade and tensile strain in the asphalt layer, so that deflection and curvature could be used as criteria for pavement design and evaluation.

One of the main conclusions reached was that, 'Curvature was definitely related to the tensile strain in the asphalt layer. The curvature-tensile strain ratio depended primarily on the surface course thickness and was independent of roadbase thickness. The use of curvature, instead of deflection, as a criterion for controlling fatigue, was highly desirable because the curvature tensile strain ratios were not affected by modular ratios'.

The findings of were similar to those of Dehlen and suggest that curvature is related to the properties of the upper bound layer of a flexible pavement.
4.5.3 Flexible Pavement Evaluation and Strengthening Design Procedures

Elastic theory has shown that curvature is greatly influenced by pavement properties and that maximum deflection is greatly influenced by subgrade properties. Several investigators have made use of these findings and incorporated theoretically derived relationships, between maximum deflection and pavement/subgrade strength, and curvature and pavement/subgrade strength into flexible pavement evaluation and strengthening design procedures.

These deflection and curvature relationships are usually expressed in chart form and can be used to estimate the properties of the upper bound layer (in terms of either the thickness or the modulus) and the modulus of the subgrade, only if measurements are made of both the maximum deflection and curvature.

These derived strength parameters can then be used to estimate the remaining life of a pavement and in some cases to design an overlay to extend the life of a pavement.

4.6 RELATING CURVATURE, DEFINED AS A MULTI VALUE MEASUREMENT, TO PAVEMENT PERFORMANCE

The Dynaflect and a modified Deflectograph are the only two pieces of deflection measuring equipment capable of defining the deflected shape as a series of deflection measurements.

A method has been proposed for separately estimating the properties of the pavement and the subgrade from multi value curvature measurements recorded with a Dynaflect. The method is based upon theoretical relationships between:

(i) the deflected shape in the vicinity of the maximum deflection and the properties of the upper bound layer.

(ii) the deflected shape at distance from the maximum deflection and the properties of the subgrade.
This evaluation procedure is different to those outlined previously because it is based upon the analysis of the deflected shape characterised by a number of deflection measurements, and not on relationships between two pavement strength parameters defined as single values.

4.6.1 An Evaluation and Strengthening Design Procedure based upon Multi Value Curvature Measurements

The studies conducted by De Kiewit et al. are of particular interest because of the way in which the curvature measurements, recorded by the Dynaflect, were interpreted to separately indicate the properties of the pavement and the subgrade.

Curvature measurements recorded by a Dynaflect (Figure 4.7), are usually expressed in terms of the single value spreadability, Sp.

\[
\text{Spreadability} = \frac{d_1 + d_2 + d_3 + d_4 + d_5}{5d_1} \times 100\%
\]

Figure 4.7 Defining Spreadability

However, De Kiewit et al. not only presented the curvature measurements in this way, but also in three additional forms; a maximum deflection plot, a surface curvature index (SCI) plot, and a base curvature index plot (BCI) plot. The SCI value was obtained by subtracting the \(d_2\) reading from the \(d_1\) reading and the BCI value by subtracting the \(d_5\) reading from the \(d_4\) reading.

De Kiewit, et al. reported the SCI value as giving an indication of the pavement surface properties and the BCI value applied to the
subgrade properties. The $d_5$ deflection reading was also said to have good correlation with the subgrade properties, and less dependent on the thickness and the stiffness of the pavement layers. The spreadability value gave information on the modular ratio of the surface layer and subgrade.

This evaluation and strengthening design procedure checks the spreadability, maximum deflection, SCI and BCI data statistically for significant difference using a computer programme. On the basis of this data the programme tests every pavement section against every other pavement section to determine which sections are significantly different. These sections are then considered separately for the remainder of the design.

4.6.2 Relating the Deflected Shape, as defined by a Multi Value Curvature Measurement, to the Properties of both the Pavement and the Subgrade.

De Kiewit, et al, have proposed a method for separately estimating the properties of the pavement and the subgrade from an analysis of the actual deflected shape of the pavement's surface, using the relationships derived by Majidzadeh. (52)

Fundamental to this evaluation procedure is the requirement that the deflected shape be defined by a sufficient number of deflection measurements, to allow the sections of the deflection bowl, influenced by the properties of the pavement and the subgrade, to be identified. In practice, this was achieved by De Kiewit, et al, (49) using a Dynaflect which records the deflected shape as a series of five deflection measurements.

The method of interpretation proposed by Majidzadeh (52) is presented in diagrammatic form in Figure 4.8. The response of a pavement structure to a given load is outlined, together with the sections of the overall shape suggested as being influenced by the properties of the upper bound layer and subgrade, respectively. Reference to Figure 4.8 indicates that the shape in the vicinity of the maximum deflection was suggested as being related to the properties of the
Figure 4.8 Deflection Basin Parameters
(after Majidzadeh) (52)
upper bound layer and the shape at distance from the position of maximum deflection was suggested as being related to properties of the subgrade.

The fact that Majidzadeh used a two layer elastic model to represent pavement response meant that he was only able to identify the sections of the overall deflected shape influenced by the properties of the upper pavement layer and the lower subgrade layer. The suggestion that the shape in the vicinity of the maximum deflection can be used as an indication of the strength of the upper bound layer is supported by the findings of Dehlen (44) and Kung. (42) Both investigators related curvature, represented as a single value, to the condition of the upper bound layer and stated that the evidence suggested that curvature could be used to indicate the high stress levels associated with excessive flexure of the upper bound layer.
AN ALTERNATIVE TO CONVENTIONAL TECHNIQUES FOR ESTIMATING THE THICKNESS OF BITUMINOUS MATERIAL IN A FLEXIBLE PAVEMENT STRUCTURE

The previous chapters have given details of the way in which measurements of the deflected shape of the pavement's surface have been used to indicate the present condition of flexible pavement structures.

In several countries measurements of maximum deflections and/or curvature have been used, together with empirically or theoretically derived relationships to estimate the remaining life in a pavement and, in some instances, to design overlays to extend the life.

5.1 THE FLEXIBLE PAVEMENT EVALUATION AND OVERLAY DESIGN PROCEDURE FOR USE IN THE UNITED KINGDOM

In the United Kingdom the present flexible pavement evaluation and strengthening design procedure is based upon empirically derived relationships between deflection and performance developed from measurements made over a 20 year period on the full-scale design experiments. (Details of the background and development of this method are given in Part 2 of this report). The basic inputs into the procedure are measurements of the maximum deflection recorded by a Deflection Beam or a Deflectograph.

At present most Deflectographs only measure and record the maximum deflection response of a pavement. The deflection measurements collected are subsequently processed using computers that are normally remote from the road being surveyed.

A more suitable approach would be for the deflection measurements to be processed on the machine as they are recorded, to allow immediate identification of suspect areas. Provision of real time analysis would encourage closer contact between those taking the measurements and those responsible for making decisions on the basis of the measurements.
A requirement of an on-board processing system would be an assessment of the thickness of the bituminous layer of the pavement being surveyed.

An indication of the thickness of the bituminous layer is important because bitumen is temperature susceptible and the degree of temperature susceptibility is a function of the total pavement stiffness contributed by the bound layer.

The TRRL structural maintenance design method incorporates a number of temperature and deflection relationships for pavements of various bituminous layer thicknesses, which can be used to standardise deflection measurements obtained at any pavement temperature within the range 5 - 30°C to equivalent values at 20°C.

A knowledge of the thickness of the bituminous layer means that the appropriate relationship can be selected.

5.1.1 Present method of determining the construction information

Construction information is usually obtained by coring and from trial pits.

However, due to the present economic climate, the funds available to the Local Authorities for investigative work falls far short of that required with the result that cores are generally spaced at wide intervals; \( \frac{1}{2} \) km or 1 km intervals. It is also noticeable that the number of trial pits opened on a routine basis is in many cases minimal, particularly on the roads for which the Local Authority is directly responsible. The outcome is that a great many assumptions are at present being made about the construction of the pavement between the cores.

The assumptions may be valid on newly constructed designed roads where construction thicknesses are nominally the same for the length of the scheme, but they are certainly not valid for lightly trafficked undesigned roads, where the present construction thickness is the product of years of strengthening work.
An indication of the variation in the thickness of the bituminous material on a number of lightly trafficked test sites in Devon and Somerset can be obtained from Table 11.5, Chapter 11.

What is required, therefore, is a method capable of giving a continuous estimate of the thickness of the bituminous material in a flexible pavement structure.

5.2 THE EQUIPMENT TO RECORD THE DEFLECTED SHAPE OF THE PAVEMENT'S SURFACE

For obvious reasons it would be totally impracticable to use conventional destructive techniques to give a continuous estimate of the thickness of the bituminous material. The equipment would therefore have to be non-destructive and as such be capable of measuring the response of the pavement structure to a given input.

Non-destructive equipment has already been developed to record the deflected shape of the pavement's surface.

Reference to Chapters 3 and 4 will show that several pieces of equipment are capable of continuously recording deflected shape measurement at predetermined intervals along the road.

The use of equipment of this type could be a possibility if its measure of deflected shape could be related to the thickness of bituminous material in the pavement structure.

5.3 RELATING DEFLECTED SHAPE MEASUREMENTS TO THE CONDITION OF THE PAVEMENT

The use of maximum deflection and curvature as indicators of the condition of flexible pavements has also been outlined in Chapters 3 and 4.

The measurement of maximum deflection has several disadvantages.
The most important in the context of its possible use as an indicator of the thickness of the bituminous material are that it is particularly sensitive to the strength of the subgrade and that as a single measurement it cannot differentiate between changes in the pavement properties and changes in the subgrade properties.

In contrast, the measurement of curvature has the distinct advantage that it can be used to reflect the condition of the upper pavement layers without being influenced to any great extent by the strength of the subgrade.

Another advantage of curvature is that as a multivalue measurement, it can be used to separately estimate the properties of the upper pavement layers and the subgrade. This is significant because it would suggest that measurements of curvature could be used to give an indication of the thickness of the bituminous material, particularly if it were recorded as a multivalue curvature measurement.

5.4 MEASURING THE DEFLECTED SHAPE WITH A DEFLECTOGRAPH

A Deflectograph has been modified by TRRL to record the deflected shape of the pavement's surface as a maximum deflection and ten ordinate deflections; i.e. a multivalue curvature measurement, Figure 5.1.

Figure 5.1 Recording the Deflected Shape as a Maximum Deflection and ten Ordinate Deflections
Recording the deflected shape in this way means that the curvature of the pavement's surface can be expressed as a differential deflection and that the magnitude of the differential deflection can be used to give an indication of the condition of the pavement structure.

Differential deflection is calculated as the difference between the maximum deflection and an ordinate deflection and is specified in terms of the horizontal distance between the two points of measurement, i.e. the differential deflection 50 mm from the maximum deflection relates to the shape of the deflection dish over that distance, Figure 5.2.

For a given deflected shape the differential deflection will increase in magnitude with increase in horizontal distance away from the point of maximum deflection. It is therefore important that curvature measurements expressed as differential deflections should only be compared if they refer to the deflected shape over the same horizontal distance.

The concept behind the use of differential deflection as a measure of curvature is that the condition of the pavement will be reflected in the magnitude of a differential deflection at a specific distance from the point of the maximum deflection. This can best be illustrated
by the two extreme cases shown in Figure 5.3.

Figure 5.3(a) Wide, Shallow Deflection Dish

Figure 5.3(b) Narrow, Deep Deflection Dish

Figure 5.3(a), defined by a small differential deflection, indicates very little change of shape in the vicinity of the maximum deflection characteristic of a wide, shallow deflection dish associated with pavements having a weak subgrade. Figure 5.3(b), defined by a large differential deflection, indicates rapid change of shape in the vicinity of the maximum deflection indicative of the narrow, deep deflection dish associated with pavements having weak surfacing layers.

5.5 USING DEFLECTED SHAPE MEASUREMENTS TO ESTIMATE THE THICKNESS OF THE BITUMINOUS MATERIAL

(42,44) (49)

Several investigators, including De Kiewit, have shown that the deflected shape in the vicinity of the maximum deflection can be used as an indicator of the strength of the upper bound layer.

The response of a particular layer in a flexible pavement structure to a given load is dependent upon both the thickness and the modulus of the material in that layer and, to a certain extent, upon the
degree of support given by the underlying layers.

To interpret the deflected shape measurements, as indicators of the thickness of the bituminous layer, will require assumptions to be made about the modulus of the bituminous material and the degree of support from the underlying layers; the degree of support being governed primarily by the modulus and thickness of the individual layers.

The following chapter describes investigations into the use of deflected shape measurements recorded by a Deflectograph, to estimate the thickness of bituminous material present in a flexible pavement structure. The development of a method capable of giving a near continuous estimate of the thickness of the bituminous material would go a long way towards removing one of the major obstacles standing in the way of the automatic processing of deflection measurements on board the Deflectograph. The use of the deflected shape measurement recorded by the Deflectograph, to estimate the thickness of bituminous material, would be a neat solution to the problem, because it would mean that all the necessary information is being recorded by one piece of equipment. Even if this aim is never reached, such a method would help to eliminate a great deal of the uncertainty that exists at present, concerning the pavement construction between locations where cores have been removed.
6.1.1 Theoretical Studies

6.1.1.1 The pavement model

The finite element model developed represents a longitudinal pavement section loaded at its mid-point by a load equal to that applied to the road surface by each dual wheel assembly of the Deflectograph.

The model was sub-divided into a number of rectangular elements connected at their nodal points. A graded division into elements was adopted to allow a more detailed study in the region under the load.

Boundary conditions were such that the bottom of the model was constrained from moving in the horizontal and vertical directions,
the sides constrained in the horizontal direction only, and the surface of the model free to move in both directions. The external load acting on the actual pavement structure was replaced by an equivalent system of forces acting at the element nodes.

6.1.1.2 Calibrating the model

Before the finite element model could be used to investigate a relationship between deflected shape and thickness of the bituminous layer, it was necessary to ensure that the response of the theoretical model was similar to that of an actual pavement structure.

Absolute deflected shapes measured with displacement gauges on two pavement structures, one with a granular roadbase and one with a bituminous roadbase, were used to define the pavement response to be modelled.

A theoretical model was developed that accurately predicts the response under load of pavement structures with granular and bituminous roadbases.

6.1.1.3 Prediction of bituminous layer thickness from absolute deflected shape measurements.

A relatively simple relationship between deflected shape and bituminous layer thickness was developed that would have formed the basis of a bituminous layer thickness estimation procedure, if it could have been assumed that the strengths and thicknesses of the other pavement layers and subgrade remained constant along a length of road.

Variations in these values do occur in practice and it was therefore necessary to quantify the possible effect of each likely change on the deflected shape.

Analysis of the absolute deflected shapes produced by the model has resulted in the construction of a chart showing the relationship between curvature, defined as the differential deflection, 200 mm
from the point of maximum deflection, and the maximum deflection for pavements with various thicknesses of bituminous material.

This chart could be used in conjunction with measurements of the absolute deflected shape of the pavement's surface to estimate the thickness of bituminous material.

6.1.1.4 Prediction of bituminous layer thickness from equivalent Deflectograph deflected shape measurements

A procedure was developed to convert the predicted absolute deflected shape measurements into equivalent Deflectograph deflected shape measurements.

These measurements were then used to define a similar relationship between curvature and maximum deflection for pavements with various thicknesses of bituminous material. The relationship could be used, together with deflected shape measurements recorded with a Deflectograph, to estimate the thickness of bituminous material present in a flexible pavement.

6.1.1.5 The accuracy of the estimate of bituminous layer thickness

The accuracy of the proposed method was assessed by quantifying the effect of change in either the strength or thickness of each of the pavement layers and subgrade on the estimate of bituminous layer thickness.

The effect of soft spots in the granular sub-base, and cracking in the bituminous layers, on the estimate of bituminous layer thickness was also studied using the model.

6.1.2 Practical Studies

In situ deflected shape measurements made with a Deflectograph on a wide range of pavement types were used to develop an empirical relationship between curvature and maximum deflection.
6.1.2.1 Selecting suitable lengths of road on which to record the
deflected shape measurements

Lengths of the existing test sites were used to record the deflected
shape measurements. They were selected because the information
from the coring programme indicated a relatively constant thickness
of bituminous material.

6.1.2.2 Correcting the deflected shape measurements for the effect
of temperature on the stiffness of the bituminous material

Prior to the analysis all deflected shape measurements were temperature
corrected to equivalent values at 20°C using previously derived
relationships between deflection and temperature. (35)

6.1.2.3 Deriving an empirical relationship between curvature and
maximum deflection for pavements with various thicknesses of
bituminous material

The in situ deflected shape measurements were analysed using linear
regression techniques to develop an empirical relationship between
curvature, defined as a differential deflection, 200 mm from the point
of maximum deflection, and maximum deflection for pavements with
various thicknesses of bituminous material. The results obtained
covered a range of pavement types which in general were substantially
different from those used to develop the model.

These empirical relationships were similar to those derived theoretic­
ally, but their numerical values differed due mainly to the differences
between the pavement structures modelled and those surveyed with the
Deflectograph.

These empirical relationships form the basis of a bituminous layer
thickness estimation chart, which could be used in conjunction with
deflected shape measurements recorded with a Deflectograph to estimate
the thickness of bituminous material present in a typical lightly
trafficked pavement.
6.2 THEORETICAL ANALYSIS OF FLEXIBLE PAVEMENT STRUCTURES USING A FINITE ELEMENT APPROACH

The theoretical response of a number of flexible pavement structures, to an applied load, has been investigated using a commercially available finite element programme. The finite element method has been used by several investigators to model pavement response. No selection for use in this work was influenced primarily by the fact that it was readily available on the IBM mainframe computer.

The finite element method represents an extension of matrix methods for skeletal structures to the analysis of continuum structures. The analysis of a continuum differs from the analysis of a skeletal structure in two basic aspects only, namely in the initial subdivision into elements and in the derivation of the element stiffness characteristics.

6.2.1 Subdivision of the continuum into elements

The finite element technique requires that the continuum be subdivided into a number of artificial elements before the matrix method of analysis can be applied. The artificial elements, known as 'finite elements' or 'discrete elements', are usually chosen to be either rectangular or triangular in shape. In reality, these elements are connected together along their common boundaries. However, in order to make a solution by the matrix method of structural analysis possible, it is assumed in the case of simple elements that these elements are only interconnected at their nodes. This assumption by itself means that continuity requirements are only satisfied at the nodal points. Clearly the relaxation of continuity requirements along the sides of the elements would make the structure very much more flexible than it actually is, since it could allow gaps to form between the elements.

However, in the finite element method, the individual elements are constrained to deform in specific patterns. Hence, although continuity is only specified at the nodal points, the choice of a suitable pattern of deflection for the elements can lead to the satisfaction of some, if not all, of the continuity requirements along the sides of adjacent elements.
6.2.2 Derivation of element stiffness characteristics

The principle of virtual work is used in deriving the stiffness properties of various elements. This principle is concerned with the relationship which exists between a set of external loads and the corresponding internal forces which together satisfy the equilibrium condition, and also with sets of node displacements and the corresponding number of deformations which satisfy the conditions of compatibility.

The principle may be stated in general terms as follows:
the virtual work done by the external loads is equal to the internal work absorbed by the structure and can be expressed in mathematical terms as:

\[ \Sigma F \delta = \int \sigma \epsilon \cdot d(\text{Vol}) \]

where \( F \) refers to the system of external loads, \( \delta \) to the deflections of the loads, \( \sigma \) to the system of internal forces and \( \epsilon \) to the internal deformations of the structure.

6.2.3 The finite element programme

A commercially available finite element programme (PAFEC 75) was used to investigate the theoretical response of a number of flexible pavement structures to an applied load.

The PAFEC 75 finite element suite consists of ten separate computer programmes which, when executed sequentially, gives a complete engineering analysis. Each of these programmes is known as a PHASE of PAFEC 75 and Table 6.1 gives a synopsis of the operations undertaken by them.

Some phases are more important, as far as obtaining printed results, than others. PHASE 1, used for read-in and data expansion, is an essential part of the whole process. PHASES 4, 6 and 7 play an equally important role. These four phases form the minimum requirements for any analysis where the structure is fully defined. The remaining six phases provide the many extra facilities available, such as
<table>
<thead>
<tr>
<th>PHASE</th>
<th>SHORT DESCRIPTION</th>
<th>DETAILED DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>READ</td>
<td>Data modules are read in, default values are inserted and the modules are placed onto backing store. The NODES module is expanded so that all mid-side nodes are included.</td>
</tr>
<tr>
<td>2</td>
<td>PAFBLOCKS</td>
<td>Any PAFBLOCK data is replaced by the full nodal co-ordinate and topological description of the complete mesh of elements.</td>
</tr>
<tr>
<td>3</td>
<td>INDRAW STRUCTURE</td>
<td>The structure itself is drawn. At this stage it is not possible to show any results such as displacements, stresses or temperatures since these have not yet been evaluated.</td>
</tr>
<tr>
<td>4</td>
<td>PRE-SOLUTION HOUSEKEEPING</td>
<td>In this PHASE the constraints on the problem are considered and a number system for the degrees of freedom is derived.</td>
</tr>
<tr>
<td>5</td>
<td>INDRAW CONSTRAINTS</td>
<td>This PHASE is very similar to the PHASE 3 except the constraints which have been applied are shown. Conversely the degrees of freedom can be indicated on a drawing.</td>
</tr>
<tr>
<td>6</td>
<td>ELEMENTS</td>
<td>The stiffness (or other such as conductivity, mass etc.) matrices of all the elements are found and put onto backing store.</td>
</tr>
<tr>
<td>7</td>
<td>SOLUTION</td>
<td>The system equations are solved for displacements, temperatures or whatever happens to be the primary unknowns in the problem being tackled.</td>
</tr>
<tr>
<td>8</td>
<td>OUTDRAW DISPLACEMENTS</td>
<td>The primary unknowns in the problem (i.e., displacements or temperatures) are drawn.</td>
</tr>
<tr>
<td>9</td>
<td>STRESS</td>
<td>The stresses are found.</td>
</tr>
<tr>
<td>10</td>
<td>OUTDRAW</td>
<td>Stress contour, stress vector plots etc. are produced.</td>
</tr>
</tbody>
</table>

Table 6.1 Brief Description of the Ten Phases of PAFEC 75(53)
complete passive graphics capability, automatic mesh generation and
element stressing.

6.2.4 The Method of Analysis

The longitudinal model of the pavement structure was analysed as a
plane elasticity problem.

Plane elasticity problems involve continua that are loaded in their
plane, an assumption that could be made if the theoretical model
represented a section of the pavement structure immediately below
the dual wheels of the Deflectograph.

There are two types of plane elasticity problems, namely, plane stress
and plane strain.

In a plane stress problem the continuum, such as a plate, is generally
thin relative to the other dimensions and the stresses normal to the
plane are neglected. Typical examples of plane stress problems are
diaphragm plates in box sections and plate girder webs.

In a plane strain problem, the strain normal to the plane of loading
is assumed to be zero. A typical example of a plane strain problem
is a retaining wall, where analysis is achieved by taking a transverse
slice out of the wall and assuming zero strains normal to the wall.

Although neither of the assumptions of zero stress or zero strain in
a direction normal to the plane of loading is strictly correct, it
was felt that the latter would be a more realistic approach because
the problem was closer to that of the retaining wall than a thin
diaphragm plate in a box section.

For this reason the pavement section was analysed as a plane strain
problem.
6.2.5 The Pavement Model

The finite element model developed represented a longitudinal pavement section, 12 metres in length, loaded at its mid-point by a load equal to that applied to the road's surface by each dual wheel assembly of the Deflectograph. A distance of 12 metres was selected because it approximated to the length of road influenced by the action of the loaded wheels on all but the stiffest of pavement structures.

The symmetry of the model meant that it could be divided at the point of application of the load, because it was shown that the response of half of the model under half of the load was identical to the response of the whole model under the total load. Physically reducing the size of the model by half resulted in a great saving in the computer time required to analyse this particular problem.

The model, Figure 6.1, was sub-divided into a number of rectangular elements connected at their nodal points.

The nature of the finite element idealisation means that, in general, the accuracy of the solution increases with the number of elements taken.

It must be realised that as the number of elements taken increases, the computer time required to obtain a solution also increases, with a consequent increase in cost.

A graded division into elements was adopted to allow a more detailed study to be made in the region under the load.

Such a selective distribution of elements is efficient and can lead to economy in solution time without any loss of accuracy.

Boundary conditions were such that the bottom of the model was constrained from moving in both the horizontal and vertical directions, the sides constrained in the horizontal direction only and the surface of the model free to move in both directions.
Figure 6.1 The original pavement model

All dimensions in millimetres
The external wheel loads acting on the actual pavement structure were replaced by an equivalent system of forces acting at the element nodes. Care was taken, with the selection of the finite element mesh, to ensure that nodes occurred at the points of application of the forces.

6.2.6 Calibrating the Model

Before the finite element model could be used to investigate the ability of the deflected shape measurements to characterise the pavement construction, it was first necessary to ensure that its response was similar to that of an actual pavement structure.

Initially it was envisaged that this would be achieved by reproducing the response of a lightly trafficked pavement structure as measured by a Deflectograph.

However, experimental and theoretical studies\(^{[54]}\) have demonstrated that the deflections measured under a rolling wheel cannot be readily depicted by multi-layer elastic theory, and that inclusion of non-linear behaviour does not resolve the difference.

To overcome these problems, absolute deflections as measured with displacement gauges on two pavement structures, one with a granular roadbase and one with a bituminous bound roadbase, were obtained from instrumented road sections. These measurements, made under the loading action of a Deflectograph, defined the pavement response to be modelled.

6.2.6.1 Pavement with a granular roadbase

The pavement structure with a granular roadbase was represented by the layered system shown in Figure 6.2

<table>
<thead>
<tr>
<th>Pavement Surface</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Bound Layer (E_1 V_1)</td>
<td></td>
<td>(h_1)</td>
</tr>
<tr>
<td>Intermediate Unbound Layer (E_2 V_2)</td>
<td></td>
<td>(h_2)</td>
</tr>
<tr>
<td>Lower Subgrade Layer (E_3 V_3)</td>
<td></td>
<td>(h_3)</td>
</tr>
</tbody>
</table>

Figure 6.2 The Pavement Structure
Of the parameters controlling the response of each layer, the stiffness, $E$, Poisson's ratio, $\nu$, and thickness, $h$, to the applied load, only the thicknesses of the asphalt surfacing, subbase and the sub-base were known for the structures investigated.

In developing the model, values for the elastic parameters ($E$ and $\nu$) of the individual pavement layers and subgrade were initially deduced from typical published values for each material type. These values were subsequently modified within closely defined limits until the response of the theoretical model was similar to that of the actual pavement structure.

6.2.6.1.1 The Upper Bound Layer

The response of the upper bound layer was controlled by the three parameters, Stiffness $E_1$ (analogous to Young's Modulus at short loading times), Poisson's Strain Ratio $\nu_1$ and the thickness $h_1$.

Previous work has indicated that the stiffness of the asphalt mix can vary from around $1.0 \times 10^7$ N/m$^2$ to about $5.0 \times 10^9$ N/m$^2$. The upper range of stiffness values, $1.0 \times 10^9$ to $5.0 \times 10^9$ N/m$^2$ were determined for a large number of asphalt mixes by means of dynamic or semi-static tests at various temperatures and under different loading conditions.

An initial value of $E_1 = 5.0 \times 10^9$, equivalent to a reasonably stiff layer, was adopted for the first model.

Generally, Poisson's ratio has proved difficult to measure since it requires very accurate measurement and has been shown to vary with stress, temperature etc. (55)

At short loading times Poisson's Ratio was found to be about 0.35 and independent of stress. An initial value of $\nu_1 = 0.35$ was adopted for the first model. It would appear from the literature that values of this parameter have not been measured as often as stiffness.

A value of $h_1 = 100$ mm was fixed by the thickness of the bituminous layer present in the pavement to be modelled.
6.2.6.1.2 Intermediate Granular Layer

The response of the granular layer was controlled by the Resilient Modulus $E_2$ (identical to Young's modulus when determined from repeat load triaxial test), Poisson's Strain Ratio $v_2$ and the layer thickness $h_2$.

The modulus of the unbound layer has been shown to be to a large extent stress dependent.

Field measurements supported by theoretical analysis have shown that the modulus of the unbound base layer $E_2$ depends on its thickness $h_2$ and the modulus of the underlying subgrade $E_3$, according to the relationship:

$$E_2 = K_2 E_3^{0.45}$$

where $K_2 = 0.2 h_2$ with $h_2$ in mm, with the limits $2 < K_2 < 4$.

An initial value of $E_2 = 5.0 \times 10^8$ N/m² was thought realistic for the combination of a wet mix macadam base and type 2 sub-base present in the pavement to be modelled.

An initial value of $v_2 = 0.35$ was adopted for this granular layer.

A value of $h_2 = 450$ mm was fixed by the combined depths of the wet mix macadam base and the type 2 sub-base layer.

6.2.6.1.3 Lower Subgrade Layer

The response of the subgrade was controlled by the Resilient Modulus $E_3$, Poisson's Strain Ratio $v_3$ and a limiting thickness $h_3 = 4$ m.

This depth was selected because it was felt that very little significant deflection would be apparent below this level.

It has been reported that subgrade soils show stress dependent behaviour and relationships have been developed between stress, strain and modulus.
Bleyenberg, et al, have demonstrated from actual measurements on pavements in full scale road experiments, that linear elastic theory can be used to describe pavement response, provided that the moduli of the materials were determined under appropriate loading conditions.

Therefore subgrade modulus should be determined from in situ measurements using equipment such as the surface wave propagation technique.

Where such data was not available, use can be made of the empirical relationship between dynamic subgrade modulus $E_3$ and the CBR value,

$$E_3 = 10^7 \text{CBR N/m}^2$$

Actual data on the subgrade modulus did not exist for the pavement to be modelled and therefore a relatively high value of CBR = 5%, appropriate at formation level, was selected giving a subgrade modulus $E_3 = 5 \times 10^7$ N/m$^2$.

The finite element technique allowed the 4 m subgrade depth to be represented as $8 \times 0.5$ m deep elements, whose modulus varied linearly from $5 \times 10^7$ N/m$^2$ at formation level to $1.0 \times 10^8$ N/m$^2$ at 4 m below formation level.

An initial value of $v_3 = 0.35$ was adopted for the subgrade layer.

6.2.6.1.4 Matching the Response of the Theoretical and Actual Pavement Structures

The elastic constants and thicknesses of the initial model are given in Figure 6.3.

![Figure 6.3 Elastic Constants and Thicknesses of the Original Model](image-url)

Moduli units are N/m$^2$
Analysis of the response of this model to load immediately indicated that an assumed value or values was incorrect because the maximum deflection was almost three times that recorded on the actual pavement's surface and the deflected shape was deeper and shorter.

The obvious conclusion was that the theoretical model was not stiff enough to resist the applied load.

The parameter most influencing the maximum deflection was the stiffness of the subgrade and it was therefore decided to reduce the maximum deflection and depth of the deflection dish by increasing the stiffness of this layer.

This was achieved by the introduction of a layer characterised by a modulus equal to $1 \times 10^{15} \text{Pa}^2$.

The response of the model to the applied load was determined firstly with the stiff layer 1 m below formation level and secondly with the stiff layer 2 m below formation level. The response of each of these models, in terms of the differential deflection, together with that of the actual pavement structure is given in Table 6.2.

The results indicate that the response of the model with the stiff layer 2 m below formation level was the closest to that of the actual pavement.

The differential deflections characterizing the deflected shape of this model were generally lower than those of the actual pavement, indicating that although the maximum deflection values were similar, the theoretical deflection dish was too deep.

Further refinements were therefore necessary to improve the accuracy of match between the theoretical and actual deflected shapes.

6.2.6.1.5 Further refinements to the theoretical model

Two major alterations were made to the finite element model at this stage.
<table>
<thead>
<tr>
<th>DISTANCE FROM MAXIMUM DEFLECTION</th>
<th>DIFFERENTIAL DEFLECTION</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ACTUAL RESPONSE</td>
<td>1 m BELOW FORMATION</td>
<td>2 m BELOW FORMATION</td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>2.0</td>
<td>1.3</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>8.8</td>
<td>4.4</td>
<td>5.8</td>
<td></td>
</tr>
<tr>
<td>600</td>
<td>17.6</td>
<td>8.7</td>
<td>11.6</td>
<td></td>
</tr>
<tr>
<td>900</td>
<td>24.8</td>
<td>13.4</td>
<td>17.6</td>
<td></td>
</tr>
<tr>
<td>1200</td>
<td>27.6</td>
<td>16.7</td>
<td>21.8</td>
<td></td>
</tr>
<tr>
<td>1500</td>
<td>29.2</td>
<td>22.6</td>
<td>26.4</td>
<td></td>
</tr>
<tr>
<td>1875</td>
<td>30.2</td>
<td>28.2</td>
<td>30.8</td>
<td></td>
</tr>
</tbody>
</table>

Table 6.2 Differential deflection at various distances from the point of maximum deflection.
Firstly, the number of elements making up the model was increased to allow the deflected shape of the pavement's surface to be more accurately defined in the vicinity of the maximum deflection. This meant that the deflected shape was now specified at 50, 100, 200, 300, 400 and 500 mm from the maximum deflection.

Secondly, the intermediate granular layer was divided into two layers; one representing the granular roadbase and the other, the granular sub-base. This alteration was necessary because future work was to include an assessment of the effect of change in either the granular roadbase or the granular sub-base on the estimate of the bituminous layer thickness.

Figure 6.4 shows the amended model used for subsequent analyses.

Having made these changes, the model was then used to improve the accuracy of match between the theoretical response and the actual response of the pavement structure. This proved to be very much a trial and error exercise of altering the moduli of the pavement layers and subgrade within closely defined limits.

The most accurate theoretical representation of the actual pavement response, Figure 6.5, was produced by loading the pavement structure shown in Figure 6.6.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Modulus $E$ ($\times 10^9$)</th>
<th>Poisson's Ratio $\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hot Rolled Asphalt</td>
<td>$1.5 \times 10^{10}$</td>
<td>0.35</td>
</tr>
<tr>
<td>Wet Mix Macadam</td>
<td>$6 \times 10^{8}$</td>
<td>0.35</td>
</tr>
<tr>
<td>Type 2 Sub-base</td>
<td>$6 \times 10^{8}$</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>$1.5 \times 10^{8}$</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>$5 \times 10^{8}$</td>
<td>0.35</td>
</tr>
<tr>
<td>SUBGRADE</td>
<td>$8 \times 10^{9}$</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>$8 \times 10^{10}$</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Figure 6.6 Details of the pavement structure with a granular roadbase.
Figure 6.4 The amended pavement model
Figure 6.5 Comparison of the actual and theoretical response of a pavement with a granular roadbase.

- Hot Rolled Asphalt
- Wet-mix Macadam
- Type 2 Sub-base

PAVEMENT CONSTRUCTION

Distance from the maximum deflection (mm)

- Actual response
- Theoretical response

Distance from the maximum deflection (mm)
A theoretical model was developed that accurately predicts the response under load for a pavement containing a granular roadbase.

6.2.6.1.6 Comparison of the stress levels in the bottom of the bituminous layer

As a further check on the accuracy of the response of the theoretical model with a granular roadbase, the computed stress levels were compared with the average stress levels suggested by the Shell Method\(^{(32)}\) for a structure with similar elastic properties and thicknesses.

The Shell Method\(^{(32)}\) presents a procedure for calculating the average stress levels in the bituminous layer.

To use this procedure it is necessary to divide the bituminous layer into three sub-layers; the first two layers should be 40 mm thick and the third layer equal to the total bituminous layer thickness less 80 mm.

The reason given for this method of subdividing the thickness of the bituminous layer is that the uppermost layers are subject to the greatest temperature changes and are usually made of a different type of mix from that usual in the lower layers.

The average stress in each sub-layer is dependent upon six variables \(h_{1-3}, E_3, h_2, E_{1-3}, E_{1-2}\) and \(E_{1-1}\), where \(E_3\) = subgrade modulus, \(h_2\) = thickness of the unbound layer, \(E_{1-3}\) = modulus of the bottom asphalt layer, \(E_{1-2}\) = modulus of the intermediate asphalt layer, \(E_{1-1}\) = modulus of the top asphalt layer and \(h_{1-3}\) = the thickness of the lower bituminous sub-layer.

The procedure calculates the average stress in each sub-layer as the product of the contact stress of the standard design wheel, \(6 \times 10^5\) N/m\(^2\) and the proportionality factor \(z\).

The appropriate proportionality factor \(z\) is determined using the six previously specified variables as input into a number of data tables.
For the pavement structure with a granular roadbase,

\[ E_3 = 1.5 \times 10^8 \text{ N/m}^2 \]

\[ h_2 = 450 \text{ mm} \]

\[ h_{1-3} = 20 \text{ mm} \]

\[ E_{1-3} = 1.5 \times 10^{10} \text{ N/m}^2 \]

\[ E_{1-2} = 1.5 \times 10^{10} \text{ N/m}^2 \]

\[ E_{1-1} = 1.5 \times 10^{10} \text{ N/m}^2 \]

Proportionality factors \((z)\) for this non typical structure of 100 mm of bituminous material on top of 450 of granular material are not contained in the data tables presented in the Shell Method.\(^{(32)}\) Use was therefore made of the tables corresponding to the structures closest to that investigated. Details of these structures and the corresponding \(z\) factors derived from the tables in the Shell Method\(^{(32)}\) are given in Table 6.3.

<table>
<thead>
<tr>
<th>Subgrade Modulus N/m²</th>
<th>Thickness of unbound layer (mm)</th>
<th>Thickness of the mid-asphalt sub-layer (mm)</th>
<th>(z_1)</th>
<th>(z_2)</th>
<th>(z_3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 (\times) (10^8)</td>
<td>300</td>
<td>0</td>
<td>-0.3</td>
<td>1.1</td>
<td>-</td>
</tr>
<tr>
<td>2 (\times) (10^8)</td>
<td>300</td>
<td>50</td>
<td>0</td>
<td>0.5</td>
<td>0.8</td>
</tr>
<tr>
<td>2 (\times) (10^8)</td>
<td>600</td>
<td>0</td>
<td>-0.1</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>2 (\times) (10^8)</td>
<td>600</td>
<td>50</td>
<td>0.1</td>
<td>0.5</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Table 6.3

In all cases \(E_{1-1}, E_{1-2},\) and \(E_{1-3}\) were equal \(1 \times 10^{10} \text{ N/m}^2\).

The proportionality factors \((z)\) of interest were those corresponding to the lower bituminous sub-layer and reference to Table 6.3 will show that the range lies between 0.8 and 1.1. This would suggest an average level of between \(4.8 \times 10^5 \text{ N/m}^2\) and \(6.6 \times 10^5 \text{ N/m}^2\) in the lower bituminous layer.
The stress level at the bottom of the bituminous layer of the structure with a granular roadbase was calculated, using the finite element approach to be equal to $1.5 \times 10^6$ N/m$^2$.

This value is greater than that suggested by the charts in the Shell Method due probably to the differences in the magnitude of the wheel loads, layer moduli and layer thicknesses.

This exercise has shown that the stress level in the bituminous layer of the pavement model is not unrealistic.

This, together with the fact that the deflected shape of the theoretical model was very similar to that of the actual pavement structure, suggested that the response was sufficiently accurate to be used as the basis of an investigation into a relationship between curvature and bituminous layer thickness.

6.2.6.2 Pavement with a bituminous roadbase

The absolute response of a pavement with a bituminous roadbase has also been reproduced theoretically by defining the structure as the layered system, shown in Figure 6.7.

<table>
<thead>
<tr>
<th>Hot Rolled Asphalt</th>
<th>$E = 1.5 \times 10^{10}$</th>
<th>$v = 0.35$</th>
<th>100 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bituminous Roadbase</td>
<td>$E = 1.5 \times 10^{10}$</td>
<td>$v = 0.35$</td>
<td>75 mm</td>
</tr>
<tr>
<td>Type 2 Sub-base</td>
<td>$E = 6 \times 10^8$</td>
<td>$v = 0.35$</td>
<td>150 mm</td>
</tr>
<tr>
<td>SUBGRADE</td>
<td>$E = 1.5 \times 10^8$</td>
<td>$v = 0.35$</td>
<td>500 mm</td>
</tr>
<tr>
<td></td>
<td>$E = 5 \times 10^8$</td>
<td>$v = 0.35$</td>
<td>500 mm</td>
</tr>
<tr>
<td></td>
<td>$E = 8 \times 10^9$</td>
<td>$v = 0.35$</td>
<td>1000 mm</td>
</tr>
<tr>
<td></td>
<td>$E = 8 \times 10^{10}$</td>
<td>$v = 0.35$</td>
<td>2000 mm</td>
</tr>
</tbody>
</table>

Figure 6.7 Details of the pavement structure with a bituminous roadbase.
Again, only the thicknesses of the pavement layers were known for the actual structure. However, in this case it could be assumed that the moduli and Poisson's ratio values for the hot rolled asphalt, the type 2 sub-base and the subgrade, were the same as those for the structure with a granular roadbase, because both structures were adjacent sections in the small road system at the Transport and Road Research Laboratory. This meant that the only unknown was the modulus of the bituminous roadbase. Initial values were taken from typical published data and were subsequently altered within clearly defined limits to improve the accuracy of match between the response of the theoretical model and that of the actual pavement structure. The most accurate representation of the actual response of a pavement with a bituminous roadbase is given in Figure 6.8.
Figure 6.8 Comparison of the actual and theoretical response of a pavement with a bituminous roadbase.
6.3 IDENTIFYING THE SECTION OF THE DEFLECTED SHAPE MOST INFLUENCED BY THE PROPERTIES OF THE BITUMINOUS MATERIAL

A prerequisite to developing a relationship between curvature and thickness of the bituminous layer was the need to identify the section of the deflected shape most sensitive to changes in the thickness of the bituminous layer and least sensitive to changes in the strength of the subgrade.

This was determined by independently varying the thickness of the bituminous layer and strength of the subgrade and expressing the effects, on the deflected shape, as percentage changes in the differential deflection at various distances from the point of maximum deflection, Figure 6.9.

Reference to Figure 6.9 will show that the thickness of the bituminous layer has the greatest influence on the deflected shape in the vicinity of the maximum deflection and that subgrade strength has the greatest influence on the deflected shape at distance from the point of maximum deflection.

6.3.1 Selecting the position of a suitable ordinate deflection

Having determined that the shape in the vicinity of the maximum deflection was of most interest, the next step was to select the position of the ordinate deflection within this area whose changes in magnitude, as a result of changes in the thickness of the bituminous layer, were large enough to be recorded by a Deflectograph, i.e. there would be no point in selecting the ordinate deflection 50 mm from the point of maximum deflection if the corresponding differential deflection value at this point changed by less than \( \frac{1}{100} \) mm when the thickness was reduced from 300 mm to 50 mm.

The most suitable position of the ordinate deflection was determined from Figure 6.10, which shows the differential deflection at various distances from the point of maximum deflection for pavements, with 50, 100, 150 and 300 mm of bituminous material.
Figure 6.9 The effect of an increase in bituminous layer thickness and a decrease in subgrade CBR on the differential deflection at various distances from the point of maximum deflection.
Figure 6.10 Chart showing the differential deflection at various distances from the point of maximum deflection for an asphalt mixture with 50, 100, 150 and 300 mm of bituminous material.
Figure 6.10 indicates that the magnitude of the differential deflection increases with decrease in the thickness of the bituminous material and that the range of differential deflection, for a given range of layer thickness, increases with increase in distance from the point of maximum deflection.

The differential deflection 200 mm from the point of maximum deflection was selected as the measure of curvature, because the ever decreasing range of differential deflections nearer to the maximum deflection would have made it extremely difficult to detect changes in bituminous layer thickness.
6.4 INVESTIGATING THE RELATIONSHIP BETWEEN DEFLECTED SHAPE AND THICKNESS OF THE BITUMINOUS LAYER

Analysis of the effect of different thicknesses of bituminous material on the response of the theoretical model has resulted in the derivation of a relationship between curvature and thickness of the bituminous material, Figure 6.11. This relatively simple relationship could have formed the basis of a bituminous layer estimation procedure if it could have been assumed that the strengths and thicknesses of the other pavement layers and subgrade remained constant along a length of road.

However, variations in these values do occur in practice and it was therefore necessary to investigate the possible effect of each likely change on the deflected shape and hence estimate of thickness.

A range of thicknesses and moduli, Table 6.4, thought likely to occur for the materials present in the pavement layers and subgrade, were incorporated into the model. In each case the magnitude of all other parameters remained unaltered when the magnitude of either the modulus or thickness of one of the layers was changed.

The effect of these changes on the deflected shape was determined for pavements with 50, 100, 150 and 300 mm of bituminous material.

(The values underlined correspond to the initial pavement structure modelled).

<table>
<thead>
<tr>
<th>Layer</th>
<th>Modulus (N/m²)</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bituminous Surfacing</td>
<td>$4.2 \times 10^8, 1.9 \times 10^9$</td>
<td>$50,100,150,300$</td>
</tr>
<tr>
<td></td>
<td>$6.2 \times 10^9, 1.5 \times 10^{10}$</td>
<td></td>
</tr>
<tr>
<td>Granular Roadbase</td>
<td>$2 \times 10^8, 6 \times 10^8, 9 \times 10^8$</td>
<td>$200,300,400$</td>
</tr>
<tr>
<td>Granular Sub-base</td>
<td>$2 \times 10^8, 6 \times 10^8, 9 \times 10^8$</td>
<td>$100,150,200$</td>
</tr>
<tr>
<td>Subgrade</td>
<td>$2.5 \times 10^7, 5.0 \times 10^7, 1.0 \times 10^8, 1.5 \times 10^8, 2.5 \times 10^8$</td>
<td>Constant at $4000$</td>
</tr>
</tbody>
</table>

Table 6.4 Typical Moduli and Thickness Values
Figure 6.11 Chart showing the relationship between curvature, defined as a differential deflection 200 mm from the point of maximum deflection, and thickness of the bituminous layer.
Analysis of the deflected shape measurements produced by the model for each parameter change has resulted in the construction of a chart, Figure 6.12, showing the relationship between curvature, defined as a differential deflection 200 mm from the point of maximum deflection, and maximum deflection for pavements with various thicknesses of bituminous material.

The slopes of the lines on the chart suggest that a measure of the deflected shape in the vicinity of the maximum deflection is more sensitive to changes in the properties of the bituminous material (greater slope) than the properties of the other pavement materials, and that the maximum deflection is greatly influenced by the properties of the subgrade (one of the main problems associated with its use as a measure of pavement condition).

This trend becomes less pronounced as the thickness of the bituminous material increases.

Figure 6.12 demonstrates that variations in the properties of the pavement layers and subgrade will effect the deflected shape, especially for pavements with thin layers of bituminous material, and that these changes in shape will also influence the estimate of the thickness of bituminous material. Quantification of the errors that may be introduced in the estimates of thickness as a result of these variations in material properties are considered later in this chapter.

This chart, Figure 6.12, could be used in conjunction with measurements of the absolute deflected shape of the pavement's surface, to estimate the thickness of bituminous material present in a flexible pavement structure.

However, to allow estimates of layer thickness to be made from curvature measurements recorded by a Deflectograph, it is necessary to express this relationship in terms of equivalent Deflectograph deflected shape measurements.
Figure 6.12 The influence of change in the stiffness of the bituminous layer and resilient modulus of the roadbase, sub-base and subgrade on the predicted absolute deflected shape measurements for pavements with 50, 100, 150 and 300 mm of bituminous material.
6.5 CONVERTING ABSOLUTE DEFLECTION MEASUREMENTS INTO EQUIVALENT DEFLECTOGRAPH DEFLECTION MEASUREMENTS

The Deflectograph, like the Deflection Beam, does not measure the absolute deflection produced by its rear wheels; only a proportion of the deflection is measured.

The proportion of the deflection not seen by the measuring system is related to the fact that the vehical loading influences the pavement over a large area, Figure 6.13, and the fact that the datum frame rotates during the measurement cycle as the position of the wheels change relative to the datum points.

![Diagram of Deflectograph measurement cycle](image)

**CONDITIONS AT THE START OF THE MEASUREMENT CYCLE**

**UNDISTURBED ROAD SURFACE**

deflection midway between wheelpaths

dataim frame

measuring arm

Deflection in nearside wheelpath

**CONDITIONS AT THE POINT OF MAXIMUM RECORDED DEFLECTION**

**UNDISTURBED ROAD SURFACE**

dataim frame

measuring arm

Figure 6.13
These effects make the choice of datum frame and measurement arm critical and also dictate that the location of the measurement cycle relative to the loaded wheels should be accurately defined and constant for each measurement.

6.5.1 Estimating the deflection measured by a Deflectograph

An equation has been derived that can be used to estimate the accuracy of the deflection as measured by a Deflectograph:

\[ \text{Recorded deflection} = (Y_{BT} - Y_{BO}) - \frac{L_2}{L_1}(Y_{CT} - Y_{CO}) - (1 - \frac{L_2}{L_1})(Y_{AT} - Y_{AO}) \]

where \( Y_A \) = movement of datum point A
\( Y_B \) = movement of datum point B
\( Y_C \) = movement of datum point C

The suffixes O & T refer to the initial and maximum position of the datum points relative to the undisturbed road surface.

\[ \text{Diagram showing reference frame and measuring arms.} \]

It was not possible to use Equation 6.1 to determine directly the equivalent Deflectograph response from a knowledge of the absolute deflection, because it represented the response of a two-dimensional model of a flexible pavement structure and, as such, gave no indication of the movement of the datum point B at the end of the reference frame.

An alternative method of transforming absolute deflections into equivalent Deflectograph deflections was therefore required.

The method adopted was to initially convert the absolute deflection measurements into equivalent Deflection Beam deflections and then use
correlation factors to convert these deflections into equivalent Deflectograph deflections.

N.B. The equation for converting absolute deflections into equivalent Deflection Beam deflections is given later and reference to it will show that information is not required on the movement mid-way between the wheelpaths to convert absolute deflections into Deflection Beam deflections.

6.5.2 Derivation of the correlation factors between the Deflection Beam and Deflectograph deflections.

The deflected shape information, Figure 6.14, used initially to ensure that the response of the theoretical model was similar to that of an actual pavement structure, also included a measure of the deflection mid-way between the wheelpaths (movement of datum point B). This meant that this information could be used to determine,

(i) the equivalent Deflectograph deflections using Equation 6.1;

and

(ii) the equivalent Deflection Beam deflections using the following general equation:

\[
\text{Deflection} = \frac{1}{4} \times \text{movement of the measuring toe} - \frac{7}{5} \times \text{movement of the front feet} - \frac{9}{10} \times \text{movement of the rear feet}.
\]

The overall deflection measurement is composed of the initial, maximum and final recorded deflections, i.e.:

\[
\text{Overall deflection} = 2 \times \text{recorded maximum reading} - \text{recorded final reading}.
\]

---Equation 6.2

As is the case with the Deflectograph, the deflection measurement recorded by the Deflection Beam is related to the movement of the reference frame during the measuring cycle. The reference frame
Figure 6.14 Deflection in nearside wheelpath and midway between the wheelpaths on a pavement with a granular roadbase.
of the Deflection Beam is mounted on two sets of feet and it is the movement of these front and rear feet which influences the recorded deflection.

Figure 6.15 shows the equivalent Deflectograph and Deflection Beam deflections calculated using the absolute deflection measurements (Figure 6.14) and Equations 6.1 and 6.2 respectively.

The relationship is very much as expected; the response measured by the Deflectograph is less than that measured by the Deflection Beam and that both are less than the absolute deflected shape.

Analysis of the deflected shapes shown in Figure 6.15 has allowed correlation factors to be derived for the relationship between Deflection Beam deflections and Deflectograph deflections at 100 mm intervals from the point of maximum deflection, Table 6.5.

<table>
<thead>
<tr>
<th>Distance from the maximum deflection (mm)</th>
<th>Deflection Beam deflection ($\frac{1}{100}$ mm)</th>
<th>Deflectograph deflection ($\frac{1}{100}$ mm)</th>
<th>Correlation Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>31.8</td>
<td>30.5</td>
<td>0.96</td>
</tr>
<tr>
<td>100</td>
<td>30.14</td>
<td>29.3</td>
<td>0.97</td>
</tr>
<tr>
<td>200</td>
<td>26.42</td>
<td>23.4</td>
<td>0.89</td>
</tr>
<tr>
<td>300</td>
<td>21.78</td>
<td>17.5</td>
<td>0.81</td>
</tr>
<tr>
<td>400</td>
<td>17.67</td>
<td>12.0</td>
<td>0.68</td>
</tr>
<tr>
<td>500</td>
<td>13.00</td>
<td>7.5</td>
<td>0.58</td>
</tr>
<tr>
<td>600</td>
<td>9.06</td>
<td>3.7</td>
<td>0.40</td>
</tr>
<tr>
<td>700</td>
<td>4.76</td>
<td>1.4</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Table 6.5 Correlation coefficients for deflection measurements on a structure with a granular roadbase.

The correlation factors, based upon the response of a structure with a substantial granular roadbase on top of a granular sub-base, suggests that the Deflectograph deflections are at their closest to the Deflection Beam deflections in the vicinity of the maximum deflection.
Figure 6.15 Equivalent Deflection Beam and Deflectograph Deflections based upon the response of the actual pavement.
and that the difference between the two increases with increase in distance away from the point of maximum deflection.

The correlation factors also appear to be dependent upon the response of the structure being surveyed.

Presented in Table 6.6 are the correlation factors for a structure with a bituminous roadbase and comparison of these values with those in Table 6.5 will indicate smaller correlation factors for measurements in the vicinity of the maximum deflection on the stiffer structures (bituminous roadbase). This would suggest a greater difference between the deflected shapes recorded by a Deflection Beam and a Deflectograph on stiffer structures in the vicinity of the maximum deflection, but closer agreement at distance from the maximum deflection.

<table>
<thead>
<tr>
<th>Distance from the maximum deflection (mm)</th>
<th>Deflection Beam deflection ((\frac{1}{100}) mm)</th>
<th>Deflectograph deflection ((\frac{1}{100}) mm)</th>
<th>Correlation Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>14.4</td>
<td>11.9</td>
<td>0.83</td>
</tr>
<tr>
<td>100</td>
<td>13.6</td>
<td>11.33</td>
<td>0.83</td>
</tr>
<tr>
<td>200</td>
<td>12.3</td>
<td>10.05</td>
<td>0.82</td>
</tr>
<tr>
<td>300</td>
<td>11.2</td>
<td>8.5</td>
<td>0.76</td>
</tr>
<tr>
<td>400</td>
<td>9.6</td>
<td>6.8</td>
<td>0.71</td>
</tr>
<tr>
<td>500</td>
<td>7.9</td>
<td>4.9</td>
<td>0.62</td>
</tr>
<tr>
<td>600</td>
<td>6.0</td>
<td>3.0</td>
<td>0.50</td>
</tr>
<tr>
<td>700</td>
<td>4.0</td>
<td>1.6</td>
<td>0.40</td>
</tr>
</tbody>
</table>

Table 6.6 Correlation coefficients for deflection measurements on a structure with a bituminous roadbase.
6.6 THE DEVELOPMENT OF A BITUMINOUS LAYER THICKNESS ESTIMATION CHART FROM PREDICTED DEFLECTOGRAPH CURVATURE AND DEFLECTION MEASUREMENTS.

The equivalent Deflectograph curvature (defined in terms of the differential deflection 200 mm from the point of maximum deflection) and maximum deflection readings were used to construct a bituminous layer thickness estimation chart, Figure 6.16, of the type shown in Figure 6.12.

The relationship between curvature and maximum deflection is shown for pavements with 50, 100, 150 and 300 mm of bituminous material. The influence that layer stiffness and thickness can have on the relationship is also presented.

The fact that the deflected shapes recorded by the Deflectograph are steeper than the absolute, Figure 6.15, has resulted in a difference in the magnitude of the relationships in Figure 6.16, when compared to those shown in Figure 6.12; for a given maximum deflection, the corresponding differential deflection calculated from the Deflectograph deflections is greater than the absolute differential deflection.

These differences between the absolute deflected shape and those recorded by the Deflectograph are entirely due to the influence that the movement of the deflection bowl has on the measuring arm of the Deflectograph during the measuring cycle.

The differences in magnitude are not important because this chart, Figure 6.16, could be used in conjunction with the deflected shape measurements recorded by a Deflectograph to estimate the thickness of bituminous material present in a pavement structure.

148
Figure 6.16 The influence of change in the stiffness of the bituminous layer and resilient moduli of the roadbase, sub-base and subgrade on the equivalent deflected shape, as recorded by a Deflectograph on pavement structure with 50, 100, 150 and 300 mm bituminous material.
The accuracy of the proposed method was assessed by quantifying the effect of changes in either the modulus or the thickness of each of the pavement layers and subgrade on the estimate of bituminous layer thickness.

In each case, values considered to represent the extremes of the range for the modulus and the thickness, Table 6.4 were incorporated into the model.

For each parameter change, the effect on the deflected shape and estimate of bituminous layer thickness is expressed as an accuracy chart, which can be used to determine the consequences of a particular change on the estimate of a range of bituminous thicknesses.

Also considered is the effect that soft spots in the granular sub-base and cracks in the bituminous layer can have on the estimate of bituminous layer thickness.

### 6.7.1 Stiffness of the bituminous material

Temperature and traffic loading are the two parameters most likely to affect the stiffness of the bituminous material.

#### 6.7.1.1 Changes in stiffness due to temperature

Temperature can have a great influence on the stiffness of the bituminous material and hence the measurement of curvature. It was therefore decided to investigate the possible effect that an increase in temperature from $10^\circ C$ to $20^\circ C$ can have on the estimate of bituminous layer thickness.

Use was made of Figure 6.17 to determine the temperatures within 50, 100, 200 and 400 mm of bituminous material for Mean Monthly Air Temperatures of $10^\circ C$ and $20^\circ C$, Table 6.7.
Figure 6.17 Relationship between Effective Asphalt Temperature and MMAT or w-MAAT (after Claessen, et al) [32]
Table 6.7 Pavement temperatures corresponding to air temperatures of 10 and 20°C.

Reference to Table 6.7 will show that for a given air temperature a higher temperature exists within the thinner bituminous layers.

These pavement temperatures were then used in conjunction with curve S1 in Figure 6.18 to estimate the corresponding stiffness of the mix, Table 6.8.

Table 6.8 Mix stiffnesses corresponding to air temperatures of 10 and 20°C.

An assumption was made at this stage as to the applicability of the curves in Figure 6.18.

The lack of more detailed information resulted in curve S1 being selected as the most appropriate for the bituminous material present in the pavement, because it represented the average stiffness.
Figure 6.18 Characteristic Relationship between Mix Stiffness and Mix Temperature (after Claessen et al.) (32)
characteristics of most common dense base course mixes.

The thicknesses and stiffnesses shown in Table 6.8 were incorporated into the theoretical model and the resulting effect on the deflected shape and estimate of bituminous layer thickness is expressed in Figure 6.19.

The results were as to be expected; the lower stiffnesses brought about by the higher temperatures greatly reduced the overall strength of the road as indicated by the increased curvature values.

Reference to Figure 6.19 will show that an increase in air temperature from 10°C to 20°C on a pavement with 100 mm of bituminous material could result in about a 45% underestimate of the thickness present. A similar increase in temperature on a pavement with 400 mm of bituminous material could result in about a 40% underestimate of the thickness present.

The strong influence that the effect of temperature on the stiffness of the bituminous material can have on curvature is clearly demonstrated, and must be taken into consideration in any procedure for estimating bituminous layer thickness from curvature measurements.

Details are given later in this chapter of a procedure for temperature correcting curvature measurements, recorded with a Deflectograph, to equivalent values at 20°C.

6.7.1.2 Changes in stiffness due to traffic loading

Even in a well designed pavement the stiffness of the bituminous material will very gradually decrease as the number of standard axles carried by the pavement increases. Changes in stiffness due to traffic would not occur during the time taken to survey a given length of road with a Deflectograph.
Figure 6.19 The effect of the stiffness of the bituminous material on the estimate of the thickness of the bituminous material.
6.7.2 The modulus and thickness of the granular layers

6.7.2.1 Changes in modulus

Changes in the resilient modulus of the granular layers can occur as a result of either the presence of water or changes in the material type, density, etc. The problem of water entering a granular layer is dealt with more fully later in this section.

Two moduli values representing the typical upper and lower limits for the materials present in the roadbase and sub-base were incorporated into the theoretical model.

The effect of these roadbase and sub-base moduli on the deflected shape and estimate of thickness of the bituminous layer is shown in Figures 6.20 and 6.21 respectively.

Reference to Figure 6.20 will show that a reduction in roadbase modulus of about 66 per cent can result in a 40 per cent underestimate in the thickness of a 100 mm layer of bituminous material, or a 10 per cent underestimate in the thickness of a 250 mm bituminous layer.

An increase in roadbase modulus of about 33 per cent can result in a 15 per cent overestimate in the thickness of a 100 mm bituminous layer, or an 8 per cent overestimate in the thickness of a 250 mm bituminous layer.

The effect of similar increases and decreases in the sub-base modulus, Figure 6.21, was less than that associated with corresponding changes in the roadbase modulus. This was probably due to the fact that the thickness of the sub-base was half that of the roadbase.

6.7.2.2 Changes in thickness

Changes in the thickness of the roadbase and the sub-base can occur either during the initial construction period or as a result of subsequent strengthening work.
Figure 6.20 The effect of roadbase modulus on the estimate of bituminous layer thickness.
Figure 6.21 The effect of sub-base modulus on the estimate of bituminous layer thickness.
The effect of an increase or decrease in the thickness of the roadbase and sub-base on the deflected shape and estimate of bituminous layer thickness have been studied and the results presented in Figures 6.22 and 6.23 respectively.

Reference to Figure 6.22 will show that an increase in the roadbase thickness of about 33 per cent can result in an 8 per cent overestimate in the thickness of a 100 mm bituminous layer, or a 2 per cent overestimate in the thickness of a 250 mm bituminous layer.

A decrease in the roadbase thickness of about 33 per cent can result in a 4 per cent underestimate in the thickness of a 100 mm bituminous layer, or a 2 per cent underestimate of a 250 mm bituminous layer.

Increasing or decreasing the thickness of the sub-base by about 33 per cent has little effect (less than 5 per cent) on the estimate of bituminous layer thickness, Figure 6.23.

It would appear that for both the roadbase and sub-base the limits of thickness used have less effect on the estimate of bituminous layer thickness than the limits of modulus. It would therefore follow that more attention should be paid to the likelihood of changes in the modulus of these materials than to changes in their thickness.

6.7.3 The modulus of the subgrade

Changes in subgrade modulus are common and can occur for a variety of reasons; change in material type, compaction, density, moisture content, etc.

Subgrade strength is normally specified in terms of the California Bearing Ratio (CBR); strong subgrades have a high CBR value 15+ and weak subgrades have a low CBR value, less than 5.

The theoretical model was used to show the effect that a range of subgrade CBR values (2.5, 5, 10, 15 and 25) would have on the deflected shape and estimate of the thickness of the bituminous material. The results of this work are presented in Figure 6.24.
Figure 6.22 The effect of roadbase thickness on the estimate of bituminous layer thickness.
Figure 6.23 The effect of sub-base thickness on the estimate of bituminous layer thickness.
Figure 6.24  The effect of subgrade modulus on the estimate of bituminous layer thickness.
Reference to Figure 6.24 will show that a reduction in subgrade modulus of about 66 per cent (equivalent to a CBR change from 15 to 5) can result in a 50 per cent underestimate in the thickness of a 100 mm bituminous layer, or in a 40 per cent underestimate in the thickness of a 250 mm bituminous layer.

An increase in subgrade modulus of about 66 per cent (equivalent to a CBR change from 15 to 25) can result in a 15 per cent overestimate in the thickness of a 100 mm bituminous layer or a 26 per cent overestimate in the thickness of a 250 mm bituminous layer.
6.7.4 The effect of soft spots in the granular sub-base on the estimate of bituminous layer thickness

The term, 'soft spot', is often used to describe a section of granular material significantly weakened by the presence of water.

The existence of soft spots will result in an increase in both the maximum deflection and the curvature, defined as a differential deflection 200 mm from the point of maximum deflection.

Any increase in the magnitude of the differential deflection will be interpreted as a decrease in the thickness of bituminous material present in the pavement, i.e. it will lead to an underestimate of the thickness.

The object of this investigation was to quantify the extent of the underestimate caused by the existence of soft spots of various sizes, introduced at different positions in the sub-base relative to the point of application of the load.

To achieve this objective it was not necessary to make any major changes to the structure of the theoretical model, Figure 6.4.

Figure 6.25 represents a 1.5 m section of the model showing the elements making up the bituminous surfacing layer, the granular roadbase layer and the granular sub-base layer.

The elements making up the subgrade are not detailed.

The sub-base elements have been numbered 1 to 9 to allow easy identification of the position of the weaker areas, characterized by a modulus equal to \( 1 \times 10^5 \) N/m\(^2\).

Table 6.9 gives details of the size and position of the soft spots and expresses their relative effect in terms of a percentage underestimate of the thickness of the bituminous layer. It can be concluded from Table 6.9 that the magnitude of the underestimate of a 100 mm thick bituminous layer, by the introduction of weak
Figure 6.25 A 1.5 m section of the pavement model showing the bituminous layer and granular roadbase and sub-base layers.
Table 6.9 The effect of a soft spot on the estimate of thickness of bituminous material.
areas in the sub-base, is dependent upon their size and position relative to the differential deflection used in the estimate of layer thickness.

The vast decrease in strength associated with the soft spots was selected on purpose to demonstrate the effect that these extreme conditions can have on the estimate of bituminous layer thickness.

It must be emphasised that these underestimates only apply to the particular structure investigated and that they cannot be assumed to be representative of the possible influence of soft spots on the estimate of bituminous layers of different thicknesses.
6.7.5 The effect of single vertical cracks on the estimate of bituminous layer thickness

Work was undertaken to quantify the effect that fatigue cracking in the bottom of the bituminous layer can have on the estimate of bituminous layer thickness. Fatigue cracks arise as a result of excessive tensile strains being developed at the bottom of the bituminous layer; a phenomenon associated with a reduction in the stiffness of the bituminous material.

Assumptions were made as to the size of the vertical cracks and the thickness of the bituminous layer. No provision was made for investigating the rate of propagation of the crack through the bituminous material.

Any estimate of the effect of cracking in the lower half of a bituminous layer on the estimate of thickness was therefore only applicable to one particular instance in time relative to the propagation of the crack through the material and to one thickness of bituminous material.

It was not thought that this approach was unduly restrictive, because it was felt that the results would at least give a first indication of the possible effects of cracking on the estimate of bituminous layer thickness.

A number of changes were made to the finite element model to allow the individual cracks to be modelled. Firstly, the number of rows of elements representing the bituminous layer was increased from 1 to 3, to allow the length of the crack to be specified. Secondly, to make use of the assumptions in the CRACK TIP routine of PAFEC 75, it was necessary to rearrange the nodes of the elements in the region of the crack tip.

The elements of this particular model had one mid-side node and these were moved to the $\frac{1}{4}$ position at each crack tip, Figure 6.26.
Figure 6.26 Distortion of the Elements in the Region of the Crack Tip

Note that the crack itself was identified by specifying two nodes at the same position, Figure 6.26, i.e. node 1 would be included in the list of nodes for element A and node 2 (although at the same position as node 1) would be included in the list of nodes defining element B.

Vertical cracks were introduced into the lower half of a 175 mm thick bituminous layer at distances of 250, 500, 750, 1000, 1500 and 2000 mm from the point of application of the load. Specifying the cracks at various locations relative to the point of application of the load, allowed an assessment to be made of the effect that the position within the layer can have on the estimate of thickness.

The effect of vertical cracks at these positions on the estimate of bituminous layer thickness is given in Table 6.10.

<table>
<thead>
<tr>
<th>Position of the crack relative to the point of application of the load (mm)</th>
<th>250</th>
<th>500</th>
<th>750</th>
<th>1000</th>
<th>1500</th>
<th>2000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percentage underestimate in the thickness of the bituminous layer.</td>
<td>6</td>
<td>4</td>
<td>2</td>
<td>2</td>
<td>&lt;1</td>
<td>&lt;1</td>
</tr>
</tbody>
</table>

Table 6.10 The Effect of a Single Crack on the Estimate of Bituminous Layer Thickness.
A single vertical crack resulted in an underestimate of bituminous layer thickness; the magnitude of the underestimate was generally less than 6% and was largely dependent upon the position of the crack relative to the differential deflection used to define deflected shape.
6.8 MEASURING CURVATURE WITH THE DEFLECTOGRAPH

A Deflectograph was used to measure the deflected shape of the road's surface on the lightly trafficked test sites in Devon and Somerset. Digital equipment has been added to the basic Deflectograph in order that the deflected shape or curvature of the road's surface, as seen by the transducer in the measuring head, is recorded as a series of ordinate deflection readings, Figure 6.27.

The deflected shape or curvature of the pavement's surface is recorded on a time basis and the total number or ordinate deflection readings making up each curvature measurement is therefore dependent upon the speed of the Deflectograph. This can best be explained by considering the cycle of operation as illustrated in Figure 6.28.

At this start of the measuring cycle the dual rear wheels are in position A and the measuring beam assembly is stationary. The tips of the measuring arms are then at a position approximately 1100 mm ahead of the centre-line of the rear axle. To connect the beam arm extension to the transducer core, a solenoid is energised; this causes two anvils to grip the vertical spring of the transducer. The tips of the measurement arms are by then approximately 990 mm in front of the centre-line of the rear axle, and it is from this point B that the measurement of deflection begins. As the rear wheels continue to move towards the measuring arms, the road surface at point C senses the bowl of deflection moving forward with the wheel. This downward movement is detected by the measuring arm and is transferred to the measurement transducer via the extension arm and the clamping...
Figure 6.28 Cycle of Operation (After Kennedy, et al) (25)
solenoid in the recording track. When the centre-line of the rear wheels has reached a point D, 230 mm in front of the tip of the measurement arms, the clamping solenoids are de-energised; this allows the transducer armature and vertical spring to fall back to their rest position.

The distance over which the wheels move through (B to D), in relation to the stationary beam assembly, is relatively constant and it is the speed of the Deflectograph which governs the time for which the transducer is operational.

The changes in speed referred to were relatively small and occurred when the Deflectograph was either going up or down hills and it follows that the severity of the steepness of the hill will control the rate of change of the total number of ordinate deflection readings making up each curvature measurement.

Use of the Deflectograph on lightly trafficked roads has indicated that the total number of ordinate deflection readings generally lies between 550 and 650.

A decrease in speed results in more ordinate deflection readings per curvature measurement and this will result in them being more closely spaced in terms of horizontal distance. Conversely, an increase in speed means less ordinate deflection readings per curvature measurements resulting in them being more widely spaced in terms of horizontal distance.

As an example, for the ordinate deflection reading range given previously, the lower limit gives a horizontal spacing of $\frac{1220}{550} = 2.2 \text{ mm}$, and the upper limit gives a spacing of $\frac{1220}{650} = 1.9 \text{ mm}$.

6.8.1 Selecting the position of the ordinate deflections to define the deflected shape

The digital equipment has been so arranged that it is possible to pre-select ten of the ordinate deflection readings to define the shape of the deflection dish.
A maximum of four different spacings between the groups of readings is possible and a typical selection would be

4 at 30  3 at 60  2 at 100  1 at 150.

The first figure (4) is the number of ordinate deflection readings on a scale from 1 to 9 and second figure (30) is the spacing between each reading on a scale from 1 to 99.

The above selection would result in the 30th, 60th, 90th, 120th, 180th, 240th, 300th, 400th, 500th and 650th ordinate deflection readings being recorded.

6.8.2 Methods used to record the deflected shape measurements

Curvature output from the Deflectograph is in three forms:

(i) A chart recorder which plots the deflection dish as seen by the transducer.

(ii) A line printer which types the results in the following format:

(a) 060 006 012 018 024 030 036 042 048 054 060
(b) 030 003 006 009 012 015 018 021 024 027 030

The row of results shown as (a) corresponds to the curvature measurement in the nearside wheelpath and those shown as (b) corresponds to the curvature measurement in the offside wheelpath. The first three digit number in both rows (a) and (b) represents the magnitude of the maximum deflection and the next ten three digit numbers represents the magnitude of the ordinate deflections defining the curvature. All deflection measurements are in hundredths of a millimeter.

(iii) A paper tape punch produces a paper tape for direct input into the computer, allowing the results to be processed using the DEFLEC suite of computer programmes.
6.8.3 Problems associated with recording deflected shape on a time basis

The variation in the number of ordinate deflections making up each curvature measurement was the major problem associated with recording the deflected shape on a time basis because these variations often resulted in the definition of different portions of the overall deflection dish.

Variations of this nature meant that it was not possible to detect changes in the condition of the pavement by directly comparing the recorded curvature values; an adjustment to the points defining the shape was needed.

An initial solution to the problem involved the use of the output from the line printer on board the Deflectograph. This output included the total number of ordinate deflections making up each curvature measurement and constant reference to this figure by the operator allowed him to compensate for any change by altering the relative positions of the preselected ordinate deflections. This action ensured that the same proportion of the overall deflected shape was recorded each time.
Correcting curvature measurements for the effects of temperature on the stiffness of the bituminous material is necessary to allow direct comparison of readings recorded at different temperatures on pavements of various thicknesses of bituminous material. The effect that temperature can have on the stiffness and hence the deflected shape was demonstrated using the theoretical model.

The lack of any detailed information concerning the relationship between curvature and temperature meant that use had to be made of the existing relationships between deflection and temperature. Curvature was defined as the difference between the maximum and ordinate deflection readings (differential deflection) and both of these deflections were temperature corrected to equivalent values at 20°C, using the relevant deflection and temperature relationship in LR833. The appropriate temperature correction chart was selected on the basis of the overall thickness of bituminous material, and proportion of dense bituminous material present in the pavement. The equivalent differential deflection (curvature measurement) at 20°C was calculated as the difference between the temperature corrected maximum and ordinate deflection readings.

Although this method of accounting for the influence of temperature on the stiffness of the bituminous material is not ideal, it can be argued that the previously derived relationship between maximum deflection and temperature cannot be all that different from a curvature and temperature relationship when curvature is defined as the difference between the maximum deflection and an ordinate deflection 200 mm from the point of maximum deflection.

6.9.1 The effect of temperature correction on the magnitude of the deflection readings.

Table 6.11 shows the effect of temperature correction on a range of maximum, ordinate and differential deflections, initially recorded
<table>
<thead>
<tr>
<th>Thickness of Bituminous Material (mm)</th>
<th>Recorded Maximum Deflection $\frac{1}{100}$th mm</th>
<th>Recorded Ordinate Deflection $\frac{1}{100}$th mm</th>
<th>Differential Deflection $\frac{1}{100}$th mm</th>
<th>Equivalent Maximum Deflection $\frac{1}{100}$th mm</th>
<th>Equivalent Ordinate Deflection $\frac{1}{100}$th mm</th>
<th>Equivalent Differential Deflection $\frac{1}{100}$th mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>25</td>
<td>6</td>
<td>19</td>
<td>32</td>
<td>10</td>
<td>22</td>
</tr>
<tr>
<td>40</td>
<td>31</td>
<td>15</td>
<td>16</td>
<td>38</td>
<td>20</td>
<td>18</td>
</tr>
<tr>
<td>40</td>
<td>47</td>
<td>26</td>
<td>21</td>
<td>57</td>
<td>33</td>
<td>24</td>
</tr>
<tr>
<td>40</td>
<td>54</td>
<td>31</td>
<td>23</td>
<td>65</td>
<td>38</td>
<td>27</td>
</tr>
<tr>
<td>40</td>
<td>65</td>
<td>36</td>
<td>29</td>
<td>77</td>
<td>44</td>
<td>33</td>
</tr>
<tr>
<td>40</td>
<td>71</td>
<td>40</td>
<td>31</td>
<td>84</td>
<td>49</td>
<td>35</td>
</tr>
<tr>
<td>40</td>
<td>81</td>
<td>47</td>
<td>34</td>
<td>95</td>
<td>57</td>
<td>38</td>
</tr>
<tr>
<td>40</td>
<td>94</td>
<td>57</td>
<td>37</td>
<td>111</td>
<td>68</td>
<td>43</td>
</tr>
<tr>
<td>100</td>
<td>43</td>
<td>28</td>
<td>15</td>
<td>65</td>
<td>41</td>
<td>24</td>
</tr>
<tr>
<td>100</td>
<td>52</td>
<td>36</td>
<td>16</td>
<td>79</td>
<td>53</td>
<td>26</td>
</tr>
<tr>
<td>100</td>
<td>64</td>
<td>44</td>
<td>20</td>
<td>99</td>
<td>66</td>
<td>33</td>
</tr>
<tr>
<td>100</td>
<td>71</td>
<td>50</td>
<td>21</td>
<td>110</td>
<td>77</td>
<td>33</td>
</tr>
<tr>
<td>100</td>
<td>83</td>
<td>60</td>
<td>23</td>
<td>119</td>
<td>85</td>
<td>34</td>
</tr>
</tbody>
</table>
on a number of pavements with 40 mm and 100 mm of bituminous material.

A comparison of the measured and standardised values shows that the temperature correction procedure affects the magnitude of both the maximum and the differential deflections. Variations between the measured and the temperature corrected values will be dependent upon the magnitude of the original readings, the temperature at the time of the survey and the temperature correction chart selected; (the latter being dependent upon the thickness of bituminous material in the pavement).

Differences in the magnitude of the measured and temperature corrected values of maximum and differential deflection are not important because the bituminous layer thickness estimation chart is based upon temperature corrected measurements and will be used in conjunction with temperature corrected curvature readings to estimate thicknesses of bituminous material.
6.10 SELECTING LENGTHS OF PAVEMENT WITH A RELATIVELY CONSTANT THICKNESS OF BITUMINOUS MATERIAL

The existing construction information, gathered from the coring programme, was used to identify relatively long lengths of pavement with constant thicknesses of bituminous material, Table 6.12.

<table>
<thead>
<tr>
<th>SITE</th>
<th>Chainage (m)</th>
<th>Length (m)</th>
<th>Thickness of bituminous material (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tolchmoor Gate</td>
<td>3275 to 4510</td>
<td>1235</td>
<td>40</td>
</tr>
<tr>
<td>Cornwood</td>
<td>0 to 380</td>
<td>380</td>
<td>100</td>
</tr>
<tr>
<td>Cadover Bridge</td>
<td>6600 to 6800</td>
<td>200</td>
<td>100</td>
</tr>
<tr>
<td>Okehampton</td>
<td>2450 to 2650</td>
<td>200</td>
<td>100</td>
</tr>
<tr>
<td>Totnes</td>
<td>600 to 750</td>
<td>150</td>
<td>150</td>
</tr>
</tbody>
</table>

Table 6.12

The existing data was supplemented with that from additional cores if it was thought that the original core spacing was too great to assume constant thickness between them.
6.11 DERIVING A RELATIONSHIP BETWEEN CURVATURE AND MAXIMUM DEFLECTION FOR PAVEMENTS WITH VARIOUS THICKNESSES OF BITUMINOUS MATERIAL

The temperature corrected deflected shape measurements recorded on the lengths of road detailed in Table 6.11 were used to investigate a relationship between curvature, defined as the differential deflection 200 mm from the point of maximum deflection, and maximum deflection for pavements with 40 and 100 mm of bituminous material, Figure 6.29 and 6.30, respectively.

The differential deflection 200 mm from the point of maximum deflection was used to define curvature because the results from the theoretical work had indicated that changes in the magnitude of this offset, as a result of changes in the thickness of the bituminous material, were large enough to be recorded by the equipment on board the Deflectograph.

6.11.1 Defining a curvature and maximum deflection relationship.

Reference to Figures 6.29 and 6.30 will show that a basic relationship appears to exist between the two variables; namely that for a given thickness of bituminous material, the differential deflection increases with increase in maximum deflection. This relationship is in agreement with that derived from the analysis of the theoretical response of the pavement model.

There is a great deal of variation in the magnitude of the deflected shape measurements and it is difficult to identify the exact reasons for this, because unlike the theoretical work, it was not possible to define a characteristic response for a pavement with a given thickness of bituminous material and then investigate the effects on that response of variations in the moduli and thickness of the pavement layer and subgrade.

However, an attempt was made to minimise the influence of very weak pavement and subgrade layers on the curvature and maximum deflection relationship by only considering curvature readings recorded on pavements showing no sign of visual distress at the surface, i.e.
Figure 6.29 Relationship between curvature and maximum deflection for pavements with 40 mm of bituminous material.
Figure 6.30 Relationship between curvature and maximum deflection for pavements with 100 mm of bituminous material.
pavements in a SOUND condition. Rutting and cracking of the pavement's surface are generally good indicators of weakness in the pavement layers or subgrade.

Having kept the influence of large changes in the moduli and thickness on the relationship to a minimum it is fair to assume that the wide range of readings could be due either to smaller changes in the pavement layer moduli and thickness, or more probably to changes in the strength of the subgrade.

This assessment is borne out by the theoretical relationships, Figure 6.16, which indicates that the subgrade stiffness can have a much greater effect on the magnitude of the maximum deflection than on the magnitude of the differential deflection, a characteristic opposite to the effect that the pavement layer moduli and thicknesses can have on the relationship.

6.11.2 Analysis Techniques

Having decided that the range of readings reflects the effect of the subgrade strength on the relationship between curvature and maximum deflection, a least squares regression analysis was carried out to identify the 'best fit' line to define the relationship.

This analysis was performed assuming initially that the maximum deflection and subsequently that the differential deflection was the independent variable.

The 'best fit' relations determined from this analysis only differed by a small amount and the average of the two relations has been adopted in each case and shown in Figures 6.29 and 6.30.

The 'best fit' lines, shown in Figures 6.29 and 6.30 have been reproduced onto the same diagram, Figure 6.31, to allow an appreciation to be made of their relative positions.

The two constants in the equation defining the 'best fit' lines are the intercept with the 'y' (differential deflection) axis and the slope.
Figure 6.31 Relationship between curvature (differential deflection) and maximum deflection for pavements with 40 mm and 100 mm of bituminous material.
The intercepts for the two 'best fit' lines are dissimilar, 9.52 for the line through the data recorded on pavements with 40 mm of bituminous material and 5.51 for the line through the data recorded on pavements with 100 mm of bituminous material.

The slopes for the two 'best fit' lines are about the same, 0.258 for the line through the data recorded on pavements with 40 mm of bituminous material and 0.27 for the line through the data recorded on pavements with 100 mm of bituminous material.

Reference to Figures 6.29 and 6.30 indicates that the two samples of data overlap and there is therefore a need to show that, although the slopes of the two lines are similar, they define the best straight lines for samples of data that are significantly different, i.e. they are not drawn from the same population.

This can be investigated by applying the Student's t test to the null hypothesis that the two samples being compared are drawn from the same population.

If the samples belong to the same population then the sample means (being means of random samples) are normally distributed about the population mean, even if the distribution within the samples is not normal.

The combined (population) variance $S_c^2$ is estimated by pooling the sums of the squares of the residuals $(x-x)$ of both samples and dividing by the total number of degrees of freedom.

Thus, $S_c^2 = \frac{\sum (x_1 - \bar{x_1})^2 + \sum (x_2 - \bar{x_2})^2}{(n_1-1) + (n_2-1)}$

The standard deviation of the difference of the means is thus

$$S_d = \sqrt{\frac{S_c^2}{n_1} + \frac{S_c^2}{n_2}}$$
The significance of the difference is measured by the ratio of the difference to its standard deviation, and is denoted by $t$, so that

$$t = \frac{\bar{x}_1 - \bar{x}_2}{S_d}$$

For the two samples of data presented in Figures 6.29 and 6.30

$$S_c^2 = \frac{18096 + 107529.76}{59 + 65} = 1013.11$$

$$S_d = \sqrt{\frac{1013.11 + 1013.11}{60 + 66}} = 5.68$$

$$t = \frac{122.4 - 73.1}{5.68} = 8.68$$

The number of degrees of freedom $= (n_1-1) + (n_2-1) = 124$.

Reference to the standard statistical table for the $t$ test indicates that the calculated $t$ is greater than the tabulated value at the 1 per cent level of significance. The null hypothesis is rejected and it is concluded that the difference is significant, i.e. the two samples are not drawn from the same population. This is important because it suggests that there is a basic difference in the curvature and maximum deflection relationships for pavements with 40 mm and 100 mm of bituminous material.

N.B. A least squares regression analysis was also carried out on the deflected shape measurements recorded on a 150 m section of the road at Totnes. This section of road was the only available length with a bituminous layer thickness of 150 mm and, as a result, the total number of deflected shape readings was small compared to the number available for pavements with 40 and 100 mm of bituminous material.

The relationship suggested by the least squares regression analysis differed from those predicted for pavements with 40 and 100 mm of bituminous material due possibly to the influence of outliers on the small sample of measurements.
6.11.3 Correlation Coefficient

A test of goodness of fit was carried out on both sets of data and in each case the correlation coefficient was calculated as +0.6.

The correlation coefficient \( r \) must lie between +1 and -1. For \( r = +1 \), all the observed points lie on a straight line which has a positive slope; for \( r = -1 \), all the observed points lie on a straight line that has a negative slope.

It is possible to perform a significance test to see if the observed correlation coefficient is significantly different from zero.

If there is no correlation between the variables it is still possible that a spuriously high sample correlation value may occur by chance.

A two tailed test is appropriate for a positive or negative correlation.

The correlation coefficient is significantly different from zero at the \( \alpha \) level of significance if

\[
\left| \frac{r \sqrt{(n-2)}}{\sqrt{1-r^2}} \right| > t_{\alpha/2, n-2}
\]

For pavements with 40 mm of bituminous material, \( r = 0.6 \) and \( n = 60 \).

Therefore we have

\[
\frac{0.6 \sqrt{(60-2)}}{\sqrt{1-0.6^2}} = 5.7
\]

For pavements with 100 mm of bituminous material \( r = 0.6 \) and \( n = 66 \).

Therefore we have

\[
\frac{0.6 \sqrt{(66-2)}}{\sqrt{1-0.6^2}} = 6.0
\]

In both cases \( t_{0.025, 60} = 2.00 \). Thus the correlation between curvature and maximum deflection is significant at the 5% level, i.e. there is, in both cases, a high degree of correlation between the two variables.
6.12 COMPARISON OF THE PREDICTED AND EXPERIMENTAL CURVATURE AND MAXIMUM DEFLECTION RELATIONSHIPS FOR PAVEMENTS WITH VARIOUS THICKNESSES OF BITUMINOUS MATERIAL

Analysis of the in situ deflected shape measurements using a linear regression technique has resulted in the construction of a chart showing the relationship between curvature and maximum deflection for pavements with 40 and 100 mm of bituminous material, Figure 6.31. This chart could be used, together with temperature corrected deflected shape measurements, to give an estimate of bituminous layer thickness.

6.12.1 Comparing the predicted and experimental relationships

The experimental relationships derived, Figure 6.31, are similar to those derived theoretically, Figure 6.16, but their numerical values differ from those predicted; for a given value of maximum deflection the corresponding differential deflection is greater than that suggested by the theoretical predictions.

6.12.1.1 Significance test for slope

To determine whether there is a significant difference between the theoretical slope \( b_0 \) and that given by the regression line \( b \) a \( t \) test was applied to \( |b - b_0| \).

The standard deviation of \( |b - b_0| \) is equal to the standard deviation of \( b \), \( S_b \), because the theoretical value of \( b \) is free from error.

Thus, \( t = \frac{|b - b_0|}{S_b} \)

(i) For a pavement with 40 mm of bituminous material, \( b = 0.258 \), \( b_0 = 0.125 \) and \( S_b = 0.045 \).

Therefore, \( t = \frac{0.258 - 0.125}{0.045} = 2.96 \)

The number of degrees of freedom = 60 and reference to the appropriate statistical table indicates that the calculated value is greater than the tabulated value at the 1 per cent level of significance, i.e. there is a significant difference between \( b \) and \( b_0 \).
(ii) For a pavement with 100 mm of bituminous material, \( b = 0.27 \), \( b_0 = 0.121 \) and \( S_b = 0.034 \).

Therefore, \( t = \frac{0.27 - 0.121}{0.034} = 4.38 \)

The number of degrees of freedom = 64 and reference to the appropriate statistical table indicates that the calculated value is greater than the tabulated value at the 0.1 per cent level of significance, i.e. there is a significant difference between \( b \) and \( b_0 \).

6.12.1.2 Significance test for intercept

A similar test to that applied to the slopes was applied to the intercepts.

In this case \( t = \frac{|a - a_0|}{S_a} \) where \( a \) is the intercept given by the regression line, \( a_0 \) is the theoretical intercept and \( S_a \) is the variance of the intercept \( a \).

(i) For a pavement with 40 mm of bituminous material
\( a = 9.52, a_0 = 4.6, S_a = 3.356. \)

Therefore, \( t = \frac{9.52 - 4.6}{3.356} = 1.47 \)

(ii) For a pavement with 100 mm of bituminous material
\( a = 5.51, a_0 = 2.6, S_a = 4.38. \)

Therefore, \( t = \frac{5.51 - 2.6}{4.38} = 0.662 \)

In both cases the calculated \( t \) is less than the tabulated value at the 10 per cent level of significance, i.e. there is no significant difference between \( a \) and \( a_0 \).
These tests have indicated that in both cases the slopes of the lines are significantly different but there is no significant difference between the intercepts.

This could be interpreted as suggesting that the theoretical model was more capable of defining the response of the stiffer pavements, characterised by small maximum and differential deflections, than the response of weaker pavement structures.

6.12.1.3 Basic Differences between Pavement Structures

Although the difference between the theoretical and empirical relationships may be due to such factors as the temperature correction procedure and material variability (thickness and/or stiffness) in the constituent pavement layers, a large influence on the results would almost certainly have come from the structural differences between the pavements analysed theoretically and those investigated practically.

There were two basic differences between the structures. The first was the thickness of granular material below the bituminous layer; the pavement structure modelled theoretically had a 300 mm granular roadbase on top of a 150 mm granular sub-base, whereas the pavement structures surveyed with a Deflectograph had granular layers whose thickness varied about a mean value of 100 mm.

A difference in thickness of 350 mm would affect the response of virtually any pavement structure and could almost certainly account for the differences between the predicted and experimental curvature and maximum deflection relationships.

The second difference was in the assumed modulus of the subgrade.
The structure modelled theoretically was assumed to have a subgrade modulus equivalent to a CBR of 15; the actual structure was built as part of the small road system at TRRL on a subgrade that was known to be particularly strong.

Subgrade evaluation work undertaken as part of an associated project indicated a CBR range of 4 to 7 for the sites at Cornwood, Tolchmoor Gate, Cadover Bridge and Okehampton.

6.12.2 Adjusting the structure of the theoretical model to more closely resemble that of the pavements surveyed with the Deflectograph

In an attempt to bring the predicted relationships more in line with those determined from the actual deflected shape measurements, the overall thickness of the granular layers of the theoretical model was reduced from 450 mm to 100 mm for pavements with 40 and 100 mm of bituminous material and the modulus of the subgrade was reduced from $1.5 \times 10^8$ N/m$^2$ to $5 \times 10^7$ N/m$^2$. The effect of these reductions was to increase the maximum deflection by a factor of 2.8 and the differential deflection by a factor of 3.0 for a pavement with 40 mm of bituminous material and to increase the maximum deflection by a factor of 2.4 and the differential deflection by a factor of 2.6 for a pavement with 100 mm of bituminous material.

6.12.2.1 Comparing the predicted relationships with the experimental relationships

The predicted response measurements have been superimposed on a chart showing the curvature and maximum deflection relationships derived from deflected shape measurements recorded on actual pavement structures, Figure 6.32. Reference to Figure 6.32 will show that the considerable increase in both the maximum and differential deflections, as a result of the decrease in the thickness of the granular layer and modulus of the subgrade, has brought the theoretical relationships closer to the experimental relationships. However, the theoretical response is still less than the actual response, a fact that would result in an underestimate of bituminous layer thickness if the theoretical relationships were used in conjunction with the temperature corrected curvature measurements.
Figure 6.32 Predicted curvature and maximum deflection relationships superimposed on those derived from actual deflected shape measurements.
6.12.2.2 Comparing the predicted relationships for pavements with 100 mm granular layer with those for pavements with 450 mm granular layer

The predicted response of pavements with 100 mm of granular material has been superimposed on the chart showing the predicted response of pavements with 450 mm of granular material, Figure 6.33.

The effect of a decrease in both thickness of the granular layer and modulus of the subgrade was very much as expected; the overall strength of the pavement has decreased as indicated by the increased values of maximum and differential deflection.

A decrease in both the thickness of the granular layer and the modulus of the subgrade has had a considerably greater effect on the response of a pavement with only 40 mm of bituminous material. The reason for this is that pavements with thin layers of bituminous material derive a large proportion of their overall strength from the stiffness of the granular material and the response of a thinner pavement is influenced more by the strength of the subgrade, whereas pavements with thicker layers of bituminous material derive a large proportion of their overall strength from the stiffness of the bituminous material.

The movement of the points defining the relationship between curvature and maximum deflection, corresponding to decreases in both the thickness of the granular layer and modulus of the subgrade, has shown the need to accurately define the absolute characteristic response for the pavement structure on which the curvature measurements are to be made. This characteristic response could be modelled theoretically and the assumed structure altered to show the effect of changes in the properties and thickness of the constituent layers on the relationship between curvature and maximum deflection. This relationship would then form the basis of a bituminous layer thickness estimation chart appropriate to the structure to be surveyed by a Deflectograph.
Figure 6.33 Chart showing the predicted response of pavement with 100 mm of granular material superimposed on that for pavements with 450 mm of granular material.
An assessment was made of the accuracy of prediction possible with the empirical bituminous layer thickness estimation chart by comparing the estimates of the thickness with those deduced from destructive and non-destructive techniques.

The non-destructive surface wave propagation equipment has been operated at a single frequency to determine the changes in thickness of the bituminous material.

The advantage of the single frequency technique is that it can give a continuous estimate of the thickness of the bituminous material and so reduce the degree of uncertainty associated with the estimation of bituminous layer thickness from cores spaced at predetermined intervals along a road. A disadvantage is the effect that temperature can have on the estimates of the thickness of bituminous material.

Good agreement has been shown to exist between the estimates of thickness suggested by the single frequency technique and the actual thicknesses indicated by the cores.

It was possible using the continuous estimate of thickness to subdivide the section of road into two lengths with different nominal thicknesses of bituminous material. This provided an ideal opportunity to assess the accuracy of prediction possible with the empirical bituminous layer thickness estimation chart.

Temperature corrected deflected shape measurements were used in conjunction with the empirical relationships between curvature and maximum deflection to estimate the thickness of bituminous material.

In general, the estimates of thickness were quite good although some could possibly have been influenced by the effects of changes in the strength of the lower pavement layers.
USE OF A NON-DESTRUCTIVE TECHNIQUE TO ESTIMATE CHANGES IN THE
THICKNESS OF A BITUMINOUS LAYER OF A FLEXIBLE PAVEMENT AND VERIFICATION
OF THESE CHANGES BY COMPARING THEM WITH THE ACTUAL THICKNESS OF
EXTRACTED CORES

The surface wave propagation method is a non-destructive technique
which will often provide information about the dynamic modulus and
thickness of each layer of a road. The experimental technique and
theoretical background of the method have been given elsewhere;\(^{59,60}\)
details are presented of the use of the technique at a single
frequency to identify changes in the thickness of the bituminous
layer of a flexible pavement and the verification of these changes
by comparing them with the actual thicknesses of extracted cores.

7.1.1 Theoretical Basis

The wave propagation test consists of generating continuous vibrations
at the surface of the construction under test and measuring the wave
length and phase velocity of the waves which travel along the surface
of the construction. A graph of phase velocity against wavelength
(dispersion curve) is derived from the measurements, Figure 7.1,
and the shape of the curve
joining the points on the
graph is characteristic of
the structure under test.

![Phase Velocity vs Wavelength Graph](image.png)

**Figure 7.1 The Dispersion Curve**

A method of analysing such experimental data has been presented by
Jones\(^{(59)}\) who has shown that the theory given by Lamb\(^{(61)}\) for the
propagation of waves along a plate, yields an adequate solution for
the relation between velocity and wavelength.
Lamb showed that the vibrations propagated along a plate could be of two kinds: symmetrical vibrations, often called longitudinal vibrations, and asymmetrical vibrations, often called flexural vibrations. Both types of vibration have displacements perpendicular to the surface of the plate, and also parallel to the surface in the direction of propagation. The maximum displacement of the flexural vibration is perpendicular to the surface and that for the longitudinal vibration is in the direction of propagation. Both types of vibration propagate with a velocity which changes with frequency (or wavelength) and the corresponding relations depend upon the thickness and elastic properties of the plate.

Typical experimental results are shown in Fig. 7.1 and it will be seen that the velocity approaches an upper limit as the wavelength approaches zero. This upper limit is referred to as the velocity of Rayleigh surface waves and is denoted by the symbol $\gamma$.

The Rayleigh wave velocity $\gamma$ is related to the modulus of elasticity ($E$) and density ($\rho$) of the material by the relation

$$\gamma = k \left( \frac{E}{\rho} \right)^{\frac{1}{4}}$$

where $k$ has a value which is dependent upon the Poisson's ratio of the material.

The theoretical relation between the velocity ($c$) of the surface waves and the wavelength ($\lambda$), in Lamb's analysis, can be conveniently expressed as a relation between the non-dimensional parameters $c/\gamma$ and $\lambda/H$, where $H$ is the thickness of the layer.

Table 7.1 shows computed values of $\lambda/H$ for several values of $c/\gamma$ for different Poisson's ratios.
<table>
<thead>
<tr>
<th>$\frac{c}{f}$</th>
<th>$\frac{\lambda}{H}$ (v = 0.25)</th>
<th>$\frac{\lambda}{H}$ (v = 0.33)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.8</td>
<td>2.226</td>
<td>2.328</td>
</tr>
<tr>
<td>0.7</td>
<td>3.070</td>
<td>3.216</td>
</tr>
<tr>
<td>0.6</td>
<td>4.080</td>
<td>4.274</td>
</tr>
<tr>
<td>0.5</td>
<td>5.382</td>
<td>5.637</td>
</tr>
<tr>
<td>0.4</td>
<td>7.212</td>
<td>7.550</td>
</tr>
<tr>
<td>0.3</td>
<td>10.111</td>
<td>10.580</td>
</tr>
<tr>
<td>0.2</td>
<td>15.691</td>
<td>16.413</td>
</tr>
</tbody>
</table>

Table 7.1 Computed values of $\frac{\lambda}{H}$ for several values of $\frac{c}{f}$

The computations given in Table 7.1 provide the basis for estimating layer thickness from experimental surface wave data and an example of their use now follows:

Consider the experimental data given in Figure 7.2.

Figure 7.2 Experimental Results
The extrapolated value of the Rayleigh's wave velocity for the data given in Figure 7.2 is 1400 m/s. For the curve drawn in Figure 7.2 the values of $c$ at $\frac{c}{\gamma} = 0.8, 0.7, 0.6$ and 0.5 are tabulated in column (1) for the extrapolated value of $\gamma$ equal to 1400 m/s. The values of $\lambda$ at each velocity are read off the curve in Fig. 7.2 and are tabulated in column (2). The theoretical values of $\frac{\gamma}{H}$ (where $H$ is the thickness of the layer for Poisson's ratio of 0.33) have been taken from Table 7.1 and are tabulated for each $\frac{c}{\gamma}$ in Table 7.2. The estimated thickness of the layer is obtained by dividing the wavelength (column 2) by the ratio ($\frac{\lambda}{H}$). The average of the four estimated thickness values is taken as being representative of the true thickness of the layer.

An alternative method of analysing the experimental data is to use a perspex curve to determine the value of the Rayleigh wave velocity and the thickness of the layer.

The shape of the perspex curve (Figure 7.3) is derived from the relationships given in Table 7.1 and a separate curve exists for each Poisson's ratio.

Superimposed on each perspex curve is a scale showing values of equivalent layer thickness, the positions of which are again derived from the relationships in Table 7.1.

The perspex curve is used to define the best 'fair curve' through the experimental data and the point of intersection between it and the Phase Velocity Axis (Fig. 7.2) is the appropriate value of the Rayleigh wave velocity.
velocity. The thickness of the layer is defined by the position of
the Phase Velocity Axis on the superimposed thickness scale.

Figure 7.3 Perspex Curve used to determine the Rayleigh Wave Velocity and thickness of the layer.

7.1.2 Use of the Surface Wave Propagation Method at a Single Frequency

A criticism of the wave propagation method is that it is time consuming, often taking several hours of testing to determine the material properties and thicknesses which are only appropriate to the pavement structure directly under the vibrator. The time element associated with the complete technique has made its use as a method for monitoring material strength or thickness at short intervals over long lengths of pavement extremely unattractive.

Jones and Mayhew (62) have described a method of using the equipment at a single frequency to study the variability of cemented material over
relatively long lengths of construction rather than separate estimates of thickness and elastic properties over short test lengths. They have reported achieving a continuous and fairly rapid survey of up to 100 m/hr of construction using the surface wave method at a single frequency.

Selection of an appropriate frequency is important to ensure that the waves propagated in the upper bound layer of the pavement are at a velocity greater than that of the compressional waves in the underlying layer.

As a guide to meeting this requirement it was suggested that the test frequency should have a wavelength of between two and six times the thickness of the bound layer.

Having calculated the wavelength in this manner the value was then used in conjunction with a dispersion curve, built up from measurements along a test length at several frequencies, to determine the test frequency. This technique was successfully applied by Jones and Mayhew to detect areas of weakness along a length of cemented base material.

Operation of the wave propagation equipment in this way results in the use of wavelength and phase velocity as a continuous measure of the quality of the material in question. Factors which influence this measure of quality are the layer thickness and the strength (density and elastic characteristics).

Use of the single frequency technique to identify changes in thickness or strength is only possible if it can be assumed that the parameter not being investigated remains relatively constant.

Application of the single frequency technique to bituminous materials is hampered by the variation of their elastic properties with frequency and temperature. Jones states that the effect of visco-elasticity is to cause the phase velocity to change with frequency resulting in the experimental dispersion curve deviating below the theoretical curve. However, Jones continues to state that most of
the change occurs within the lower frequency range and the changes become much smaller over the part of the frequency range for which the results fall near to the curve for the propagation of free vibrations in a plate. This part of the frequency range is that utilised by the single frequency technique and it was therefore thought that the effect of frequency on the phase velocity would not be a major problem.

The influence of temperature on the visco-elastic properties of the bituminous material was identified as a problem and an ideal way of overcoming it would have been to conduct the survey at a constant temperature. This was thought to be too restrictive to be practical and it was decided to conduct the survey and record any change in temperature to allow a correction procedure to be developed.

7.1.3 Application of the Single Frequency Technique to Determine Changes in the Thickness of the Bituminous Layer of a Flexible Pavement

The wave propagation equipment has been used in this project to determine changes in the thickness of the bituminous layer over a length of flexible pavement using the single frequency technique.

The length chosen was a section of the unclassified road at Cornwood, which was not designed, but built up from a series of strengthening measures. Data determined from the coring programme indicated a probable change of bituminous layer thickness over this length.

7.1.3.1 Defining two points of different bituminous layer thickness

It was first necessary to define two points along this section having different bituminous layer thicknesses. This was achieved using the surface wave propagation equipment at various locations and then fitting the theoretical 'free plate' curve appropriate to a Poisson's ratio of 0.33 to the experimental data, to obtain the dispersion curves.

Analysis of the dispersion curve for location A, (Figure 7.4), gave an equivalent bituminous layer thickness of 134 mm and a Rayleigh wave
Figure 7.4 Dispersion curve for Location A - Cornwood Site.
velocity of 1480 m/s, from measurements recorded at 11°C.

Analysis of the dispersion curve for location B (Figure 7.5) gave an equivalent bituminous layer thickness of 112 mm and a Rayleigh wave velocity of 1420 m/s, from measurements recorded at 15°C.

The chainages shown are those used to define the overall test section of this unclassified road.

Cores were removed from locations A and B revealing actual bituminous layer thicknesses of 160 mm and 120 mm respectively.

The thickness of bituminous material measured from the cores was greater in both instances than that suggested from the analysis of the dispersion curves.

The wave propagation equipment demonstrated a capability of prediction of thickness with an accuracy which was consistent with the technique used and the variable properties of the material under investigation.

7.1.3.2 Operation of the equipment at a single frequency

The wave propagation equipment was operated between locations A and B at a single frequency of 1500 Hz. The selection of this frequency was based upon the suggestion previously stated that the test frequency should have a wavelength of between two and six times the thickness of the bituminous layer.

The survey was conducted in the nearside wheelpath and measurements of phase change were made at 100 mm intervals. The phase change measurements were plotted against distance increment (Figure 7.6) by the operator and the wavelength was determined by drawing a straight
Figure 7.5 Dispersion curve at Location B - Cornwood Site
line through the points and calculating the inverse slope, i.e.
\[
\text{wavelength} = \frac{\text{Distance}}{\text{Phase Change}}.
\]

Figure 7.6 Phase Change and Distance Graph

7.1.3.3 Analysis of the Wavelength and Phase Velocity Data

The phase change and distance increments for the total length surveyed were plotted as a continuous trace, shown diagrammatically in Figure 7.7.

Figure 7.7 Typical Experimental Phase Change and Distance Plot
The overall shape of the trace was not of constant slope, which would indicate similar thickness over the length, but was composed of several different slopes, indicating a variation in thickness.

The wavelength (\( \lambda \)) associated with each major slope change was calculated as the inverse slope, i.e. \( \frac{\text{Distance}}{\text{Phase Change}} \).

The phase velocity \( (c) \) was then calculated using the formula \( c = f \cdot \lambda \) where \( f \) was the frequency equal in all cases to 1500 Hz and \( \lambda \) was the calculated wavelength.

Table 7.3 shows the wavelength and phase velocity associated with each major slope change.

7.1.3.4 Equivalent Depth

Figure 7.8 indicates the way in which the theoretical curve, originally fitted to the experimental data in Figure 7.6, was used in conjunction with the calculated wavelengths and phase velocities to determine the equivalent depths of bituminous material.

Three positions of the theoretical curve are shown in Figure 7.8.

**Position 1** was obtained by fitting the theoretical curve to the experimental data (Figure 7.6) to identify the Rayleigh Wave Velocity, the intercept of the curve and the phase velocity axis, and the thickness of the bituminous layer at location A.

**Position 2** shows the theoretical curve moved to the right of the original curve position and passing through two points. The first point is identified by a phase velocity equal to 991 m/s and a wavelength of 0.66 m and the second point is the Rayleigh Wave Velocity established initially. Moving the curve in this way establishes an equivalent thickness of bituminous material of 200 mm suggested by a phase velocity of 991 m/s.

**Position 3** shows the theoretical curve moved to the left of the original curve position and passing through the Rayleigh Wave.
Figure 7.8 Use of the theoretical curve to determine layer thickness.
Velocity and a point identified by a phase velocity of 649 m/s and a wavelength of 0.433 m. The position of this curve indicates an equivalent bituminous layer thickness of 66 mm for a phase velocity of 649 m/s.

The use of a single dispersion curve to determine equivalent layer thicknesses from the calculated phase velocities and wavelengths is only possible if the material under investigation is not significantly affected by temperature; if this is the case any change in wavelength and phase velocity could be attributed to a change in the thickness of the material.

However, this is not the case for bituminous materials because they are temperature susceptible and the effect of temperature on the material properties will influence the phase change measurements.

The effect of temperature on the phase change measurements has been reported by Page and Mayhew \(^{(63)}\) who have shown that an increase in temperature will result in a decrease in the calculated Rayleigh wave velocity.

This effect of temperature on the value of the Rayleigh wave velocity is important because the previously described method of estimating equivalent bituminous layer thickness is dependent upon the assumed value of the Rayleigh wave velocity.

A method of taking the effect of temperature into account would be to construct a number of dispersion curves based upon a series of phase change measurements taken at different temperatures and calculate the Rayleigh wave velocity appropriate in each case.

Use could then be made of the dispersion curve relevant to the temperature at which subsequent phase change measurements were made to determine the equivalent bituminous layer thickness.

Unfortunately there was insufficient time to determine the Rayleigh wave velocities associated with a range of temperatures and only two values have been calculated: -
A Rayleigh wave velocity of 1480 m/s was calculated for the phase change measurements recorded at 11°C at Location A on a structure with a 160 mm thick bituminous layer.

A Rayleigh wave velocity of 1420 m/s was calculated from the phase change measurements recorded at 15°C at Location B, on a structure with 120 mm of bituminous material.

In an attempt to keep the temperature effect on the measurements to a minimum, Figure 7.4 was used to determine the equivalent bituminous layer thicknesses from wave lengths and phase velocities based upon phase change measurements recorded at 11°C and 12°C.

For the same reason Figure 7.5 was used to determine equivalent bituminous layer thicknesses from the wavelengths and phase velocities based upon phase change measurements recorded at 14°C and 20°C.

These equivalent bituminous layer thicknesses are given in Table 7.3 together with the associated wavelengths and phase velocities.

7.1.3.5 Actual depth of bituminous material

Cores were extracted from within each of the lengths of road shown in Table 7.3 to determine the actual thickness of bituminous material present.

Both the actual and equivalent thicknesses applicable to those locations are shown in Table 7.4.
<table>
<thead>
<tr>
<th>CHAINAGE</th>
<th>TEMPERATURE</th>
<th>WAVELENGTH</th>
<th>PHASE VELOCITY</th>
<th>EQUIVALENT DEPTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>START</td>
<td>FINISH</td>
<td>°C</td>
<td>m/s</td>
<td>mm</td>
</tr>
<tr>
<td>1372.5</td>
<td>1374.0</td>
<td>11°C</td>
<td>0.591</td>
<td>886</td>
</tr>
<tr>
<td>1387.1</td>
<td>1388.6</td>
<td>12°C</td>
<td>0.588</td>
<td>882</td>
</tr>
<tr>
<td>1400.5</td>
<td>1402.8</td>
<td>12°C</td>
<td>0.587</td>
<td>880</td>
</tr>
<tr>
<td>1409.6</td>
<td>1411.7</td>
<td>14°C</td>
<td>0.660</td>
<td>991</td>
</tr>
<tr>
<td>1438.7</td>
<td>1440.6</td>
<td>14°C</td>
<td>0.523</td>
<td>785</td>
</tr>
<tr>
<td>1461.2</td>
<td>1462.8</td>
<td>20°C</td>
<td>0.541</td>
<td>811</td>
</tr>
<tr>
<td>1475.0</td>
<td>1476.7</td>
<td>20°C</td>
<td>0.433</td>
<td>649</td>
</tr>
</tbody>
</table>

Table 7.3 Wavelength, Phase Velocity and Equivalent Depth.

<table>
<thead>
<tr>
<th>CHAINAGE (m)</th>
<th>EQUIVALENT DEPTH (mm)</th>
<th>CORE DEPTH (mm)</th>
<th>% DIFFERENCE OF ACTUAL DEPTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>1373.0</td>
<td>147</td>
<td>160</td>
<td>-8</td>
</tr>
<tr>
<td>1387.6</td>
<td>145</td>
<td>165</td>
<td>-12</td>
</tr>
<tr>
<td>1401.0</td>
<td>144</td>
<td>157</td>
<td>-8</td>
</tr>
<tr>
<td>1410.0</td>
<td>200</td>
<td>170</td>
<td>+18</td>
</tr>
<tr>
<td>1439.0</td>
<td>108</td>
<td>100</td>
<td>+8</td>
</tr>
<tr>
<td>1461.5</td>
<td>128</td>
<td>120</td>
<td>+8</td>
</tr>
<tr>
<td>1475.8</td>
<td>72</td>
<td>90</td>
<td>-20</td>
</tr>
</tbody>
</table>

Table 7.4 Comparison of Equivalent and Core Depth.
7.1.4 Conclusions

The following conclusions can be drawn from this work:-

(1) The wave propagation equipment can be used at a single frequency to determine changes in the thickness of the bituminous layer. The greatest restriction in its use was the effect that temperature can have on the properties of the bituminous material and hence estimate of thickness. Operating the equipment at a constant temperature would be too restrictive and a better approach might be to derive a series of wavelengths and phase velocity relationships for a range of pavement temperatures. The effect of temperature on the estimate of thickness could be kept to a minimum by selecting the chart appropriate to the pavement temperature at the time the measurements were recorded.

(2) The estimates of thickness suggested by this technique were in general within ±10% of the thicknesses indicated by the cores. There were, however, two estimates nearer to ±20% of the actual and these need further comment.

The fact that the greatest underestimate was associated with the thinnest bituminous layer is possibly a coincidence. What is important is that these phase change measurements were recorded at the highest pavement temperature (20°C).

Metcalf (64) has reported that lower phase velocities at high temperatures can result in an underestimate of the thickness and this may well be the case, because the dispersion curve appropriate to a pavement temperature of 15°C was used in the estimate of thickness.

It is more difficult to propose an explanation for the 18% overestimate of the thickest layer of bituminous material, because use of the dispersion curve appropriate to a pavement temperature of 15°C to estimate the thickness from measurement made at 14°C would
suggest that the effect of temperature, in this case, was minimal.

A possible answer leads on to a discussion of the other assumption made in this work; that of constant density. It is assumed that any variation in the phase velocity is due to changes in the thickness of the pavement layer in question. However, changes in the properties of the material and especially in the density could result in an over- or under-estimate of layer thickness.

Change in material properties may well have been the cause of the 18% overestimate of the thickness of the deepest bituminous layer and may also have contributed to the 20% underestimate of the thinnest bituminous layer.

It could also be argued that the likelihood of changes in the properties of the material is increased on this unclassified road, because of the fact that it was not designed but built up from a series of strengthening measures.
7.2 USING TEMPERATURE CORRECTED DEFLECTED SHAPE MEASUREMENTS AND THE EMPIRICAL CURVATURE AND MAXIMUM DEFLECTION RELATIONSHIPS TO ESTIMATE THE THICKNESS OF BITUMINOUS MATERIAL

It was possible from the wave propagation work to subdivide the 100 m section of road into two lengths on the basis of the thickness of the bituminous layer; the thickness of the bituminous layer from ch. 1371 to ch. 1435.5 was about 165 mm and from ch. 1435.5 to ch. 1482.8 the thickness was about 100 mm.

The accuracy of prediction of the empirical curvature and maximum deflection relationships was then assessed by using these relationships in conjunction with deflected shape measurements recorded between ch. 1371 and 1428.8 to indicate the thickness of the bituminous layer. The empirical curvature and maximum deflection relationships were used because they were derived from deflected shape measurements recorded on pavement structures similar to that now being investigated.

7.2.1 Temperature correcting the curvature measurements

Both the recorded and temperature corrected deflection values making up the curvature measurements are shown in Table 7.5. Also shown in Table 7.5 are the chainages defining the points of measurement and the thicknesses of bituminous material present.

In all cases the temperature corrected deflections are greater than those recorded, due to the correction procedure of

(i) converting the Deflectograph readings to equivalent Deflection Beam values;

(ii) converting the Deflection Beam readings to equivalent values at 20°C.

The readings in Table 7.5 marked with an asterisk need further comment because they are untypical; in each case the magnitude of the differential deflection 200 mm from the point of maximum deflection is equal to the magnitude of the maximum deflection.
The unclassified road between Ch. 1371.0 and 1482.8.

Table 7.5: Recorded and corrected deflection readings for section of

<table>
<thead>
<tr>
<th>Chainage (m)</th>
<th>Thickness (mm)</th>
<th>Material</th>
<th>Corrected Maximum Ordinate Deflection (100th mm)</th>
<th>Recorded Maximum Ordinate Deflection (100th mm)</th>
<th>Corrected Differential Deflection (100th mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1371.0</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1375.3</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1379.6</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1383.9</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1388.2</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1392.5</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1396.8</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1401.1</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1405.4</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1409.7</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1414.0</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1418.3</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1422.6</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1426.9</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1431.2</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1435.5</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1440.8</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1445.1</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1449.4</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1453.7</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1458.0</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1462.3</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1466.6</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1470.9</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1475.2</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1479.5</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
<tr>
<td>1483.8</td>
<td>100</td>
<td>100</td>
<td>10</td>
<td>15</td>
<td>10</td>
</tr>
</tbody>
</table>

* indicates interpolated values.
Interpreting this situation literally suggests a short, deep deflection dish.

This type of deflected shape is normally associated with either weakened pavements or pavements with a thin overall construction thickness.

Two of these anomalous deflected shape measurements corresponded to the position where cores were extracted. Subsequent examination of the cores revealed a substantial thickness of bituminous material that did not appear weakened in any way.

If these deflected shapes are not as a result of either weakened material or thin construction, then it can only be assumed that at times there appears to be a problem associated with recording ordinate deflections when the maximum deflection is small.

This conclusion is reinforced by the fact that the problem is not confined solely to pavements with relatively thick layers of bituminous material. The deflected shape measurement at ch. 1457 m also shows a similar pattern for a low maximum deflection on a pavement with 100 mm of bituminous material.

The presence of a thicker layer of bituminous material between ch. 1371 and ch. 1435.5 m would seem to be confirmed by the magnitude of the deflection readings in Table 7.5.

Both the average maximum deflection and curvature values for this length are less than those for the section with 100 mm of bituminous material.

7.2.2 Estimating the thickness of bituminous material

The corrected curvature (differential deflection) and maximum deflection readings, originally recorded on pavements with 100 and 165 mm of bituminous material, are shown plotted on the empirical bituminous layer thickness estimation chart in Figures 7.9 and 7.10 respectively.
Figure 7.9 Relationship between curvature (differential deflection) and maximum deflection for pavements with 40 mm and 100 mm of bituminous material.
Figure 7.10 Relationship between curvature (differential deflection) and maximum deflection for pavements with 40 mm and 100 mm of bituminous material.
The number of deflected shape measurements shown in Figures 7.9 and 7.10 is small. The reason for this is that the thickness of the bituminous material was only determined, using the wave propagation equipment, for relatively short lengths which meant that the number of deflected shape measurements recorded by the Deflectograph over these lengths would also be small.

The anomalous readings referred to previously are not shown in Figures 7.9 and 7.10.

The results obtained from Figures 7.9 and 7.10 are not conclusive but do at least indicate a probable change in the thickness of the bituminous material between the two sections.

Reference to Figure 7.9 shows that five out of the eight readings lie around the line defining the relationship for pavements with 100 mm of bituminous material and would therefore suggest a thickness of about 100 mm.

This is in agreement with the thickness indicated by the cores for this section of road.

Two readings lie around the line defining the relationship for 40 mm of bituminous material and one lies above it. These three readings would suggest a bituminous layer thickness of 40 mm or less; a thickness substantially below that indicated by the cores for this section of road.

Although the bituminous layer thickness estimation charts can account for the effect of the strength of the subgrade on the relationships, the effects of changes in the properties of the lower pavement layers can only be expressed as a change in the thickness of the bituminous material.

In view of the fact that the thickness of bituminous material is known to be about 100 mm, it must be assumed that the 3 readings, indicating a thickness of 40 mm or less, represent sections of road with weaker lower layers.
The accuracy of prediction that was hoped for in Figure 7.10 was less than that in Figure 7.9 because the relationship between curvature and maximum deflection had not been defined for pavements with 165 mm of bituminous material. However, it was hoped that the majority of the deflected shape measurements would lie well below the line defining the relationship for pavements with 100 mm of bituminous material, thereby suggesting thicknesses in excess of this value.

Reference to Figure 7.10 will show that three out of eight readings lie below this line, one lies on it, one lies between the two lines and three lie either on or below the line defining the relationship for pavements with 40 mm of bituminous material. The results may at first appear to cover a wide range of thicknesses, but a case for suggesting that the thickness of the bituminous material is in excess of 100 mm can be argued on two points.

Firstly, three readings lie below the line defining the relationship for pavements with 100 mm of bituminous material and secondly, and more importantly, these readings appear to lie on a line parallel to those shown to define the relationships for 40 mm and 100 mm of bituminous material.

For the reasons given previously, the readings that indicate thinner layers of bituminous material may well, in fact, be reflecting the influence of the weaker lower pavement layers on the relationship between curvature and maximum deflection.

It is also possible that the position of these readings in relation to the lines defining the relationship is due to the problems that appear to exist at times with the equipment, when recording the ordinate deflections that correspond to low maximum deflections.

7.2.3 Conclusions

This work has demonstrated that the empirical relationship between curvature and maximum deflection can be used together with deflected shape measurements to indicate the difference in thickness of bituminous material for two sections of road.
The main problem with this investigation was undoubtedly the small sample of deflected shape readings available to define the response of each section, and the fact that this number was reduced even further by anomalous readings. The frequency of anomalous readings was quite high and further work is required on strong pavements to investigate the problems that appear to exist at times when recording the ordinate deflections that correspond to low maximum deflection readings.

A number of the deflected shape measurements shown in both Figs. 7.9 & 7.10 suggested a thickness of bituminous material less than that indicated by the cores.

These deflected shape measurements either reflected the influence of changes in the strength of the lower layers of the pavement on the relationship between curvature and maximum deflection, or they represented incorrect readings.

At present there is no way of identifying which is the most likely reason. However, it may, in the future, be possible to do this by using the theoretical model to quantify the effect of changes in the strength of the lower pavement layers on the curvature and maximum deflection relationships.

The influence of changes in the lower pavement layers could be expressed as limits on either side of the lines, which define the relationship for various thickness of bituminous material, Figure 7.11.

![Diagram](image)

- Differential Deflection (\(\frac{1}{100}\)th mm)  
- Maximum Deflection (\(\frac{1}{100}\)th mm)  

The strength of the lower pavement layers is more likely to have a greater effect on the relationship for pavements with thin layers of bituminous material; (hence wider bands).

Figure 7.11 The possible effect of changes in the strength of the lower pavement layers on the relationship between curvature and maximum deflection for pavements with 40 and 100 mm of bituminous material.

221
The aim of this study was to investigate the use of deflected shape measurements, recorded with a Deflectograph, to estimate the thickness of bituminous material in a flexible pavement structure. This estimate can be used to provide a means of continuously assessing the thickness of bituminous material in a flexible pavement structure. In effect, the assessment would be made at about every 4 m along the pavement.

In the long term the development of such an estimation procedure would remove a major obstacle to on-bound real-time processing of deflection measurements obtained with a Deflectograph.

In the short term a procedure for continuously assessing the thickness of bituminous material would help to eliminate a great deal of the uncertainty that exists at present concerning the pavement construction between locations where cores have been removed.

The investigation was tackled on two fronts; a theoretical approach that used a commercially available finite element programme to model the response of a number of pavement structures and an empirical approach using measurements of pavement response recorded with a Deflectograph.

A summary of the more important aspects of this investigation is presented below.

3.1.1 Theoretical investigations

A finite element numerical model has been developed that can predict the response under load of pavements containing granular and bituminous roadbases. The theoretical model has been used to develop relationships between curvature, defined as a differential deflection 200 mm from the point of maximum deflection, and maximum deflection for pavements with various thicknesses of bituminous material.
The model has also been used to establish correlation factors that allow absolute deflected shape measurements to be converted into equivalent Deflectograph deflected shape measurements.

The relationship between curvature and maximum deflection derived from these equivalent deflected shape measurements can be used together with Deflectograph measures of pavement response to give an estimate of the thickness of bituminous material in a flexible pavement structure.

The effect of changes in the properties of the pavement layers and the subgrade on the estimate of bituminous layer thickness has been quantified using the theoretical model.

In addition, the influence of soft spots in the granular layer and single cracks in the bituminous layer on the estimate of bituminous layer thickness has also been studied.

8.1.2 Empirical investigations

Relationships between curvature and maximum deflection have been developed from the analysis of in situ deflected shape measurements recorded on a variety of pavement structures. These deflected shape measurements were recorded by a Deflectograph on several lengths of lightly trafficked road that had relatively consistent thicknesses of bituminous material. These individual sections of road amounted to about 2.2 km.

8.1.3 Comparison of the theoretical and empirical relationships between curvature and maximum deflection

The empirical relationships although similar to those derived theoretically, differed in numerical value.

The variation between the empirical and theoretical relationships was thought to be due to the considerable differences between the structures modelled theoretically and the field surveying undertaken.
with a Deflectograph.

Subsequent work with an analytical model that more closely resembled the actual pavement structures surveyed, resulted in closer agreement between the theoretical and the empirical curvature and maximum deflection relationships.

The agreement was not exact due to the number of assumptions that had to be made in deriving these relationships between curvature and maximum deflection.

The assumptions most likely to have influenced either the theoretical or the empirical curvature and maximum deflection relationships are considered later in this chapter.

8.1.4 Validation of the empirical curvature and maximum deflection relationships

An assessment of the accuracy of prediction possible with the use of the empirical relationship between curvature and maximum deflection and actual deflected measurements was undertaken by comparing the estimates of thickness with those suggested by destructive and non-destructive techniques.

Deflected shape measurements recorded with a Deflectograph along a section of road were temperature corrected and used together with the empirical curvature and maximum deflection relationships to estimate the thickness of bituminous material.

In general, these estimates of bituminous layer thicknesses compared favourably with the actual thicknesses.

A number of the estimates of thickness did vary considerably from the actual thicknesses due possibly to problems with the measuring equipment or the influence of changes in the properties of the lower pavement layers.

The length of road on which these deflected shape measurements were recorded was identified using the non-destructive surface wave
propagation equipment at a single frequency.

The wave propagation equipment was used in this way to detect changes in the thickness of the bituminous material. Comparison of the estimates of bituminous layer thickness suggested by this technique with the actual thicknesses of the extracted cores showed that agreement was generally within ± 10%.

It can be concluded from these investigations that a properly defined relationship between curvature and maximum deflection can normally be used, together with deflected shape measurements recorded by a Deflectograph, to estimate the thickness of bituminous material in a flexible pavement structure.
8.2 FACTORS INFLUENCING THE RELATIONSHIP BETWEEN CURVATURE AND MAXIMUM DEFLECTION

Several of the assumptions made during this work will have influenced the theoretical and the empirical relationships between curvature and maximum deflection and hence any estimate of bituminous layer thickness made using these relationships.

8.2.1 Plane Elasticity

The assumption that the response of the pavement to a given load could be considered as a plane elasticity problem influenced the derived relationships between curvature and maximum deflection.

A simple two-dimensional representation was selected primarily because this work was a preliminary study of a relationship between curvature and maximum deflection and its possible use for estimating the thickness of bituminous material in a pavement structure.

Representing the behaviour of the structure in this way meant that sufficiently accurate information, on which to base the investigation, could be obtained with a reasonable amount of computer time.

It was appreciated that an axisymmetric or three-dimensional model would probably have provided more detailed and accurate information, but it was felt that this additional accuracy was not necessary for the initial study.

The plane elasticity approach considers that the forces and the displacements are in one plane and that all strains perpendicular to that plane are ignored in the calculations; i.e. it is assumed that the section remains in one plane at all times.

This idealisation is not totally unrealistic. The pavement section could be considered to be loaded in one plane if it were located directly under the dual wheel assembly of the Deflectograph.

Although the zero strain assumption in the direction perpendicular to
the plane is not strictly correct, it could be argued that the magnitude of these strains in this direction is likely to be small because of the direction of the applied loads and the restriction to movement by the material on either side of the plane section.

8.2.1.1 Material behaviour

The behaviour of the materials in the pavement structure was assumed to be linear elastic.

The general implication from previous work is that:

(i) linear elasticity can be used to describe the behaviour of the materials in the pavement structure provided that realistic values of Young's Modulus and Poisson's Ratio are used.

(ii) linear elasticity lends itself more readily to the characterisation of the behaviour of bituminous materials than to the behaviour of granular materials.

For this study the method used to determine the moduli of the materials in the pavement structure was influenced by the information that was already known.

Details of the absolute deflected shapes and layer thicknesses were available; the only unknowns were the material moduli. In view of this situation it was felt that the best method of determining appropriate moduli values was by adopting a trial and error process of force fitting moduli values to the required deflected shape.

Varying the material moduli within clearly defined limits, until the response of the theoretical model closely resembled that of the actual pavement, ensured that the values adopted were as realistic as possible.

The majority of the assumed moduli values were well within the original limits used to define the typical moduli range.
A notable exception to this is the modulus of the bituminous layer and this requires further comment. A modulus of $1.5 \times 10^{10}$ N/m$^2$ was adopted for the bituminous layer.

It was appreciated at the time that a modulus of this magnitude is generally regarded as being towards the top of the probable moduli range for bituminous material and usually associated with low temperature conditions.

However, it was clear from the analysis work that a modulus of this magnitude was necessary to accurately model the deflected shape in the vicinity of the maximum deflection.

The high modulus therefore appears to be a consequence of the fact that the moduli of the materials in the structure were the only unknowns with the result that their evaluation undoubtedly reflected the behavioural assumptions inherent in the modelling technique.

It is clear from this work that the approach used to determine the moduli values has its limitations and it is proposed that further work to develop theoretical curvature and maximum deflection relationships for other pavement structures will use moduli values, derived either using the repeat load triaxial equipment or the surface wave propagation technique.

8.2.2 Correlation Factors

The use of a standard set of correlation factors to convert the Deflection Beam deflected shapes into equivalent Deflectograph deflected shapes would undoubtedly have influenced the theoretical relationship between curvature and maximum deflection.

The correlation factors used were those appropriate to a structure with a substantial thickness of granular material. At times, the thickness of granular material was greatly reduced and at the same time the thickness of bituminous material was increased.

Under these circumstances the correlation factors corresponding to
the structure with a bituminous roadbase may well have been more appropriate.

The probable effect of not using the appropriate correlation factors would be to slightly overestimate the magnitude of the differential deflection.

The use of correlation factors to determine equivalent Deflectograph deflected shapes from the response of a two-dimensional model is cumbersome and requires further attention. It is quite possible that a more convenient solution to the problem of determining Deflectograph deflected shapes could be obtained with the use of a three-dimensional model. This idea is considered later in the section entitled 'Further Work'.

There may also be a slight error in the magnitude of the correlation factors because no account was taken of the influence of the loaded front wheels on the measured deflected shapes. It was assumed that the deflected shape in front of the rear wheels gradually reduces to zero deflection, whereas in practice this gradual reduction in deflection becomes at some stage a gradual increase in deflection brought about by the weight of the front wheels of the Deflectograph.

8.2.3 Temperature correction of the deflected shape measurements

The temperature correction procedure used to standardise the deflected shape measurements recorded by the Deflectograph would have influenced the empirical relationship between curvature and maximum deflection.

The procedure adopted resulted in the temperature correction of all deflected shape measurements to equivalent values at 20°C using previously derived empirical relationships between maximum deflection and temperature.\(^{(35)}\)

Standardising the deflected shape measurements in this way effectively eliminated the influence that any temperature associated change in the stiffness of the bituminous material could have had on the
relationship between curvature and maximum deflection.

The relationships between maximum deflection and temperature were used to correct both the maximum and ordinate deflection measurements to equivalent values at 20°C.

The use of these relationships to standardise ordinate deflections 200 mm from the point of maximum deflection was not strictly correct, because the relationships between maximum deflection and temperature will almost certainly be different to the relationship between ordinate deflection, 200 mm from the point of maximum deflection, and temperature. However these relationships were used because more appropriate relationships did not exist or could not be determined in the time available.

The influence that this temperature correction procedure had on the empirical relationships between curvature and maximum deflection was relatively unimportant in terms of its possible effect on the estimates of bituminous layer thickness; the reason being that this same temperature correction procedure was used to temperature correct the deflected shape measurements that were subsequently combined with these relationships to estimate bituminous layer thickness.

However, the temperature correction procedure would still contribute to the difference between the theoretical and empirical curvature and maximum deflection relationships.

8.2.4 The properties of the lower pavement layers

Changes in the properties of the lower pavement layers will influence the empirical relationship between curvature and maximum deflection.

It was not possible to evaluate the likely magnitude of the effect of changes in the properties of the lower layers on the relationship between curvature and maximum deflection because they were unknown quantities that could not be readily defined within the time available.
8.3 CONCLUSIONS

It can be concluded that despite the limitations of the methods used and the assumptions made, this work has demonstrated that:

(i) use can be made of a finite element programme to model the response of actual pavement structures.

(ii) the theoretical model can be used to derive relationships between curvature and maximum deflection for pavements with various thicknesses of bituminous material.

(iii) similar relationships exist for actual pavement structures.

(iv) relationships between curvature and maximum deflection, if properly defined, can be used together with measurements of the deflected shape of the pavements's surface, as recorded by a Deflectograph, to estimate bituminous layer thickness.

These conclusions suggest that a method can be developed for continuously assessing the thickness of bituminous material from the analysis of deflected shape measurements made with a Deflectograph.

Considered in the next section is the way in which such a procedure could be used.

The final section gives details of the areas that require further work to allow the relationship between curvature and maximum deflection to be incorporated into a bituminous layer thickness estimation procedure.
8.4 A PROCEDURE FOR CONTINUOUSLY ASSESSING THE THICKNESS OF BITUMINOUS MATERIAL IN A FLEXIBLE PAVEMENT STRUCTURE

The empirical curvature and maximum deflection relationships have been used in conjunction with deflected shape measurements to estimate the thickness of bituminous material in a pavement structure.

In the main, the estimates of bituminous layer thickness made in this way agreed favourably with the actual thicknesses indicated by the cores.

This validation work, although based upon a small sample of measurements has clearly demonstrated the potential use of properly defined relationships between curvature and maximum deflection, together with deflected shape measurements, for estimating bituminous layer thickness.

Considered in this section is the way in which these relationships can be incorporated into a procedure that could be used on a routine basis to estimate the thickness of bituminous material in a flexible pavement structure.

The procedure for estimating the thickness of the bituminous material, based on the relationships between curvature and maximum deflection, is not seen as an alternative to conventional coring techniques.

The role of the estimation procedure is to reduce the large amount of uncertainty that exists at present when trying to estimate the thickness of the bituminous layer between two cores and indeed if the cores are of different thickness, to identify where the change in thickness occurs.

Essential to the proposed bituminous layer thickness estimation procedure would be the need to extract cores in the conventional manner, at \( \frac{1}{4} \) km or 1 km intervals, as part of the routine coring programme before the Deflectograph survey.

Typical material types and layer thicknesses could be identified from
the extracted cores. This construction information would allow the selection of the appropriate chart defining the theoretical relationships between curvature and maximum deflection from a series of charts applicable to a wide range of pavement structures. These relationships could then be used, together with the deflected shape measurements recorded with the Deflectograph, to estimate the thickness of bituminous material for the lengths of pavement between the positions where cores had previously been extracted.

Investigations with the theoretical model has clearly demonstrated the large influence that temperature associated changes in the stiffness of the bituminous layer can have on the relationship between curvature and maximum deflection.

Temperature variations do occur in practice, posing a real problem to any evaluation method that is based upon measurements of the response of the pavement structure to a given load. A temperature correction procedure is therefore required that will allow the curvature and maximum deflection relationships to be used in conjunction with deflected shape measurements recorded over a range of pavement temperatures. A proposed temperature correction procedure is outlined in the 'further work' section.
8.5 FURTHER WORK

As a result of the investigations undertaken during this project is is felt that there are a number of areas where further work is required to accurately define a procedure that can be used, together with deflected shape measurements recorded with a Deflectograph to estimate bituminous layer thickness.

The areas of specific interest are itemised below:

1. A basic requirement is that the bituminous layer thickness estimation procedure is based upon relationships between curvature and maximum deflection that are derived from the theoretical response of pavement structures more similar to those likely to be surveyed with the Deflectograph.

Reference could be made to the results of previous coring programmes to identify a number of typical structures.

The nucleus of the bituminous layer estimation procedure would be a series of charts defining the relationships between curvature and maximum deflection for a wide range of pavement structures.

2. The present method of deriving equivalent Deflectograph deflected shapes from the absolute response of a two-dimensional model is particularly cumbersome and a more direct approach is required.

It is therefore proposed that the two-dimensional model be replaced by a three-dimensional model that would allow the absolute response to be converted directly into an equivalent Deflectograph response.

The adoption of a three-dimensional model would also allow the behaviour of the materials in the pavement structure and the method of loading to be defined more accurately.

3. The method used to ensure that the response of the theoretical model was similar to that of the actual pavement structure has several weaknesses.
The most important concerns the derived moduli values and the fact that because they were the only unknowns, their magnitudes were in effect influenced by the behavioural assumptions associated with the modelling technique.

Although an attempt was made to make sure that the moduli were realistic, it was in one instance necessary to adopt a high modulus for the bituminous material to accurately model the deflected shape in the vicinity of the maximum deflection.

A more suitable approach to the problem of accurately defining the response of the theoretical model might be to incorporate moduli values derived from tests on each material type.

In-situ moduli can be estimated using the surface wave propagation equipment. The major problems with this method are that it is time consuming and the derived information is only applicable to the pavement structure directly beneath the vibrator.

The repeat load triaxial equipment can be used in the laboratory to estimate the modulus of samples of material.

4. A temperature correction procedure is required that will allow the curvature and maximum deflection relationships to be used in conjunction with deflected shape measurements recorded over a range of pavement temperatures.

The present method employed to temperature correct the deflected shape measurements that were subsequently used to derive the empirical curvature and maximum deflection, is unsuitable for two reasons.

Firstly, it is not strictly correct to use relationships between maximum deflection and temperature to temperature correct ordinate deflections measured 200 mm from the point of maximum deflection.

Secondly, it was only possible to select the appropriate maximum deflection and temperature relationship because the thickness of bituminous material was known in advance for long lengths of pavement.
An alternative method for taking into account the effect of temperature on the response of a pavement structure is outlined below.

The proposed method involves the use of the theoretical model to prepare charts, for a number of thicknesses of bituminous material, showing the relationships between curvature and maximum deflection for a range of temperatures, Figure 8.1.

![Curvature vs Maximum deflection chart](image)

Figure 8.1 Typical chart showing the effect of temperature on the relationship between curvature and maximum deflection.

The initial selection of the appropriate relationships between curvature and maximum deflection would be made on the basis of the thickness of bituminous material, as indicated by the first core, and the initial pavement temperature.

Successive deflected shape measurements, recorded at specific temperatures, would then be used in conjunction with the selected chart to estimate the thickness of bituminous material present.

Movement from one line, defining the curvature and maximum deflection relationship at a particular temperature, to another line without an associated temperature change would be interpreted as a change in the thickness of the bituminous material.

Movement up the chart representing an apparent increase in temperature
would indicate a reduction in the thickness of bituminous material (increase in curvature).

Movement in the opposite direction would likewise indicate an increase in the thickness of bituminous material (decrease in curvature).

The correct thickness of bituminous material would be identified by the chart showing the correct relationship between curvature and maximum deflection appropriate to the temperature at the time of measurement.

5. These investigations have been primarily concerned with the development of relationships between curvature and maximum deflection for pavements in a sound condition, i.e. showing no visual signs of deterioration.

Pavements are not always in a sound condition and there is therefore a need to determine the influence that the material weaknesses associated with cracking and permanent deformation can have on the relationship between curvature and maximum deflection.

Once defined, the bituminous layer thickness estimation procedure could also be used on pavements showing visual signs of deterioration.

6. Problems were shown to exist when recording deflected shape measurements on a time basis. Variations in the position of the ordinate deflections associated with change in the speed of the Deflectograph occurred regularly.

These variations resulted in the definition of different portions of the overall deflected shape which meant that the measurements had to be corrected before they could be analysed.

To overcome this problem it is proposed that a method be developed that allows the position of each ordinate deflection to be specified on a distance basis, i.e. 25 mm from the point of maximum deflection, etc. This would ensure that at all times the ordinate deflections were recorded at the required distances from the point of maximum deflection.
9. THE BACKGROUND AND DEVELOPMENT OF THE UNITED KINGDOM FLEXIBLE PAVEMENT EVALUATION AND STRENGTHENING DESIGN METHOD.

9.1 INTRODUCTION

More than 20 years ago the Road Research Laboratory developed a standard method of deflection measurement and embarked upon a programme of deflection studies aimed at developing relationships between deflection and pavement performance.

The important concept of the deflection history was introduced to relate deflection and performance, defined in terms of the traffic carried and the condition of the pavement's surface, and is covered in detail in this chapter.

Deflection histories were plotted with deflection as the vertical axis and cumulative commercial traffic as the horizontal axis. The condition of the pavement's surface (Sound, Critical or Failed) in the immediate area of the deflection measurement was indicated by the use of an appropriate symbol. A curve was drawn through the points to separate the sound from the critical and failed conditions.

The curve was termed the deflection criterion curve.

![Figure 9.1 The Deflection Criterion Curve (after Lister)](image)

If deflection remained constant with time the deflection criterion curve could be used to estimate the remaining life of a pavement from...
a deflection measurement. Deflection does not remain constant with time, and in general increases with increase in the traffic carried. It was therefore not only necessary to define the position of the deflection criterion curve but also the way deflection changed with the traffic carried, if the remaining life was to be predicted accurately from a deflection measurement.

Most of the field measurements were made on TRRL's full-scale design experiments and the development of the relationships between deflection and performance for the different pavement types are discussed in relation to these full-scale experiments, which for the purpose of this report are divided into three groups:

(i) The early full-scale design experiments constructed from 1950 to 1956. (36)

(ii) The Alconbury Hill experiment which provided a great deal of deflection information. (36)

(iii) The full-scale experiments constructed from 1960 to 1965.

These studies show that deflection can be related to the performance, for the main roadbase types, and that use can be made of such relationships, together with a knowledge of deflection and the traffic carried, to predict the unexpired pavement life.

The development of similar relationships for overlaid pavements has lead to the construction of overlay design charts, capable of being used to define the overlay thicknesses required to extend the life of a road to any given traffic level.

A flexible pavement evaluation and strengthening design procedure has been developed for use in the United Kingdom and was first published by TRRL in 1973 as Laboratory Report LR571. This method was based upon deflection and performance relationships collected over a 15 year period.

The availability of additional information allowed a second more
comprehensive method to be published by TRRL in 1978 as Laboratory Report LR833. The use of this method (LR833) is given in detail in the next chapter, with particular reference made to areas where change has occurred in the format originally used in the first method (LR571). This second version is currently being used by engineers in the United Kingdom to predict the remaining life and, where necessary, to design strengthening measures for a wide range of flexible pavements.

9.2 DERIVATION OF DEFLECTION AND PERFORMANCE RELATIONSHIPS

The existing flexible pavement evaluation and strengthening design procedures are based upon empirical deflection and performance relationships derived from investigations into the behaviour of full-scale pavement design experiments.

This section outlines the concept of the deflection history and gives details of the work involved in the derivation of the deflection and performance relationships.

9.2.1 Deflection History

The deflection history of a pavement relates the deflection to the life of the road (in terms of the total amount of traffic carried) and the condition, as determined principally by the permanent deformation and degree of cracking in the wheel paths.

9.2.2 Deflection Measurements

The first deflection measurements were made before the road was opened to traffic. Thereafter the measurements were made twice yearly in the early life, followed in general by annual measurements in the later stages. The Deflection Beam was used to measure deflections. Observations at 6 - 12 m along the length of road were used on the experimental sections, but a wider spacing was more common on normal in service roads.

9.2.3 Pavement Condition

The condition of the road in the immediate area of the point of the
deflection measurement was defined primarily in terms of the development of deformation in the wheel paths. Cracking, where it occurred, took place relatively late in the life of the pavement.

Pavement condition, in terms of rutting and cracking, was classified into one of three major categories, SOUND, CRITICAL, or FAILED, using the recommendations given in Table 9.1.

<table>
<thead>
<tr>
<th>Classification</th>
<th>Visible Evidence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sound</td>
<td>No cracking. No appreciable rutting.</td>
</tr>
<tr>
<td></td>
<td>No cracking. Rutting less than 10 mm.</td>
</tr>
<tr>
<td>Critical</td>
<td>No cracking. Rutting between 10 mm and 20 mm.</td>
</tr>
<tr>
<td></td>
<td>Cracking confined to single crack in wheel path.</td>
</tr>
<tr>
<td></td>
<td>Rutting less than 20 mm.</td>
</tr>
<tr>
<td>Failed</td>
<td>Cracking extending over the area of the wheel path and/or rutting greater than 20 mm.</td>
</tr>
<tr>
<td></td>
<td>Dangerous to traffic. Disintegration of the road surface.</td>
</tr>
</tbody>
</table>

Table 9.1 Classification of Pavement Conditions (after Lister)

9.2.4 Traffic

The scarcity of information on wheel load spectra dictated that the cumulative total of commercial vehicles (sum in both directions) was used to define traffic. This information was deduced from Ministry of Transport records. The work from the Alconbury Hill experiment included results from automatic weighbridges and traffic was expressed in terms of equivalent 8,200 kg standard axles using 'the fourth power law'.

241
9.2.5 Deflection Criteria Curve

Deflection histories are plotted with the deflection as the vertical axis and cumulative commercial traffic as the horizontal axis. The condition of the pavement in the immediate area of the deflection measurement is indicated by the use of an appropriate symbol. In this way it is possible to draw a curve through the points to separate the sound from the critical and failed conditions.

Such a curve (Figure 9.2) has been termed a deflection criterion curve for the particular type of pavement examined. If it could be assumed that the deflection of a pavement remained constant with time, the deflection criterion curve could be used to estimate the probable life from a deflection measurement. A change in deflection can occur and under these conditions any estimate made in this way can only be approximate. Lister uses Figure 9.3 to illustrate the nature of changes which are possible in deflection studies.

Two sets of measurements are shown at different times in three areas.
Figure 9.3 The inter-relationship between deflection, traffic and road performance. (After Lister) (36)
of a pavement constructed to the same nominal specification. At area (1) the road was already failing when the first measurement was made and more serious failure was apparent at the time of the second measurement; this was reflected in the increased deflection. Area (2) was sound initially but was failing when the second series of tests were made. The decrease in deflection indicates that compaction of the pavement structure under traffic caused the deformation resulting in failure. In area (3) no failure has been observed but the increase in deflection suggests some deterioration within the pavement or its foundations, likely to lead to early failure.

On this evidence the straight line deflection criterion curve can be drawn through the two critical deflection values revealed by the measurements. The assumption of a constant deflection level equal to the first measurement would for area (2) lead to a shorter estimated life than that actually observed, whilst for area (3) the reverse would be the case.

Clearly two or more measurements made at a point after different intervals of time can be used in conjunction with a criterion curve to obtain increased accuracy in estimating life expectancy.

It follows that the deflection criterion curve so far considered is "immediate" in the sense that it is based on deflections obtained at the time critical conditions are developing in the pavement. It follows too that in using this criterion curve to estimate the future life of a pavement from a deflection value, the variation of deflection with life (in terms of the traffic carried) must be taken into account.

9.3 EARLY FULL-SCALE DESIGN EXPERIMENTS

Details of the early full-scale pavement design experiments are given in Table 9.2.
<table>
<thead>
<tr>
<th>Site</th>
<th>Date of Construction</th>
<th>Commercial Traffic in 1959 (V.P.D.)</th>
<th>Types of Base</th>
<th>Types of Surfacing</th>
<th>Soil Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boroughbridge Yorkshire</td>
<td>1950</td>
<td>2840</td>
<td>Crushed stone Tarmacadam</td>
<td>Rolled asphalt Bitumen macadam</td>
<td>Silty sand</td>
</tr>
<tr>
<td>(A1)</td>
<td>1955</td>
<td></td>
<td>Crushed stone Sand cement</td>
<td>Rolled asphalt Bitumen macadam</td>
<td>Sand</td>
</tr>
<tr>
<td>Cambridge Glos. (A38)</td>
<td>1954</td>
<td>2590</td>
<td>Crushed stone Gravel Tarmacadam</td>
<td>Rolled asphalt</td>
<td>Medium &amp; Heavy clay</td>
</tr>
<tr>
<td>Sapcote Junction Leics.</td>
<td>1951</td>
<td>1690</td>
<td>Crushed stone Tarmacadam</td>
<td>Rolled asphalt</td>
<td>Medium clay</td>
</tr>
<tr>
<td>(A46)</td>
<td>1955</td>
<td>3970</td>
<td>Crushed stone</td>
<td>Rolled asphalt Bitumen macadam</td>
<td>Sandy &amp; Heavy clay</td>
</tr>
</tbody>
</table>

Table 9.2 Details of early full-scale pavement design experiments used for deflection studies (after Lister)

These roads provide examples of a variety of types of pavement construction incorporating both asphalt and open-textured bitumen macadam surfacings laid on bases of crushed stone, open-textured macadam and sand cement.

The deflection readings associated with the traffic compaction phase were not determined because the roads were already open to traffic before the test programme began.

The main conclusions are discussed in terms of the base materials.

9.3.1 Granular Bases

Deflection measurements conformed to the expected pattern of behaviour,
greater deflection measurements were recorded on thinner sections, which also deteriorated to a critical condition before the thicker and stiffer ones. A deflection criteria curve was drawn from the results from these sections. The strength of the subgrade was shown to significantly influence the measured deflection and hence performance of the sections with a granular road base.

9.3.2 Tarmacadam Bases

Sections having a tarmacadam base gave smaller deflections than were recorded on unbound bases of equal thickness reflecting the greater stiffness of the tar coated materials.

9.3.3 Sand Cement Bases

Studies indicated that the behaviour of cement bound materials after they had begun to crack could be strongly influenced by the nature of the surfacing.

9.4 THE ALCONBURY HILL EXPERIMENT

This major full-scale road experiment was built during the Spring and Summer of 1957 and was opened to traffic in November of that year. The experiment provided valuable information on pavement design and also enabled the scope of the deflection programme to be widened to include modern road base materials such as rolled asphalt, lean concrete, and 'wet mix' slag. It proved the first opportunity to study deflection histories of pavements from the time of their construction. Because of the wide range of types and thicknesses of base and surfacing materials used, the general relevance of the definitions of pavement condition, originally derived mainly from the behaviour of pavements with crushed stone bases, were studied. The layout of the experiment was such that varying thicknesses of different types of surfacings and base materials were laid upon varying thicknesses of the same sub-base material.

9.4.1 Pavement Strength

9.4.1.1 Temperature correction of Deflection Measurements
The scope of the deflection programme allowed the relationship between deflection and the temperature of the bituminous material to be studied. Results proved that deflection was dependent upon temperature and recommendations were produced enabling deflection measurements to be corrected to an equivalent deflection at 20°C.

9.4.1.2 Factors influencing deflection

Studies of the pattern of early deflection behaviour revealed that initial deflection values were an unreliable indicator of a road's future performance under traffic, because changes in subgrade strength and stiffening of the road materials due to compaction occurred in the road's early life.

9.4.2 Pavement Condition

The pavement condition classifications shown in Table 9.1 were used in developing the deflection histories. Analysis of the data enabled typical records relating rut depth and cumulative number of standard axles carried to be constructed, Figure 9.4. From these records it was apparent that, after the initial period of compaction, the relation between rut depth and cumulative number of standard axles carried was basically linear up to and generally beyond the value of 10 mm taken as defining a critical pavement condition.

A study of rut depth at which appreciable cracking first appeared was also undertaken and results indicated a mean value close to the 10 mm given in Table 9.1 as defining the onset of critical conditions. As appreciable surface cracking would generally signify structural weakening, the initial choice of rut depth of 10 mm to define the onset of critical conditions was thus supported.

9.4.3 Traffic

Dynamic weighbridges were installed at Alconbury Hill and indicated considerable differences in the wheel load spectra of traffic passing in the two directions. These differences were significant and were reflected in the observed performance of the slow traffic lanes in the
Figure 9.4 Typical relations between rut depth and cumulative total of standard axles (after Lister)
two directions, despite uniformity of subgrade conditions and similar numbers of vehicles passing in each direction.

9.4.4 Deflection History

Deflection histories were produced for sections with bituminous rolled asphalt, 'wet mix' slag, lean concrete, sand cement and tarmacadam bases.

Typical observations by Lister\(^{[36]}\) indicated that after the early changes in deflection already considered, subsequent changes on pavements which remained in a sound condition were generally less rapid.

Most sections showed a slow increase with time prior to an unusually severe Winter of 1963. The immediate effect of this Winter was to cause large irreversible increases in the deflection of many of the surviving sections and was, in many cases, associated with the development of critical conditions.

To eliminate the possibility of a discontinuity in road performances and deflection behaviour, in 1963 the critical lives of the individual points in the experimental sections were analysed in terms of mean values of Spring deflections prior to 1963, or to mean values up to the time critical conditions developed, if this was before that date.

The deflection values were known as standard early life deflections and details of the deflection and performance relationships derived for each roadbase type tested are as follows.

9.4.4.1 Rolled Asphalt Bases

Figure 9.5 shows the standard early life deflection values for all points on sections with rolled asphalt bases plotted against their critical lives, in terms of cumulative standard axles. The results indicated a very strongly defined relation between early life deflection and performance of the form:-

\[
\text{LIFE} \propto \frac{1}{(\text{Deflection})^3}
\]

Deflection in mm.
Figure 9.5 Relation between deflection and critical life for sections with rolled asphalt bases at Alconbury Hill. (After Lister) (36)
Sub-base thickness did not influence the position of the criterion curve.

9.4.4.2 Wet Mix Slag Bases

The mean criterion curve determined from the deflection and performance results for the wet slag bases, under 100 mm rolled asphalt surfacing, was similar in form to that derived for sections with asphalt bases. No influence of sub-base thickness or subgrade strength on the criterion curve was detected.

9.4.4.3 Lean Concrete Bases

The deflection and performance plots for 150 mm and 225 mm thick lean concrete bases, under 100 mm rolled asphalt surfacings, were grouped about a line of similar slope to that derived for wet mix slag and asphalt bases, but considerably below it. Sections exhibiting extensively cracked bases performed in a similar manner to those sections with a thin granular base.

9.4.4.4 Soil Cement Bases

The deflection and performance relationships for sections with 100 mm rolled asphalt surfacings indicated, for the thicker bases, a criterion curve identical to that derived for the much stronger concrete bases.

9.4.4.5 Tarmacadam Bases

The deflection and performance relationship for open-textured tarmacadam bases closely followed the line already derived for both asphalt and wet mix slag bases under 100 mm asphalt surfacings.

9.4.5 Application of the Deflection and Performance Data

The relations between early life deflections and critical lives of pavements with 100 mm rolled asphalt surfacings, in combination with the five types of base used, are shown in Figure 9.6. The deflection criterion curves have been shown to be virtually independent of sub-base and base thickness and also of subgrade strength in the range 251
Figure 9.6 Relation between deflection and critical life for different types of base under 100 mm (4 in) rolled asphalt surfacing. (After Lister)\(^{(36)}\)
of CBR values 2.5 to 7 per cent.

The deflection histories obtained indicated that on almost all pavements, including many of those which remained in a sound condition for the duration of the experiment, a slow increase in deflection took place with time and traffic, once the compaction phase was complete.

Figure 9.7 shows the deflection values when critical conditions develop for sections with rolled asphalt bases, together with the mean trend lines of deflection from early life values towards the critical curve.

Use of such a chart allows the remaining life of a pavement to be estimated at any stage before critical conditions develop, from a knowledge of deflection and of the cumulative number of standard axles carried to date.

9.4.6 Conclusions

Several important conclusions were deduced from the Alconbury Hill experiment.

(1) Changes in moisture conditions in the subgrade, following the disturbance of the construction phase, can cause considerable changes in deflection level. Reductions in deflection can result from compaction and/or cementing action of the granular layers. Deflections measured before the road is opened to traffic are unreliable because of the time taken to achieve subgrade moisture equilibrium and for the cementing action to take place. Compaction of the granular layers is normally complete within about a year.

(2) Measured deflections can be corrected to a standard equivalent deflection at a temperature of 20°C using the derived relationships.

(3) Well-defined relationships between critical lives and mean early life deflections were obtained. For lives of greater than about one million standard axles the curves were of the form

\[
\text{LIFE} \propto \frac{1}{(\text{Deflection})^3}
\]

Deflection in mm.
Figure 9.7 The relation between deflection and traffic up to the onset of critical pavement conditions in sections with rolled asphalt bases at Alconbury Hill. (After Lister). (36)
(4) Virtually identical curves were derived for rolled asphalt, tarmacadam and wet mix slag bases. Equal performance is not implied because of the different thicknesses of different bases required to attain a given deflection and therefore life. Similarly sand cement and lean concrete shared a common curve and again equal performance is not implied.

(5) The Spring deflections measured immediately after the abnormally severe Winter of 1963 were generally appreciably greater than those of the proceeding years. The increase was most noticeable in the weaker sections surviving at the time, and whose former deflection levels were rarely re-established and the critical lives probably shortened.

However, the overall deflection performance relationships for the groups of sections were not distorted by the effects of this Winter, due probably to the abnormally high Winter water table present at this site.

It is therefore reasonable to conclude that abnormally severe Winters will also not effect deflection performance relationships for roads with more normal water table conditions.

(6) Prediction of the unexpired life of pavements from deflection measurements, made at any time before critical conditions develop, can be made from charts which take into account the changes in deflection with time and traffic.

9.5 FULL-SCALE PAVEMENT DESIGN EXPERIMENTS - 1960 to 1965

Details of the six full-scale pavement design experiments, constructed by TRRL during the period 1960 to 1965, have been reported and are given in Table 9.3.

Table 9.3 outlines the main variables investigated at each site, together with the subgrade type and strength. Traffic information is also presented, made possible by the inclusion of dynamic weighbridges at these sites to automatically record axle weights. These experimental pavement sections allowed deflection histories to be
<table>
<thead>
<tr>
<th>Year of construction</th>
<th>Location</th>
<th>Main variable (numbered) and construction details</th>
<th>Subgrade type and strength</th>
<th>Initial traffic* (commercial vehicles/day)</th>
<th>Growth rate of commercial traffic (% per annum)</th>
<th>Damaging effect of commercial traffic (standard axles/100 commercial axles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1960</td>
<td>A4091 (8 km south of Tamworth)</td>
<td>1. Type and thickness of roadbase – rolled asphalt, coated macadam, lean concrete – 100-300 mm thick</td>
<td>Fly ash embankment CBR &gt; 50%</td>
<td>800</td>
<td>3</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Surfacing – Rolled asphalt – 25 mm thick (100 mm over lean concrete)</td>
<td>Sub-base – Fly ash</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1962</td>
<td>A38 Salters Hill (16 km north of Gloucester)</td>
<td>1. Type of surfacing – asphalt, various coated macadams – 100 mm thick</td>
<td>Silty clay LL 56% PL 21% CBR 4%</td>
<td>1500</td>
<td>6</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Roadbase – Wet-mix – 200 mm thick</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sub-base – Gravel – 150 mm thick</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1963</td>
<td>A30 Nately Scures (3 km west of Hook), Hampshire</td>
<td>1. Type and thickness of roadbase – wet-mix, lean concrete, dense-coated macadam, rolled asphalt – 80-300 mm thick</td>
<td>Silty clay LL 60% PL 21% CBR 3.5%</td>
<td>1500</td>
<td>5</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Type of basecourse over coated macadam roadbases</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sub-base – Gravel – 150 mm thick</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1963</td>
<td>A40 Wheatley By-pass (12 km east of Oxford)</td>
<td>1. Grading and strength of cemented roadbase materials – 200 mm thick</td>
<td>Silty clay LL 57% PL 20% CBR 3.5%</td>
<td>850-1300†</td>
<td>3</td>
<td>20-35† (eastbound) 25-30† (westbound)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Grading and binder content of bituminous roadbase materials – 200 mm thick</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Surfacing – Asphalt – 100 mm thick</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sub-base – Gravel – 150 mm thick</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1964</td>
<td>A1 Alconbury By-pass, Cambridgeshire</td>
<td>1. Thickness of certain of the cemented and bituminous roadbase materials used in the Wheatley By-pass experiment (see above) in the range 150-250 mm</td>
<td>Silty clay (average) LL 51% (average) PL 20% CBR 5%</td>
<td>1300-2200†</td>
<td>3</td>
<td>25 (northbound) 45-55† (southbound)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Type and thickness of surfacing – rolled asphalt, bitumen macadam – 100-200 mm thick</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sub-base – Gravel – 150 or 300 mm thick</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1965</td>
<td>A1 Conington, Cambridgeshire</td>
<td>1. Type of basecourse under asphalt wearing course (total thickness 100 mm), on wet-mix roadbase 200 mm thick</td>
<td>Silty clay LL 50% PL 20% CBR 4%</td>
<td>2400</td>
<td>3</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Type of bituminous roadbase 150 mm thick under rolled asphalt surfacing 100 mm thick</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Basecourse and – Crushed rock, various gravel roadbase aggregates</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sub-base – Gravel – 150 mm thick</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Traffic on each carriageway on dual-carriageway roads, or in each direction on single carriageway roads.
†Depends on location along site and position of side roads.

Table 9.3 (After Cramm) (16)
built up for a wide range of material types.

Lister and Kennedy have presented details of the deflection programme of studies conducted on the full-scale pavements. Figures were produced to show the strongly defined relationships between early life deflection and the onset of critical conditions (Figure 9.8). These relationships were combined with similar relations between critical deflections and traffic carried (Figure 9.9) to produce a design chart for predicting unexpired pavement life (Figure 9.10). Similar design charts were also developed for pavements with unbound and cement bound roadbases.

Information was given on the development of deflection and performance relationships from a study of overlaid pavements. From this information overlay design charts (Figures 9.11 and 9.12) were constructed defining the overlay thicknesses required to extend the life of a road to any given traffic level.

An evaluation and strengthening design procedure incorporating these design charts, and based upon deflection measurements, was briefly described.

Details of this method are now presented in the next chapter.
Figure 9.8 Relation between early life deflection and critical life for pavements with bitumen and tar bound bases. (after Lister and Kennedy)\(^{(38)}\)

![Figure 9.8](image)

Figure 9.9 Relation between deflection and critical life for pavements with bituminous and tar bound bases. (after Lister and Kennedy)\(^{(38)}\)

![Figure 9.9](image)
Figure 9.10 Relation between deflection and critical life for pavements with bituminous and tar bound bases. (after Lister and Kennedy)\(^{68}\)

Figure 9.11 Thickness of overlay required to achieve a given extension of life for existing pavements with granular and bituminous bases.

(after Lister and Kennedy)\(^{38}\)

259
Figure 9.12 Thickness of overlay required to achieve a given extension of life for existing pavements with lean concrete bases.
(after Lister and Kennedy)\(^{(38)}\)
THE USE OF THE FLEXIBLE PAVEMENT EVALUATION AND STRENGTHENING DESIGN METHOD

Two flexible pavement evaluation and strengthening design procedures, for use in the United Kingdom, have been published by TRRL.

The first appeared in 1973 as Laboratory Report LR571 and was based upon deflection and performance information collected over a 15 year period.

The second method was published in 1978 as Laboratory Report LR833 and is considered to be more comprehensive than that given in LR571 because of the additional data available for analysis prior to publication.

Essentially these evaluation and design procedures consist of three main steps.

10.1 STEP ONE

Standardisation of the deflection measurements to take into account the possible effects of equipment type and temperature on the measured value. This is achieved by making use of the relationships between

(i) Deflection Beam deflections and Deflectograph deflections.
(ii) Deflection and pavement temperature.

10.2 STEP TWO

Prediction of the remaining life by using the relationship between deflection and traffic, together with the standardised deflection measurement and an estimate of the traffic carried.

10.3 STEP THREE

If the predicted remaining life is less than that required, a suitable overlay thickness to increase the life of the pavement can be designed. This is achieved by using the overlay design charts, together with
the standardised deflection measurement and the required traffic level.

The main difference between the second evaluation and strengthening design method (LR833) and the previous one (LR571) is the way in which the empirical relationships, referred to previously, are presented. Either the number and/or the form of the charts used to express these relationships has been altered as a direct result of the information gained from the analysis of the additional deflection and performance data.

A detailed breakdown of the use of the second more comprehensive evaluation and strengthening design method, in terms of the three main steps, now follows. Particular reference is made to areas where the format has changed from that originally presented in the first method.

10.4 STEP ONE: STANDARDISING THE STRUCTURAL STRENGTH MEASUREMENTS

10.4.1 Correlation of Deflection Measurements

The most important input into any strengthening design method is an assessment of the existing condition or strength of a pavement, characterised in this particular case by the measurement of the maximum transient deflection of the pavement's surface caused by a load applied to that surface.

The two pieces of equipment suggested for the measurement of maximum deflection were the Deflectograph and the Deflection Beam and details of the equipment and the standard operating procedures are given in LR834 and LR835 respectively.

Neither the Deflection Beam nor the Deflectograph measure the absolute transient deflection because of the way in which the beam assembly supports are influenced by the deflection of the pavement under the action of the front and rear wheels of the truck. To further complicate matters the proportion of absolute deflection recorded is not the same with each piece of equipment; Deflectograph readings are.
lower than Deflection Beam readings. The two main reasons for this are:

(i) the tyre spacing and area of contact of the two types of wheel assembly are dissimilar.

(ii) the beam support points are not influenced in the same manner during the measured cycle.

The dissimilarity between the two measured values of deflection meant that deflection measured with the Deflectograph could not be used directly in connection with the deflection and performance relationships because these were derived using Deflection Beam deflection measurements. Therefore deflection measurements made by any other means, such as the Deflectograph, must be correlated with the Deflection Beam measurements if the criteria outlined in the method were to be adopted.

The correlation between deflections measured by each of the two pieces of equipment was achieved by comparing the deflection values obtained at identical positions along a particular length of road. Experiments on a wide range of pavements with unbound, cemented and bituminous bases showed that the correlation was slightly dependent upon pavement temperature, but could be sufficiently closely represented by the relationship given in Figure 10.1.

This chart replaces the two deflection correlation charts presented in LR571 where distinction was made between granular and cement bound roadbase and bituminous roadbases.

10.4.2 Temperature Correction of Deflection Measurements

Information was given on the recommended times of the year during which deflection surveys should be undertaken. The preferred periods of Spring and Autumn were suggested, mainly because the pavement
Figure 10.1 Correlation between Deflection Beam and Deflectograph (after Kennedy and Lister)\(^{(35)}\)
temperature was more likely to be within the recommended range of 10° C to 30° C. Deflection measurements made outside this range were to be avoided because of the large temperature correction likely to be necessary and the correspondingly large inherent errors associated with such corrections.

The relationships between deflection and temperature, derived from a study of pavements with a wide range of bases and surfacings, were presented as six separate charts enabling deflections measured at any temperature, within the recommended range, to be corrected to 20° C. The choice of the relevant chart is dependent upon two factors; the overall thickness of bituminous material and the thickness of dense bituminous material.

Figure 10.2 shows the temperature correction chart for pavements with 75 - 195 mm of bituminous material of which at least 75 mm is dense bituminous material. These six temperature correction charts replace the three charts presented in LR571(65), allowing a more accurate estimation of the effect of temperature to be made by taking into account the proportion of dense bituminous material present.

The deflection and temperature relationships associated with pavements exhibiting inter-connected and well developed cracks are different to those on uncracked pavements. Cracked pavements are less susceptible to the effects of temperature and to account for this Table 10.1 was given to help select the appropriate temperature correction chart for severely cracked pavements.

The general effect of Table 10.1 was to recommend the use of the temperature correction chart applicable to a range of bituminous layer thicknesses immediately below the range suggested by the bound layer thickness present, i.e. the effect of temperature reduced for cracked pavements.

The method for taking into account the different deflection and temperature relationships for cracked pavements has been changed. Table 10.1 replaces the broken lines shown on each temperature correction chart in LR571, Figure 10.3.
Figure 10.2 Relation between deflection and temperature for pavements with 75 - 195 mm of bituminous material of which at least 75 mm is dense bituminous material. (After Kennedy and Lister) (35)
<table>
<thead>
<tr>
<th>Total thickness of bituminous material in the pavement mm</th>
<th>Thickness of dense bituminous material in the pavement mm</th>
<th>Use temperature correction chart for</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;324</td>
<td>—</td>
<td>275–324 (8)^A</td>
</tr>
<tr>
<td>275–324</td>
<td>—</td>
<td>195–274 (7)</td>
</tr>
<tr>
<td>195–274</td>
<td>—</td>
<td>Either</td>
</tr>
<tr>
<td></td>
<td></td>
<td>75–195 (6)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>137–195 (5) if ≥75</td>
</tr>
<tr>
<td>75–195</td>
<td>≥75</td>
<td></td>
</tr>
<tr>
<td>137–195</td>
<td>&lt;75</td>
<td>135 (4)^B</td>
</tr>
<tr>
<td>&lt;137</td>
<td>&lt;75</td>
<td>135 (4)^C</td>
</tr>
</tbody>
</table>

A Figures in brackets indicate Figure number in this Report
B The solid lines on Figure 4 should be used
C The dashed lines on Figure 4 should be used

Table 10.1 Selection of appropriate temperature correction chart for severely cracked pavements (after Kennedy and Lister) (35)
Figure 10.3 Relation between deflection and temperature for pavements with not more than 200 mm of bituminous material of which at least 100 mm is rolled asphalt. (After Norman, et al) (65)
10.4.3 Assessing the Visual Condition of the Pavement's Surface

It was suggested in this method that the visual condition of the pavement's surface be determined close to the points at which the deflection measurements were made. Standard procedures were recommended.

Rut depths were to be measured by sliding a graduated wedge under a 2 m straight edge laid transversely across the wheelpath and the measurements recorded together with the extent of any cracking.

The condition of the pavement's surface could then be classified in accordance with the recommendations given in Table 10.2.

The visual condition classifications of SOUND, CRITICAL and FAILED and the corresponding visual evidence criteria presented in this method have not been altered from that given in LR571, Table 10.3. However reference to Tables 10.2 and 10.3 will show that a coding system has been introduced allowing a further sub-division of the three main classifications. The sub-division of the FAILED classification was necessary to allow adequate definition of any cracking present.

10.5 STEP TWO: ESTIMATING THE REMAINING LIFE OF A FLEXIBLE PAVEMENT

Information from the deflection histories was used to construct charts showing the relationships between deflection and traffic for the main roadbase types.

The number of deflection and performance charts has been increased from three in the previous method (LR571) to four expressing the relationships for pavements with:

(i) granular road bases whose aggregates exhibit a natural cementing action;

(ii) non-cementing granular road bases;

(iii) bituminous road bases;

(iv) cement bound road bases.
<table>
<thead>
<tr>
<th>CLASSIFICATION</th>
<th>CODE</th>
<th>VISUAL EVIDENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>SOUND</td>
<td>1</td>
<td>No Cracking Rutting under a 2 m straight-edge less than 5 mm</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>No Cracking Rutting from 5 mm to 9 mm</td>
</tr>
<tr>
<td>CRITICAL</td>
<td>3</td>
<td>No Cracking Rutting from 10 mm to 19 mm</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Cracking confined to a single crack or extending over less than half of the width of the wheel path. Rutting 19 mm or less</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Interconnected multiple cracking extending over the greater part of the width of the wheel path. Rutting 19 mm or less</td>
</tr>
<tr>
<td>FAILED</td>
<td>6</td>
<td>No Cracking Rutting 20 mm or greater</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>Cracking confined to single crack or extending over less than half of the width of the wheel path. Rutting 20 mm or greater</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>Interconnected multiple cracking extending over the greater part of the width of the wheel path. Rutting 20 mm or greater</td>
</tr>
</tbody>
</table>

Table 10.2 Classification of the condition of the road surface (after Kennedy and Lister) (35)

<table>
<thead>
<tr>
<th>CLASSIFICATION</th>
<th>VISIBLE EVIDENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>SOUND</td>
<td>No Cracking Rutting under 2 m straight edge less than 10 mm</td>
</tr>
</tbody>
</table>
| CRITICAL       | (a) No Cracking Rutting between 10 and 20 mm  
(b) Cracking confined to a single crack in the wheel tracks, with rutting less than 20 mm |
| FAILED         | Cracking extending over the area of the wheel-track and/or rutting greater than 20 mm |

Table 10.3 Classification of pavement condition (after Norman et al) (65)
Figure 10.4 shows the relation between standard deflection and life for pavements with non-cementing granular road-bases.

Analysis of the data available for pavements with granular road-bases indicated different deflection and performance relationships for granular materials which exhibit a natural cementing action. An additional chart was included in the LR833 to cover these granular road-bases whose strength increases with time due to the cementing action of the fines in the aggregate.

The method suggests that if there was any doubt as to cementing action it should be assumed that such action takes place because more conservative results would be obtained.

Further guidance on the selection of the appropriate chart was given because individual road pavements can contain road-base and sub-base materials that belong to several or all of the categories which define the four deflection and performance charts.

The method chosen to present the deflection and performance relationships in LR833 was different to that presented in LR571: deflection was plotted to a normal scale and traffic to a logarithmic scale in LR833, whereas both deflection and traffic were plotted to a logarithmic scale in LR571. This change of presentation was in direct response to the requests received from engineers, already using the deflection and performance charts presented in the previous method, for a simpler form of presentation.

The replacement of the single deflection criterion curve shown on the charts in LR571 by four curves on the charts in LR833, representing different probabilities of achieving the desired life was made possible by the extra data available for analysis, allowing the definition of their relative positions.

Use of the lower deflection level corresponding to the 0.9 probability criteria curve means that the engineer is accepting a 10% possibility
Fig. 10.4 Relation between standard deflection and life for pavements with non-cementing granular road bases (after Kennedy and Lister)\(^{(35)}\)
that the pavement will reach critical conditions before this deflection level is reached. Use of the higher deflection level corresponding to the 0.25 probability criteria curve means that there is a 75% possibility that the pavement will reach critical conditions before this deflection level is reached.

The majority of the deflection and performance data used to derive these relationships was collected on medium and heavily trafficked roads having carried between 1 and 20 million cumulative standard axles. The lack of relevant data for the more lightly (less than 1 million cumulative standard axles) and more heavily (greater than 20 million standard axles) trafficked roads meant that the deflection criteria curves had to be extrapolated to cover these areas of the charts.

10.6 STEP THREE: DESIGNING AN OVERLAY TO STRENGTHEN A FLEXIBLE PAVEMENT

The overlay design procedure is presented as a series of charts based upon information collected from the overlaid sections of the full-scale pavement design experiments. It is a far more comprehensive overlay design procedure than that given in the first evaluation and strengthening design procedure (LR571) again mainly as a consequence of the additional amount of data available from the overlaid pavement sections.

The overlay design charts, Figure 10.5, can be used to specify the thickness of hot rolled asphalt overlay required to strengthen a pavement of given deflection in order to achieve a desired extension of life, expressed in standard axles. In defining the thickness of overlay required, the charts take into account any remaining life in the original pavement.

Different probabilities can be assigned to the extension of an existing pavement's life by overlaying. Two overlay design charts have been produced for each of the main road-base types corresponding to 0.50 and 0.90 probabilities of achieving the desired extension of life.

The design charts are for overlays using rolled asphalt materials only.
Overlay design chart for pavements with non-cementing granular road bases (0.50 probability). (after Kennedy and Lister) (35)
Information on the performance of dense coated macadam overlays and open textured macadam overlays has allowed thickness factors to be estimated, Table 10.4. These thickness factors can be used to convert a thickness of rolled asphalt overlay, determined from the design charts, into an equivalent thickness of the relevant type of macadam to give the same extension of life.

**TABLE 10.4**

Overlay thicknesses for coated macadams in relation to the requirements for rolled asphalt (after Kennedy and Lister) (35)

<table>
<thead>
<tr>
<th>Material</th>
<th>Thickness factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rolled asphalt</td>
<td>1</td>
</tr>
<tr>
<td>Dense coated macadam containing 100 pen or B54 binder</td>
<td>1</td>
</tr>
<tr>
<td>Dense coated macadam containing 200 pen or B50 binder</td>
<td>1.3</td>
</tr>
<tr>
<td>Open-textured macadam</td>
<td>2</td>
</tr>
</tbody>
</table>

10.7 SUMMARY OF THE INFORMATION REQUIRED TO USE THE FLEXIBLE PAVEMENT EVALUATION AND STRENGTHENING DESIGN METHOD

10.7.1 Measurements

(a) The deflection of the existing pavement measured either by the Deflection Beam or the Deflectograph.

(b) The temperature of the pavement at a depth of 40 mm below road surface.

10.7.2 Traffic

(a) The cumulative number of standard axles carried by the existing road since construction, or since its last major structural maintenance.
(b) An estimate of the future traffic expected during the desired life or the overlay.

10.7.3 The Pavement

Information to identify the type and condition of the pavement for the purpose of selecting the appropriate charts for

(a) correcting measured deflections to standard values at 20° C;

(b) estimating the remaining life of the pavement;

(c) designing the overlay

10.7.4 The Subgrade

(a) A broad classification of subgrade strength to determine whether it lies in the range of CBR values between 2.5 and 15 per cent.

(b) Information regarding cut and fill and other factors relevant to the drainage conditions under and near the road.

10.7.4 The Overlay

The type of material to be used.
11 VALIDATION OF THE FLEXIBLE PAVEMENT EVALUATION AND STRENGTHENING
DESIGN METHOD FOR USE ON LIGHTLY TRAFFICKED ROADS

11.1 INTRODUCTION

The existing flexible pavement evaluation and strengthening design method, developed by the Transport and Road Research Laboratory, \(^{(35)}\) is based upon deflection and performance relationships built up over a number of years from investigations into the behaviour of full-scale pavement design experiments. These experimental sections were of various construction thicknesses appropriate to heavy, medium and lightly trafficked roads and were generally incorporated into realignments of existing heavily trafficked roads. Full deflection histories were established for the deeper construction thicknesses associated with heavily and medium trafficked roads.

However, little deflection and performance data was collected from the thinner construction thicknesses, associated with the more lightly trafficked roads, because these tended to fail rapidly due to the accelerated rate of traffic loading.

Much of the data that was collected from these sections of thinner construction was omitted from the deflection and performance charts given in LR833 because of the possibility that the rapid rate of deterioration of these sections made it difficult to relate specific deflection levels with pavement condition. It was thought unlikely that the results would represent a true reflection of the performance of these construction thicknesses under normal traffic loading. This lack of data on the behaviour of lightly trafficked roads meant that the deflection and performance relationships presented in the charts in LR833\(^{(35)}\) had to be extrapolated to accommodate these roads.

In an effort to determine more information about the behaviour of lightly trafficked roads, a contract was awarded to the Department of Civil Engineering at Plymouth Polytechnic by the Transport and Road Research Laboratory to investigate the behaviour of these roads over a three-year period.
11.1.1 The Aim of the Work and the Methodology adopted

One of the aims of this work was to develop relationships between deflection and performance for lightly trafficked roads and validate the position of the critical condition curves presented in the deflection and life charts in LR833.

The time scale of the project meant that it was impossible to develop full deflection histories for each of the test sites. Instead a new methodology was derived to express the deflection and performance relationships for each particular site as a slice in time from their complete deflection histories.

This approach involved defining

(i) the range of deflections corresponding to pavements visually classified as being in a SOUND, CRITICAL and FAILED condition.

(ii) the slice in time in the total history of the pavement in terms of the cumulative number of standard axles carried by each site since construction or last major strengthening or reconstruction.

Providing that a range of traffic levels existed for the lightly trafficked test sites, an estimate of the position of the critical condition curve could be made from the relative positions of the critical deflection range for each slice in time.

11.1.2 Selecting suitable sites and determining road-base types and bituminous layer thicknesses

To determine empirical relationships between deflection and performance for lightly trafficked roads it was first necessary to select lengths of existing road as possible suitable test sites and, for each site, determine the road-base types and the thickness of the bituminous layer.
11.1.2.1 Selecting Sites

The project did not have the resources, nor the time, to construct and monitor the behaviour of full-scale experimental pavement sections and, for these reasons, studies of lightly trafficked roads were undertaken on lengths of existing road which had been in service for many years.

For the purpose of selecting suitable lengths of road, in Devon and Somerset, a lightly trafficked road was defined as not having carried more than one million cumulative standard axles since construction or last major reconstruction or strengthening.

11.1.2.2 Establishing Road-base Type

The work previously undertaken on the full-scale experimental pavement sections had shown that different deflection and performance relationships exist for the main road-base types. It was therefore necessary to determine the type of road-base present at each test site.

Investigations proved that few, if any, records of pavement thicknesses exist for the lightly trafficked test sections because, in general, they had not been designed but had been adapted to suit changing needs by a succession of strengthening measures. An alternative approach was therefore required for determining the type of road-base present at each site. The method adopted involved the extraction of 150 mm diameter cores at about 500 m intervals and the examining of both hole and core to establish the relevant information.

11.1.2.3 Establishing Bituminous Layer Thickness

A knowledge of the thickness of the bituminous layer was also required in order that the appropriate relationship between deflection and temperature could be selected for standardising the deflection measurements to equivalent values at 20°C. The thickness of the bituminous material was measured directly from the extracted cores.

11.1.3 Relating deflection to the condition of the pavement's surface

The slice in time approach requires that the deflection levels
corresponding to each of the three pavement condition classifications, be determined. It was therefore necessary to measure both the deflection and the visual condition of the pavement's surface.

11.1.3.1 Pavement Strength

Deflection surveys were undertaken on each site every Spring and Autumn for the duration of the project. These periods were preferred because experience from the full-scale design experiments indicated that during these times the yearly variation in subgrade moisture conditions and hence subgrade stiffness was small and relatively steady equilibrium deflection values could be expected to be obtained.

The measurement of deflection was made mainly with a Deflectograph and supplementary measurements were made with a Deflection Beam. Use of the Deflectograph has highlighted a number of potential operating problems on lightly trafficked roads and these are discussed in Section 7.5.

To date about 200,000 individual deflection measurements have been recorded along a total of 400 kilometres of lightly trafficked roads.

In general, the procedures recommended for the use of these two pieces of equipment in LR835 were adopted.

It was necessary to standardise the raw Deflectograph deflection measurements into equivalent Deflection Beam deflections and then temperature correct these values to equivalent deflections at 20°C to ensure that all deflection measurements were in a similar format to that used to define the deflection and performance relationships presented in the charts in LR833.

Standardisation was achieved using either

(i) the DEFLEC suite of computer programs;

(ii) a computer program specifically written for use with a PET micro-computer;

(iii) the charts in LR833.
11.1.3.2 Visual Condition

The visual condition of the pavement's surface was also assessed at the time of each Deflectograph survey in terms of the permanent deformation and cracking in the wheelpaths. The total length of each test site was then classified as being in a SOUND, CRITICAL or FAILED condition using the recommendations and code system given in TABLE 2 of LR833.\(^{35}\)

11.1.3.3 Matching Deflection and Condition

The deflection measurements were used in conjunction with the visual condition data to determine the deflection ranges for each of the three pavement condition classifications. Obtaining the required accuracy of match between deflection and condition proved to be an important part and controlling aspect of these investigations.

11.1.4 Estimating the Traffic Carried by each Test Site

The slice in time is defined by the traffic carried by each test site and is expressed in terms of the cumulative number of standard axles.

To define a volume of traffic in this way information is required on the present yearly flow of commercial vehicles, estimated growth rates, and the number of standard axles per commercial vehicle. Estimates of yearly flow were based upon data collected from traffic counts and the number of standard axles per commercial vehicle was calculated from axle weights measured with portable weighbridge equipment. Use of the portable weighbridge has demonstrated large differences between the calculated standard axle factor and the average national figures.

A method of estimating damage factors using the portable weighbridge equipment is proposed, taking into account the number and types of vehicle in each classification in the total commercial traffic flow.

11.1.5 Developing relationships between deflection and performance and validating the position of the critical condition curves for pavements with granular non-cementing and bituminous road-bases.

Using the deflection and pavement condition measurements accumulated
during this project a basic relationship has been developed between deflection and the condition of the pavement's surface for the lightly trafficked roads with granular non-cementing and bituminous road-bases.

The slice in time approach was used to compare these relationships with those published by TRRL with particular emphasis on the validation of the position of the critical condition curves.

The relationships derived from the experimental data suggest that the behaviour of lightly trafficked roads with granular non-cementing and bituminous road-bases is similar to that presented in the charts in LR833. (35)

This work has added confidence to the use of the LR833 method for evaluating the remaining life and designing strengthening measures for lightly trafficked roads.

11.2 THE 'SLICE IN TIME' METHODOLOGY

In 1978 a rational maintenance design method for flexible pavements was published, based on the measurement of the maximum transient deflection, to define structural condition of the pavement. (35) This maintenance design method made use of relationships determined from full-scale pavement design experiments located on sections of existing medium and heavily trafficked roads.

Little or no data existed prior to the start of this project on the behaviour of lightly trafficked roads.

The study of the behaviour of flexible pavements has in the past been concerned with the construction of full-scale design experiments and the monitoring of their behaviour over a substantial period of time, to produce a full deflection history. A deflection history of a pavement relates the deflection to the life of the road (in terms of the cumulative amount of traffic carried) and the condition, as determined principally by the permanent deformation and extent of cracking in the wheelpaths. By combining individual deflection histories for a number of roads it was possible to define the position
of the deflection trend lines and the critical condition curve (Figure 11.1).

![Deflection trend lines](image)

Deflection trend lines

Critical condition curve

Cumulative Traffic

Figure 11.1  Deflection History

Twenty years is generally regarded as the design life of a new road.

This project provided for only a three year study of lightly trafficked roads, thus requiring a different approach.

It was found necessary to develop a new methodology for the study of the behaviour of lightly trafficked roads. The method adopted was to consider the deflection and performance relationship, for each particular road, as a slice in time from their complete deflection histories. (Figure 11.2).

The slice in time was identified as the cumulative number of standard axles carried by each test site since construction or last major strengthening or reconstruction. The deflection levels associated with each of the three pavement condition classifications (Table 11.6) were determined for each slice in time by comparing the deflection measurements with the visual condition survey data.

The combination of this information provides a vertical ordinate on the deflection performance chart identifying the deflection level corresponding to critical conditions.
A range of traffic levels existed for the lightly trafficked test sites and by plotting on the deflection performance chart a vertical ordinate for each, it was possible to identify the position of the critical condition curve.

11.3 SITE SELECTION

11.3.1 Introduction

Investigations into the performance of lightly trafficked roads were conducted on lengths of existing road which had been in service for many years.

This approach was adopted because the project did not have the resources nor the time to construct full-scale experimental pavement sections.

Selection of suitable sites was based primarily on commercial traffic flows; a lightly trafficked road was defined as one having a traffic flow that would not produce more than about one million cumulative standard axles in a 10 to 15 year maintenance free period.

Having selected suitable lengths of road on a traffic basis, a
secondary selection was made taking the following factors into consideration:

(i) The term 'lightly trafficked' could apply to roads in any of the three main road classifications, A class, B class and unclassified, and it was therefore necessary to include a proportion of each as test lengths. It was also felt that this criterion would ensure the selection of roads covering a wide range of construction thicknesses. It has to be remembered that lightly trafficked did not necessarily mean the pavement construction was thin.

(ii) The Devon sites were to be within easy reach of Plymouth to reduce the amount of travelling time and so maximise the time spent on site.

(iii) To select the lengths of road on as many different types of subgrade as possible, to ensure that any derived relationships were applicable to the widest range of subgrade strengths. Different soil types were also a requirement of an associated investigation into the effect of subgrade moisture on deflection measurements.

11.3.2 Initial Site Selection

A preliminary list of roads that had carried less than one million cumulative standard axles was drawn up by both Somerset and Devon County Councils. All of the roads on these lists were investigated in detail; many were discounted because they proved to be either too lightly trafficked or an equivalent site was located closer to Plymouth.

Table 11.1 details the sites initially selected from the preliminary lists and all comply with the traffic criterion and factors previously stated.
Table 11.1

The A38 is not a lightly trafficked road and was selected because, as one of the main trunk roads into the South West, it carries a large number of commercial vehicles per day thus putting it into the same classification as many of the roads from which the data expressed in the charts in LR833 was gathered. The results from the A38 would be plotted on the existing charts showing the deflection and performance relationships as a check on the compatibility of the deflection levels recorded during this project. This was important when trying to use such data to define the position of the critical curves at the lower traffic levels.

11.3.3 Soil Types and Strengths

The subgrade information used to select these sites was determined
from a combination of geology maps of the local area and laboratory analysis of hand excavated material. Table 11.2 gives general soil types and strengths for a selection of sites.

<table>
<thead>
<tr>
<th>TEST SITE</th>
<th>SUBGRADE TYPES</th>
<th>C.B.R.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A379</strong> Modbury to Aveton Gifford</td>
<td>Decomposed slates and shales with sandy clay layers. L.L. 30% P.I. 10%</td>
<td>2-3</td>
</tr>
<tr>
<td></td>
<td>Silty clays. L.L. 40% P.I. 20%</td>
<td>2-3</td>
</tr>
<tr>
<td><strong>A381</strong> Totnes to Halwell</td>
<td>Slates and shales with sandy silt layers. Very low fines. Non Plastic.</td>
<td>10-15</td>
</tr>
<tr>
<td><strong>B3210</strong> Ermington</td>
<td>Decomposed slates with silty sand and clay layers. L.L. 48% P.I. 8%</td>
<td>&gt;15</td>
</tr>
<tr>
<td></td>
<td>Sandy clay. L.L. 48% P.I. 27%</td>
<td>5</td>
</tr>
<tr>
<td><strong>B3218</strong> Okehampton</td>
<td>Heavy clays. L.L. 67% P.I. 35%</td>
<td>4</td>
</tr>
<tr>
<td><strong>Unclassified</strong> Cornwood to Cadover Bridge</td>
<td>Decomposed / kaolionised granite and sandy gravel with organic silts and clays. L.L. 39% P.I. 15%</td>
<td>&gt;20</td>
</tr>
<tr>
<td></td>
<td>Well graded sandy river gravels with organic silts.</td>
<td>10-15</td>
</tr>
<tr>
<td><strong>Unclassified</strong> Lee Moor to Plympton</td>
<td>Generally as for Cornwood, but with some areas of slates and shales with organic silts. L.L. 50% P.I. 15%</td>
<td>8</td>
</tr>
<tr>
<td><strong>Unclassified</strong> Meare to Ashcott Corner</td>
<td>Peat. L.L. 400%</td>
<td>&lt;2</td>
</tr>
</tbody>
</table>

Table 11.2 General Subgrade Types on Test Sections.
11.3.4 Site Re-Selection

The sites shown in Table 11.1 represent a total length of about 100 km; a length surveyed with the Deflectograph in Autumn 1978 and Spring 1979.

The total length of lightly trafficked road surveyed was reduced at the end of 1979 because analysis of the data showed that the performance of several sections along a particular road was similar. No benefit would be gained from such duplication of results.

The sites shown in Table 11.3 were retained because of their particular interest or because they were within easy reach of Plymouth.

<table>
<thead>
<tr>
<th>REVISED SITES SURVEYED IN DEVON AND SOMERSET 1980 &amp; 1981</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DEVON</strong></td>
</tr>
<tr>
<td>SITE 1 A381 - Totnes to Halwell</td>
</tr>
<tr>
<td>SITE 2 A379 - Modbury to Aveton Gifford</td>
</tr>
<tr>
<td>SITE 3 B3210 - A379 Junction to Ugborough</td>
</tr>
<tr>
<td>SITE 4 B3218 - Okehampton to Halwell</td>
</tr>
<tr>
<td>SITE 5 Unclassified - Cornwood to Cadover Bridge</td>
</tr>
<tr>
<td>SITE 6 Unclassified - Lee Moor to Plympton</td>
</tr>
<tr>
<td>SITE 7 A38 - South Brent, Ivybridge south-bound carriageway, Lee Mill north-bound carriageway.</td>
</tr>
<tr>
<td><strong>SOMERSET</strong></td>
</tr>
<tr>
<td>SITE 8 Unclassified - Meare to Ashcott Corner</td>
</tr>
</tbody>
</table>

Table 11.3
11.3.5 Sites considered in detail

The six lightly trafficked test sites in Devon comprise of two A class roads, two B class roads and two unclassified roads. For analysis purposes these six test sites were divided into two groups containing a test site from each of the three road classifications; the idea being that the two research assistants working with the data were responsible for the analysis of half of the information collected. The results from the two analyses would be combined to show the deflection and performance relationships derived for all of the lightly trafficked test sites.

The test sites considered in detail in this report are shown in Table 11.4.

<table>
<thead>
<tr>
<th>SITE</th>
<th>ROAD</th>
<th>SECTION</th>
<th>LOCATION</th>
<th>ROAD BASE TYPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site 1</td>
<td>A381</td>
<td>Section 1A</td>
<td>Totnes Hill Ch. 0.00 - 1.25</td>
<td>BITS</td>
</tr>
<tr>
<td>Site 1</td>
<td>A381</td>
<td>Section 1D</td>
<td>Halwell to River Wash Bridge Ch. 9.25 - 7.33</td>
<td>GNCA</td>
</tr>
<tr>
<td>Site 4</td>
<td>B3218</td>
<td>Section 4A</td>
<td>Ashbury Junction to 'x' Roads Ch. 0.00 - 2.60</td>
<td>GNCA</td>
</tr>
<tr>
<td>Site 5</td>
<td>Unclassified</td>
<td>Section 5A</td>
<td>Cornwood to the Quarry Ch. 0.00 - 1.615</td>
<td>GNCA</td>
</tr>
<tr>
<td>Site 5</td>
<td>Unclassified</td>
<td>Section 5B</td>
<td>Tolchmoor Gate to Cadover Bridge Ch. 3.00 - 7.00</td>
<td>GNCA</td>
</tr>
</tbody>
</table>

Table 11.4

11.4 PAVEMENT CONSTRUCTION

11.4.1 Introduction

Investigations by TRRL into the behaviour of full-scale experimental pavement sections\(^{(35)}\) has shown that different deflection and performance relationships exist for the main types of road-base and that different deflection and temperature relationships exist for
various thicknesses of bituminous material.

It was therefore necessary to identify:-

(i) the road-base type for each site to ensure that the relationships between deflection and performance, developed as a result of this work, would be related to the existing data for the correct road-base type.

(ii) the variation in the thickness of the bituminous layer to allow the appropriate relationship between deflection and temperature to be used to standardise the deflection measurements.

11.4.2 Establishing Pavement Construction Data

The majority of the test sites are lengths of road which were not designed to carry a specific amount of traffic. They are mostly old roads which have, over the years, been improved by a series of strengthening measures to increase their load carrying capacity to somewhere near that required by modern commercial vehicles. Neither drawings showing pavement construction thickness, nor design specifications, were available to give details of material types and thicknesses and consequently an alternative means of obtaining this information was required.

The method adopted was the extraction of 150 mm diameter cores at predetermined intervals along the length of each site using initially an Atlas Copco Minuteman Drilling Rig and a 300 mm long core barrel.

Adequate lubrication to the cutting edge was achieved using a pump drawing water either from a nearby stream or from a water bowser. On average four or five cores were removed from each test length at about 500 m intervals, depending upon the depth of penetration and availability of water. It was found that a great deal of time was spent both positioning and removing the road signs necessary to warn approaching traffic of the obstruction caused by the vehicles and drilling equipment.
On removal from the road the cores were measured and clearly labelled identifying the site and the chainage from which they came. The hole was also inspected to identify the type of road-base material present and to especially check for any evidence of cement-bound material. This was a more reliable check because it was not always possible to identify cement-bound material from the removed core because it can be reduced to give the appearance of a granular material by the action of the core barrel. The holes were backfilled with a bitumen macadam containing a retarding agent, to slow down the setting process until the material was compacted. Subsequent removal of cores was undertaken with a drilling rig specifically designed by the Department of Civil Engineering and built by the Technicians within the Department.

For the particular requirement of extracting bitumen cores, this drilling rig had two distinct advantages over the Minuteman.

Firstly, it required less water to lubricate the cutting edge, resulting in more cores being extracted with a given volume of water providing an overall saving in time because locating a water source, once on site, and filling the storage tanks were time-consuming operations. Secondly, the drilling rig and ancillary equipment were considerably lighter requiring less personnel to undertake the coring programme.

11.4.3 Defining Road-base Type

There was a need to identify the types of road-base material present at each site because of the different deflection and performance relationships shown to exist for the main types of road-base. The criteria stated in LR83 and expressed as Figure 11.3 in this report was used for this purpose.

11.4.4 Defining the thickness of Bituminous Layer

Information was required on the change of thickness of the bituminous layer along the length of each test site, to ensure that the correct temperature correction chart was selected.
More than 100 mm of sound cement-bound material present → YES

Use chart for cement bound road-base

NO

Cumulative total of bituminous material present greater than 150 mm → YES

Deduct thicknesses of striped material
Deduct thickness of surface dressing layers unless more than 25 mm thick

NO

Is cumulative thickness of bituminous material still greater than 150 mm → YES

Use chart for bitumen bound road-base

NO

Cumulative total of granular road-base whose aggregate is naturally cementing is greater than 150 mm → YES

Use chart for granular road-base with natural cementing action

NO

Use chart for granular road-base without cementing action

Figure 11.3 Selection of the Appropriate Deflection Performance Chart
The effect of temperature on the measured value of deflection is dependent upon the proportion of the pavement's stiffness that is derived from the bituminous materials. The greater the thickness of bituminous material present the greater the effect of temperature on the measured deflection value.

Broadly linear relationships between deflection and pavement temperature have been established for the wide range of pavement types built on subgrades within the C.B.R. strength limit of 2.5 to 15 per cent. Temperature correction charts have been developed for the adjustment of deflections, measured within the working range of 10°C to 30°C, to equivalent values at the standard temperature of 20°C. The variation in bituminous layer thickness along each site was determined directly from the extracted cores and is given in Table 11.5.

11.5 THE USE OF THE DEFLECTOGRAPH ON LIGHTLY TRAFFICKED ROADS

11.5.1 Introduction

A Deflectograph on loan from TRRL was used to measure the maximum deflection and curvature of the pavement's surface on all of the lightly trafficked test sites in Devon and Somerset.

Maximum deflection was recorded in the usual manner and additional digital equipment allowed the curvature to be defined as a series of ordinate deflection readings.

![Figure 11.4 Deflected shaped defined by the maximum deflection and ten ordinate deflections.](image_url)
<table>
<thead>
<tr>
<th>SITE 1A - A381 - TOTNES TO HALWELL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Position (metres)</td>
</tr>
<tr>
<td>------------------------</td>
</tr>
<tr>
<td>500</td>
</tr>
<tr>
<td>1240</td>
</tr>
<tr>
<td>1750</td>
</tr>
<tr>
<td>2156</td>
</tr>
<tr>
<td>2500</td>
</tr>
<tr>
<td>2991</td>
</tr>
<tr>
<td>6500</td>
</tr>
<tr>
<td>7250</td>
</tr>
<tr>
<td>7725</td>
</tr>
<tr>
<td>8397</td>
</tr>
<tr>
<td>8872</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SITE 1B - A381 - HALWELL TO RIVER WASH BRIDGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Position (metres)</td>
</tr>
<tr>
<td>------------------------</td>
</tr>
<tr>
<td>8365</td>
</tr>
<tr>
<td>7996</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SITE 2 - A379 - MODBURY TO AVETON GIFFORD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Position (metres)</td>
</tr>
<tr>
<td>------------------------</td>
</tr>
<tr>
<td>250</td>
</tr>
<tr>
<td>375</td>
</tr>
<tr>
<td>500</td>
</tr>
<tr>
<td>650</td>
</tr>
<tr>
<td>700</td>
</tr>
<tr>
<td>1250</td>
</tr>
<tr>
<td>2750</td>
</tr>
<tr>
<td>3250</td>
</tr>
<tr>
<td>3750</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SITE 3 - B3210 - A379 JUNCTION TO UGBOROUGH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Position (metres)</td>
</tr>
<tr>
<td>------------------------</td>
</tr>
<tr>
<td>360</td>
</tr>
<tr>
<td>1250</td>
</tr>
<tr>
<td>1900</td>
</tr>
<tr>
<td>2150</td>
</tr>
<tr>
<td>2750</td>
</tr>
</tbody>
</table>

Table 11.5 (Part 1) 296
<table>
<thead>
<tr>
<th>SITE 4 - B3218 - OKEHAMPTON TO HALWELL</th>
<th>Depth of bituminous bound material (millimetres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Position (metres)</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>140</td>
</tr>
<tr>
<td>750</td>
<td>180</td>
</tr>
<tr>
<td>1700</td>
<td>140</td>
</tr>
<tr>
<td>3000</td>
<td>190</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SITE 5A - UNCLASSIFIED - CORNWOOD TO THE QUARRY</th>
<th>Depth of bituminous bound material (millimetres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Position (metres)</td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>100</td>
</tr>
<tr>
<td>260</td>
<td>115</td>
</tr>
<tr>
<td>380</td>
<td>100</td>
</tr>
<tr>
<td>515</td>
<td>135</td>
</tr>
<tr>
<td>907</td>
<td>170</td>
</tr>
<tr>
<td>1175</td>
<td>210</td>
</tr>
<tr>
<td>1374</td>
<td>160</td>
</tr>
<tr>
<td>1450</td>
<td>120</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SITE 5B - UNCLASSIFIED - TOLCHMOOR GATE TO CADOVER BRIDGE</th>
<th>Depth of bituminous bound material (millimetres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Position (metres)</td>
<td></td>
</tr>
<tr>
<td>3275</td>
<td>50</td>
</tr>
<tr>
<td>3680</td>
<td>50</td>
</tr>
<tr>
<td>4020</td>
<td>40</td>
</tr>
<tr>
<td>4510</td>
<td>50</td>
</tr>
<tr>
<td>5000</td>
<td>160</td>
</tr>
<tr>
<td>5456</td>
<td>160</td>
</tr>
<tr>
<td>6438</td>
<td>85</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SITE 6 - UNCLASSIFIED - LEE MOOR TO PLYMPTON</th>
<th>Depth of bituminous bound material (millimetres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Core Position (metres)</td>
<td></td>
</tr>
<tr>
<td>170</td>
<td>160</td>
</tr>
<tr>
<td>1214</td>
<td>110</td>
</tr>
<tr>
<td>2300</td>
<td>110</td>
</tr>
<tr>
<td>2997</td>
<td>120</td>
</tr>
<tr>
<td>3514</td>
<td>140</td>
</tr>
<tr>
<td>4107</td>
<td>155</td>
</tr>
</tbody>
</table>

Table 11.6 (Part 2) 297
Deflectograph surveys were undertaken every Spring and Autumn, from October 1978 to April 1981, with the machine active for an average of eight hours per day covering in that time a distance of between ten and sixteen kilometres.

Permission to conduct these surveys was sought from the relevant County Councils and details were circulated to the Divisional Surveyors and local Police prior to each survey.

Five Deflectograph surveys have been completed representing about 200,000 individual deflection measurements recorded on a total of 400 kilometres of lightly trafficked road.

11.5.2 Problems associated with operating the Deflectograph on Lightly Trafficked Roads

Initially the recommended procedures for the operation of a Deflectograph, as stated in LR835, were adopted. However, the use of the Deflectograph on lightly trafficked roads immediately highlighted a number of potential operating problems.

Sight distances at brows of hills and on sharp bends and the difficulty of providing traffic control on narrow sections of the carriageway were important from a safety aspect.

These problems were overcome using a modified operating procedure involving two men controlling the traffic with stop/go boards. One man controlled the oncoming traffic from about 50 metres in front of the Deflectograph and a second man controlled the following traffic from about 20 metres behind the machine. This system of traffic control was operated on all five Deflectograph surveys and worked extremely well in each case.

Distance measurement is normally recorded with sufficient accuracy using a fifth wheel fitted at the rear of the vehicle. However, under extreme conditions of camber and rutting on the lightly trafficked roads, the fifth wheel on occasions either completely lost contact with the road surface or slewed as a result of the reduction
in the available contact pressure. Both eventualities resulted in an under-recording of the distance travelled.

At times distance was over-recorded due to the wander of the machine as a consequence of an insufficiently defined nearside wheelpath.

Under and over-recording of distance by the fifth wheel made it extremely difficult to locate the exact position of the individual deflection readings, especially on sections with few features capable of being used for location purposes.

Subsequent sections will show that the identification of each point of deflection measurement was shown to be essential to allow deflection to be related to the pavement condition at the exact point of measurement. Failure to accurately define the positions of the deflection measurements often resulted in a mismatch between deflection and visual condition because of the effect on the deflection measurements of the rapid changes in strength both across and along these lightly trafficked road sections.

Similar accuracy was not thought necessary for Deflectograph surveys associated with routine maintenance. Identification of the position of each deflection measurement was aided by painting chainage marks at 250 m intervals and entering these marks as events on the deflection profile in the normal manner.

11.6 PROCESSING THE DEFLECTION DATA

11.6.1 Introduction

A vast amount of deflection data was obtained with the Deflectograph from five separate fourteen day surveys of the test sites in Devon and Somerset, amounting to about 200,000 individual deflection readings. The deflection data was output from the Deflectograph in three forms: paper tape from the tape punch, influence lines from the chart recorder, and tabulations from the line printer. In order to take into account the effect of equipment type and pavement temperature on the measured value of deflection, it was necessary
initially, to standardise it to equivalent Deflection Beam deflections temperature correct to 20°C.

Standardisation was usually achieved by inputting the paper tapes for each site directly into the mainframe computer at the Polytechnic and processing them using the DEFLEC suite of computer programmes. The DEFLEC system allows large amounts of deflection data to be processed quickly in accordance with the method outlined in LR833. (35)

Smaller amounts of raw Deflectograph deflection measurements collected on lengths of particular interest were standardised to equivalent Deflection Beam deflections and temperature corrected using a BASIC computer programme written for use with a PET desk top computer. (66) This particular method allowed individual deflection readings to be processed in accordance with the method outlined in LR833. (35)

An alternative method used to correct individual raw Deflectograph deflection readings, prior to the development of the BASIC computer programme, was the manual use of the appropriate charts given in LR833. (35) This method of correction was slow in comparison with the use of the DEFLEC programme and PET desk top computer.

The graphical output from the chart recorder on board the Deflectograph was not normally used for correction purposes because the information was also available in more convenient forms: paper tape and line printer tabulation.

This graphical deflection measurement information was, however, often used to determine the validity of particular deflection values during correction using data from either paper tape or the line printer.

11.6.2 Use of the Mainframe Computer

The source programmes for the DEFLEC system, appropriate to the methods given in LR833, (35) were obtained half way through the project from the Transport and Road Research Laboratory and were successfully amended for use on the Prime Dual 550 computer system at the Polytechnic. In fact this was the second DEFLEC system to be amended and used on the Polytechnic computer. The first system was available at the start of
this project and allowed processing of the deflection data to the previous strengthening design method outlined in LR571(65).

A large amount of deflection data was processed using this initial DEFLEC system because of the need to use the deflection data to make decisions early in the project concerning site selection and the subsequent reduction in the total length of road surveyed.

Deflection data processed to the method outlined in LR571(65) has also been re-processed using the more detailed method given in LR833.

The DEFLEC system now in use is a suite of computer programmes designed to process measurements of transient deflection according to the methods outlined in LR833(35). The programmes are written in FORTRAN and are suitable for running on most Mainframe and Minicomputer installations.

The DEFLEC system will accept data produced by either of the two recording systems in current use in the United Kingdom.

The DEFLEC system is divided into three stages:

- Input stage ------ TIDY ------ TIN
- Editing stage ------ EDIT ------ EIN
- Output stage ------ DISPLAY ------ DIN

11.6.2.1 Input Stage

All deflection data produced on paper tape is input to the system through the input stage using programme TIDY. The functions performed by this programme are vetting, filing and listing the data.

11.6.2.2 Editing Stage

This stage involves programme EDIT and provides the user with the facility for editing the deflection data and for input of information
describing the construction, condition, traffic, past and future life, overlay thicknesses, life probabilities and wheel loadings for each site surveyed. Control data (EIN) is used to define the form of output from both the editing and output stages.

The traffic data is processed and threshold maximum deflection levels determined for the future required pavement life. These levels are determined for the existing and overlaid pavement.

The input and processed data is merged together with the deflection data in order that it may be listed during the output stage.

11.6.2.3 Output Stage

Programme DISPLAY is used to process the deflection data from each wheelpath with corrections made for non-standard wheel loadings, for differences between the Deflectograph and the Deflection Beam and for surface temperature.

The resulting values are output as deflection profiles.

Two types of output can be produced. The output may show two deflection profiles for both wheelpaths, or a single profile of the maximum deflection taken from both wheelpaths. In the latter case the residual pavement life corresponding to each deflection value is output also. The residual life is shown to the nearest whole year except when the pavement has exceeded the critical condition in which case a value of -99 is shown.

The deflection values shown on the profiles may consist of individual measurements as described above or means of three consecutive readings, or means of all readings occurring within a \(10\) m length of the pavement. In the case of the maximum deflection output meaning may be between measurements which are not in the same wheel track.

11.6.3 Use of the Minicomputer

A Basic computer programme capable of processing individual deflection
measurements in accordance with the method given in LR833\(^{(35)}\) has been developed within the Department of Civil Engineering for use on a PET desk top computer.\(^{(66)}\)

The programme can be used to predict the residual life and recommend possible overlay thicknesses based on an individual characteristic deflection value for a particular length of road. It can also be used for a sensitivity analysis to rapidly determine the effect on a particular strengthening recommendation of a change in one or other of the input parameters, i.e., higher or lower characteristic deflection level, error in traffic estimation, error in pavement temperature, etc.

### 11.7 VISUAL CONDITION SURVEYS

The visual condition of the pavement's surface, in terms of the amount of rutting and cracking in the wheelpaths, was recorded at the time of each Deflectograph survey. Interpretation of this information, in terms of the recommendations given in Table 11.6 LR 833\(^{(35)}\), allowed the total length of each test site to be classified as being in a SOUND, CRITICAL or FAILED condition.

Combining this information with the measured deflections resulted in the definition of typical deflection levels associated with lightly trafficked pavement sections in a SOUND, CRITICAL or FAILED condition.

#### 11.7.1 The procedure adopted

Two people were required to undertake a visual condition survey of the pavement's surface. One person would walk the length of each test site with a distance measuring device recording the size of any rutting under a two metre straightedge laid transversely across the carriageway and the extent of any cracking. A second person would follow behind the first driving a car with a flashing beacon and hazard warning lights switched on to give notice to on-coming vehicles of the possible danger.

Visual condition information was recorded for both nearside and offside wheelpaths, and for routine condition surveys was assessed in 10 m
<table>
<thead>
<tr>
<th>CLASSIFICATION</th>
<th>CODE</th>
<th>VISIBLE EVIDENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>SOUND</td>
<td>1</td>
<td>No Cracking. Rutting under a 2 m straight-edge less than 5 mm.</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>No Cracking. Rutting from 5 m to 9 mm.</td>
</tr>
<tr>
<td>CRITICAL</td>
<td>3</td>
<td>No Cracking. Rutting from 10 mm to 19 mm.</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Cracking confined to a single crack or extending over less than half of the width of the wheel path. Rutting 19 mm or less.</td>
</tr>
<tr>
<td>FAILED</td>
<td>5</td>
<td>Interconnected multiple cracking extending over the greater part of the width of the wheel path. Rutting 19 mm or less.</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>No Cracking. Rutting 20 mm or greater.</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>Cracking confined to a single crack or extending over less than half of the width of the wheel path. Rutting 20 mm or greater.</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>Interconnected multiple cracking extending over the greater part of the width of the wheel path. Rutting 20 mm or greater.</td>
</tr>
</tbody>
</table>

Table 11.6 Classification of the condition of the road surface (after Kennedy and Lister)
lengths. Subsequent surveys on shorter lengths for which greater detail was required were undertaken at 5 or 1 m intervals. In total some 200 Km of road was surveyed at 10 m intervals and 10 Km at 5 m and 1 m intervals.

11.7.2 Condition Survey Sheets

Figure 11.5 shows the format of the sheets used to record the visual condition information. Each rectangle on the sheet represented a pavement length of either 10 m, 5 m, or 1 m, depending upon the detail required.

The first row marked EVENT L and the row marked EVENT R were used to record the position of the events identified during the Deflectograph survey. The events normally recorded were road junctions, telephone and post boxes, telegraph poles, signposts, etc. Chainage markers painted on the pavement's surface were also entered as events.

The second row marked WT Crakg L and the row marked WT Crakg R were used to record the extent of any cracking in the nearside (L) and offside (R) wheelpaths. A single line was used to denote single cracks and double lines with smaller lines at right-angles to them were used to denote interconnected multiple cracking.

The third row marked WT Rut L and the fourth row marked WT Rut R were used to record the size of any rutting in the nearside (L) and offside (R) wheelpaths.

Typical records for a 10 and 1 m grid are shown in Figure 11.5.

11.7.3 Classification of pavement condition

The information recorded on these sheets was used to classify the condition of each test site in accordance with the recommendations given in Table 11.6.

The visual evidence relating to each classification given in this table was deduced from observations made of the development of permanent deformation (rutting) and cracking on the full-scale experimental pavement design sections monitored by TRRL (full details given in 305
Figure 11.5 Condition Survey Sheet
Each 10, 5 or 1 m length of each test site was classified as being in a SOUND, CRITICAL or FAILED condition.

This pavement condition information was subsequently used in conjunction with the deflection data, measured by the Deflectograph, to determine the deflection levels associated with each of these three conditions.

11.8 COMMERCIAL VEHICLE STUDIES

11.8.1 Introduction

The slice in time methodology, for relating deflection and performance, requires that the slice in time is identified in terms of the cumulative number of standard axles carried by the pavement since construction, or more appropriately for the undesigned roads considered in this project since last major strengthening or reconstruction.

To express commercial traffic as a cumulative number of standard axles, information is required on the present yearly flow of commercial vehicles and the number of standard axles per commercial vehicle, the growth rate and change in standard axle factor with time.

Estimates of the present yearly flow were made from the average number of commercial vehicles using each site daily; a figure determined either from existing data or from specific traffic counts.

Details are given of the traffic counts undertaken and the findings are discussed in relation to published data.

The number of standard axles per commercial vehicle or damage factor was determined as the product of the average number of axles per commercial vehicle obtained from the traffic counts, and the number of standard axles, per commercial axle, derived from the axle weight data. Commercial vehicle axle weights were measured using a portable weighing system and details of this equipment, its accuracy and use on the
experimental sites are presented.

The measured axle and vehicle damage factors are compared with the values suggested by the published data as being applicable to the sites under consideration.

A method of estimating damage factors is proposed, taking into account the number and types of vehicle in each classification.

11.8.2 Commercial Vehicle Traffic Counts

Commercial vehicles travelling in both directions were counted for the duration of each one day axle weight survey, using the classifications shown in Figure 11.6. The method adopted for these counts was based upon the recommendations suggested by TRRL for accurately estimating annual flows from short period counts\(^{(67)}\)

11.8.2.1 Twelve hour, five consecutive day traffic count

In addition a twelve hour, five consecutive day traffic count was undertaken on the A379 Modbury to Aveton Gifford site on Monday, 12th May, to Friday, 16th May 1980. May was the month suggested\(^{(67)}\) as giving the greatest accuracy for calculating a yearly flow from a five day count. The purpose of this extended count was to determine the daily flow of commercial vehicles in each direction over a continuous period and thus obtain an indication of the change in the daily estimation of the average number of axles per commercial vehicle.

The results are shown in Table 11.7 and are presented as average numbers of axles per commercial vehicle. The figures in brackets are the numbers of commercial vehicles counted during the 12 hours.

Comparison of the average daily values for both directions shows that they are virtually identical, which suggests that there is no great bias towards one particular type of vehicle travelling in either direction. This does not mean that the axle load spectra are the same for both directions, only that a similar number of axles passed in each direction. Directional variation in axle load cannot be readily
<table>
<thead>
<tr>
<th>Class</th>
<th>Vehicle</th>
<th>No.</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Moped, scooter, motorcycle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Car, light van, taxi</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Light goods vehicle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>Car or light goods vehicle + 1 axle caravan or trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>Car or light goods vehicle + 2 axle caravan or trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>31</td>
<td>Rigid 2 axle heavy goods vehicle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>32</td>
<td>Rigid 3 axle heavy goods vehicle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>33</td>
<td>Rigid 4 axle heavy goods vehicle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>34</td>
<td>Rigid 3 axle heavy goods vehicle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>Rigid 4 axle heavy goods vehicle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>41</td>
<td>Rigid 2 axle HGV + 2 axle drawbar trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>42</td>
<td>Rigid 2 axle HGV + 3 axle drawbar trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>43</td>
<td>Rigid 3 axle HGV + 2 axle drawbar trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>44</td>
<td>Rigid 3 axle HGV + 3 axle drawbar trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>45</td>
<td>Rigid 2 axle HGV + 1 axle caravan or trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>46</td>
<td>Rigid 2 axle HGV + 2 axle (close coupled) trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>51</td>
<td>Artic, 2 axle tractor + 1 axle semi-trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>52</td>
<td>Artic, 2 axle tractor + 2 axle semi-trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>53</td>
<td>Artic, 3 axle tractor + 1 axle semi-trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>54</td>
<td>Artic, 3 axle tractor + 2 axle semi-trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>55</td>
<td>Artic, 2 axle tractor + 3 axle semi-trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>56</td>
<td>Artic, 3 axle tractor + 3 axle semi-trailer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>61</td>
<td>Bus or coach, 2 axle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>62</td>
<td>Bus or coach, 3 axle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>61</td>
<td>Vehicle with 7 or more axles</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6N</td>
<td>6 axle vehicle not otherwise classified</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Vehicle with 7 or more axles</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 11.6 TRRL Vehicle class listing compatible with EEC regulation
<table>
<thead>
<tr>
<th>DIRECTION</th>
<th>MONDAY 12.5.80</th>
<th>TUESDAY 13.5.80</th>
<th>WEDNESDAY 14.5.80</th>
<th>THURSDAY 15.5.80</th>
<th>FRIDAY 16.5.80</th>
<th>AVERAGE DAILY</th>
<th>WEDNESDAY 24.10.79</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOWARDS</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aveton Gifford</td>
<td>2.26 (116)</td>
<td>2.24 (115)</td>
<td>2.41 (120)</td>
<td>2.31 (105)</td>
<td>2.14 (90)</td>
<td>2.27 (109)</td>
<td>2.12 (30)</td>
</tr>
<tr>
<td>TOWARDS</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Modbury</td>
<td>2.25 (105)</td>
<td>2.25 (110)</td>
<td>2.44 (121)</td>
<td>2.24 (119)</td>
<td>2.23 (98)</td>
<td>2.28 (110)</td>
<td>2.24 (42)</td>
</tr>
</tbody>
</table>

Table 11.7 Number of Axles per Commercial Vehicle for the Modbury to Aveton Gifford Site
determined when comparing numbers of vehicles in each direction. It
is of great importance because pavement damage is not related to
vehicle numbers but axle load applications.

The 12 hour commercial vehicle flows are not the same for each of the
five days indicating the problem of choice of day on which to
undertake a traffic count.

The duration of the survey is also important and is reflected by the
difference between the average daily 12 hour figure (2.27) and that for
a 4 hour count on 24th October 1979 (2.12). The problem is that
similar proportions of vehicles in each classification are not
normally achieved with counts of different duration. Although this
was thought to be the case for the difference stated, other factors,
such as seasonal variation and the date difference, could also have
played a part.

11.8.2.2 Number of Axles per Commercial Vehicle

The number of axles per commercial vehicle for each site is shown in
Table 11.8 and was determined from seven hour traffic counts for all
sites, apart from the figure for the Modbury site which was determined
from the more accurately weekly count.

<table>
<thead>
<tr>
<th>Site</th>
<th>Number of Axles per Commercial Vehicle</th>
</tr>
</thead>
<tbody>
<tr>
<td>A381 Harberton</td>
<td>2.19</td>
</tr>
<tr>
<td>A379 Modbury</td>
<td>2.27</td>
</tr>
<tr>
<td>B3210 Ermington</td>
<td>2.56</td>
</tr>
<tr>
<td>B3218 Okehampton</td>
<td>2.29</td>
</tr>
<tr>
<td>Unclassified Cornwood</td>
<td>3.00</td>
</tr>
<tr>
<td>Unclassified Lee Moor</td>
<td>2.72</td>
</tr>
<tr>
<td>A38 Plympton By-Pass</td>
<td>2.69</td>
</tr>
</tbody>
</table>

Table 11.8 Average number of Axles per Commercial Vehicle
The calculated average number of axles per commercial vehicle for each site are shown in relation to the estimated national average values in Figure 11.7. All of the sites considered, except the A38, have a daily flow in each direction which is considerably less than 250 commercial vehicles. The A38 has a daily flow in each direction of between 1000 and 2000 commercial vehicles. The large values, corresponding to the unclassified roads at Cornwood and Lee Moor, are due to the large proportion of multi-axle vehicles using the roads to the quarries. It is obvious that use could not be made of the appropriate national average factor to estimate the number of axles per commercial vehicle on these 'special' roads.

Other roads which could fall into this 'special' category are roads leading to and from industrial estates, oil refineries and ports.

It is felt that the position shown for the Ermington site is abnormally high and shows the effect of several multi-axle vehicles on a single small traffic count.

11.8.3 Portable Weighbridge Surveys

11.8.3.1 Weighbridge Equipment

A portable twin weighpad system was purchased from Trevor Deakin Ltd. of Bath, primarily to determine the axle load spectra for each site. The equipment is completely portable and consists of an axle weighing indicator unit and two aluminium alloy weighbridge platforms. The axle weighing indicator unit provides a digital read-out and contains its own power source which usually has enough power for 4 to 6 hours continuous work. Longer survey periods require that an external source, such as a vehicle battery, be connected for subsequent work.

The indicator unit has zero setting and sensitivity controls, a facility for electrical calibration of the weighpads and a battery voltage checking control. Each weighpad can be read individually giving nearside and offside wheel loads, or together to give the axle load.
Figure 11.7 The calculated number of axles per commercial vehicle for each site superimposed on the estimated changes in number of axles per commercial vehicle for four traffic levels: 1945 to 2005 (after Currer and O'Connor) (7)
The weighpads are of high strength aluminium alloy and consist of an upper and lower platform which sandwich six load cells. Each platform measures 700 x 500 x 90 mm and weighs less than 45 kg.

The six load cells are so placed to reduce adverse effects arising from eccentric loading of the platforms. Overloading or incorrect positioning of a vehicle's wheels on the platform will activate the overload switches, which in turn operate a lamp on the indicator unit so giving a warning. Each platform has a range of 0 - 10,000 kg and the digital indicator unit gives readings in increments of 10 kg for each platform to a total of 19,990 kg for both platforms.

11.8.3.2 Use of the Equipment

The weighbridge system can be used either with ramps or recessed in pits. The use of ramps was adopted in this study to allow the system to be taken to any lay-by or length of superseded road on the test sites. Wooden ramps and dummy platforms were constructed to ensure that the vehicle being weighed was as near as possible on the one horizontal plane. Figure 11.8 shows the position of the weighbridge in relation to the ramps and dummy platforms.

The main criticism of the portable weighbridge equipment is that the internal power source is only sufficient for about 4 hours continuous operation. A vehicle battery normally had to be connected to the system to finish a survey.

Precipitation during a number of the surveys meant that on occasions the weighbridge was operated in damp conditions. The main problem associated with the operation under these conditions was vehicle wheels slipping on the wet wooden dummy pads and ramps. Slipping of the wheels was effectively controlled by spreading sand on the wooden dummy pads and ramps to increase the available frictional resistance.

11.8.3.3 Accuracy of Portable Equipment

The accuracy of two identical weighbridge systems (system A and system B) was achieved by comparing gross vehicle weights obtained using such
Figure 11.8 Axle Weighbridge with Ramps and Dummy Platforms.
systems with the results from three different public weighbridges. The procedure adopted in all cases was to measure the weight of each commercial vehicle with the portable weighbridge immediately after it had been weighed on the public weighbridge. The results obtained with system A for two-axle rigid commercial vehicles are presented in Tables 11.9 and 11.10, and those obtained with system B, also for two-axle rigid commercial vehicles, are presented in Table 11.11.

Percentage differences in gross vehicle weight are given in these tables; a positive sign indicates an overestimate by the portable equipment and a negative sign indicates an underestimate.

Analysis of the results given in Tables 11.9 and 11.10 suggests that system A generally overestimates the gross vehicle weight by an average of 2 per cent.

Analysis of the results given in Table 11.11 suggests that system B generally underestimates the gross vehicle weight by 2.7 per cent.

These inaccuracies are well within acceptable limits because of the vast increase in the overall accuracy of the estimate of traffic obtained when using the data determined with the portable equipment compared with the inherent inaccuracies associated with the use of the national average figures. Tables 11.12 to 11.15 inclusive show comparison data obtained when weighing multi-axle commercial vehicles and although the number of vehicles involved in each case is small, this information serves to show the continued accuracy of the equipment when used to weigh multi-axle vehicles.

Tests were undertaken to determine the effect of inclination of the weighpad on the measured value of the load.

Loads of different magnitude were applied to the weighpad inclined at angles up to 15 degrees to the horizontal. The effect of inclination both along and across the weighpad was investigated. In each case the error in the measured load was found to be within ± 5% of the true load.
### TWO AXLE RIGID VEHICLES

<table>
<thead>
<tr>
<th>PUBLIC WEIGHBRIDGE (kg)</th>
<th>PORTABLE WEIGHBRIDGE (kg)</th>
<th>% DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>12170</td>
<td>12390</td>
<td>+ 1.8</td>
</tr>
<tr>
<td>8450</td>
<td>8400</td>
<td>- 0.6</td>
</tr>
<tr>
<td>8195</td>
<td>8000</td>
<td>- 2.4</td>
</tr>
<tr>
<td>7165</td>
<td>7260</td>
<td>+ 1.3</td>
</tr>
<tr>
<td>8310</td>
<td>8510</td>
<td>+ 2.4</td>
</tr>
<tr>
<td>8810</td>
<td>9100</td>
<td>+ 3.2</td>
</tr>
<tr>
<td>6705</td>
<td>6670</td>
<td>- 0.5</td>
</tr>
<tr>
<td>8160</td>
<td>8290</td>
<td>+ 1.6</td>
</tr>
<tr>
<td>9035</td>
<td>9170</td>
<td>+ 1.5</td>
</tr>
<tr>
<td>8345</td>
<td>8330</td>
<td>- 0.2</td>
</tr>
<tr>
<td>8905</td>
<td>9120</td>
<td>+ 2.4</td>
</tr>
</tbody>
</table>

**Average Overestimate = 2%**

**Table 11.9 Comparison Gross Vehicle Weight - City Engineers**

<table>
<thead>
<tr>
<th>PUBLIC WEIGHBRIDGE (kg)</th>
<th>PORTABLE WEIGHBRIDGE (kg)</th>
<th>% DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>4000</td>
<td>4030</td>
<td>+ 0.75</td>
</tr>
<tr>
<td>16550</td>
<td>16390</td>
<td>- 0.97</td>
</tr>
<tr>
<td>16150</td>
<td>15940</td>
<td>- 1.3</td>
</tr>
<tr>
<td>16900</td>
<td>16940</td>
<td>+ 0.2</td>
</tr>
<tr>
<td>15800</td>
<td>15930</td>
<td>+ 0.8</td>
</tr>
<tr>
<td>16150</td>
<td>16340</td>
<td>+ 1.2</td>
</tr>
<tr>
<td>15950</td>
<td>16000</td>
<td>+ 0.3</td>
</tr>
<tr>
<td>16000</td>
<td>16190</td>
<td>+ 1.2</td>
</tr>
<tr>
<td>16900</td>
<td>16980</td>
<td>+ 0.5</td>
</tr>
<tr>
<td>16450</td>
<td>16610</td>
<td>+ 1.0</td>
</tr>
<tr>
<td>16100</td>
<td>16250</td>
<td>+ 0.9</td>
</tr>
<tr>
<td>15350</td>
<td>15540</td>
<td>+ 1.2</td>
</tr>
<tr>
<td>15650</td>
<td>15790</td>
<td>+ 0.9</td>
</tr>
</tbody>
</table>

**Average Overestimate = 0.8%**

**Table 11.10 Comparison Gross Vehicle Weight - Lee Moor**

317
<table>
<thead>
<tr>
<th>PUBLIC WEIGHBRIDGE (kg)</th>
<th>PORTABLE WEIGHBRIDGE (kg)</th>
<th>% DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>6490</td>
<td>6420</td>
<td>-1.1</td>
</tr>
<tr>
<td>16340</td>
<td>16000</td>
<td>-2.1</td>
</tr>
<tr>
<td>16150</td>
<td>15000</td>
<td>-7.1</td>
</tr>
<tr>
<td>11060</td>
<td>10760</td>
<td>-2.7</td>
</tr>
<tr>
<td>5760</td>
<td>5640</td>
<td>-2.1</td>
</tr>
<tr>
<td>11800</td>
<td>11650</td>
<td>-1.3</td>
</tr>
<tr>
<td>7340</td>
<td>7150</td>
<td>-4.0</td>
</tr>
<tr>
<td>8480</td>
<td>8240</td>
<td>-2.8</td>
</tr>
<tr>
<td>12890</td>
<td>12820</td>
<td>-0.5</td>
</tr>
<tr>
<td>9670</td>
<td>9460</td>
<td>-2.2</td>
</tr>
<tr>
<td>12640</td>
<td>12670</td>
<td>+0.2</td>
</tr>
<tr>
<td>4420</td>
<td>4460</td>
<td>+0.9</td>
</tr>
<tr>
<td>11240</td>
<td>11110</td>
<td>-1.2</td>
</tr>
<tr>
<td>6760</td>
<td>6810</td>
<td>+0.7</td>
</tr>
<tr>
<td>11590</td>
<td>11860</td>
<td>+2.3</td>
</tr>
<tr>
<td>5010</td>
<td>5200</td>
<td>+3.8</td>
</tr>
<tr>
<td>8730</td>
<td>8620</td>
<td>-1.3</td>
</tr>
<tr>
<td>8510</td>
<td>8820</td>
<td>+3.6</td>
</tr>
<tr>
<td>7550</td>
<td>7260</td>
<td>-3.8</td>
</tr>
<tr>
<td>7240</td>
<td>6970</td>
<td>-3.7</td>
</tr>
<tr>
<td>8840</td>
<td>8620</td>
<td>-2.5</td>
</tr>
<tr>
<td>5070</td>
<td>4920</td>
<td>-3.0</td>
</tr>
<tr>
<td>5350</td>
<td>5250</td>
<td>-1.9</td>
</tr>
<tr>
<td>5230</td>
<td>5030</td>
<td>-3.8</td>
</tr>
<tr>
<td>15930</td>
<td>15670</td>
<td>-1.6</td>
</tr>
<tr>
<td>7050</td>
<td>6950</td>
<td>-1.4</td>
</tr>
<tr>
<td>8230</td>
<td>7880</td>
<td>-4.3</td>
</tr>
<tr>
<td>5190</td>
<td>5190</td>
<td>0</td>
</tr>
<tr>
<td>8950</td>
<td>8830</td>
<td>-1.4</td>
</tr>
<tr>
<td>9320</td>
<td>10110</td>
<td>+8.5</td>
</tr>
<tr>
<td>9000</td>
<td>8830</td>
<td>-1.9</td>
</tr>
<tr>
<td>7780</td>
<td>7560</td>
<td>-2.8</td>
</tr>
<tr>
<td>15380</td>
<td>15010</td>
<td>-2.4</td>
</tr>
<tr>
<td>5470</td>
<td>5100</td>
<td>-6.8</td>
</tr>
<tr>
<td>15720</td>
<td>15560</td>
<td>-1.0</td>
</tr>
<tr>
<td>4670</td>
<td>4470</td>
<td>-4.3</td>
</tr>
<tr>
<td>6060</td>
<td>6060</td>
<td>0</td>
</tr>
<tr>
<td>10580</td>
<td>10360</td>
<td>-2.1</td>
</tr>
</tbody>
</table>

Average Underestimate = 2.7%

Table 11.11 Two Axle Rigid - Comparison Gross Vehicle Weight
### Table 11.12 Three Axle Rigid - Gross Weight Comparison

<table>
<thead>
<tr>
<th>PUBLIC WEIGHBRIDGE (kg)</th>
<th>PORTABLE WEIGHBRIDGE (kg)</th>
<th>% DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>15650</td>
<td>15460</td>
<td>- 1.2</td>
</tr>
<tr>
<td>22190</td>
<td>22240</td>
<td>+ 0.2</td>
</tr>
<tr>
<td>20120</td>
<td>20510</td>
<td>+ 2.0</td>
</tr>
</tbody>
</table>

### Table 11.13 Four Axle Rigid - Gross Weight Comparison

<table>
<thead>
<tr>
<th>PUBLIC WEIGHBRIDGE (kg)</th>
<th>PORTABLE WEIGHBRIDGE (kg)</th>
<th>% DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>24050</td>
<td>24880</td>
<td>+ 3.5</td>
</tr>
<tr>
<td>30190</td>
<td>30720</td>
<td>+ 1.8</td>
</tr>
<tr>
<td>19280</td>
<td>19850</td>
<td>+ 3.0</td>
</tr>
<tr>
<td>24730</td>
<td>25110</td>
<td>+ 1.5</td>
</tr>
<tr>
<td>26710</td>
<td>27980</td>
<td>+ 4.8</td>
</tr>
</tbody>
</table>

### Table 11.14 Three Axle Articulated - Comparison Gross Vehicle Weight

<table>
<thead>
<tr>
<th>PUBLIC WEIGHBRIDGE (kg)</th>
<th>PORTABLE WEIGHBRIDGE (kg)</th>
<th>% DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>10230</td>
<td>9600</td>
<td>- 6.2</td>
</tr>
<tr>
<td>13030</td>
<td>12120</td>
<td>- 7.0</td>
</tr>
<tr>
<td>11930</td>
<td>11780</td>
<td>- 1.3</td>
</tr>
</tbody>
</table>

### Table 11.15 Four Axle Articulated - Comparison Gross Vehicle Weight

<table>
<thead>
<tr>
<th>PUBLIC WEIGHBRIDGE (kg)</th>
<th>PORTABLE WEIGHBRIDGE (kg)</th>
<th>% DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>29410</td>
<td>29220</td>
<td>- 0.7</td>
</tr>
<tr>
<td>31880</td>
<td>32420</td>
<td>+ 1.7</td>
</tr>
<tr>
<td>20970</td>
<td>21420</td>
<td>+ 2.2</td>
</tr>
</tbody>
</table>
11.8.3.4 Survey Procedure

One day commercial vehicle axle weight surveys were undertaken on 3 A-class roads (sites 1, 2 and 7), 2 B-class roads (sites 3 and 4) and 2 unclassified roads (sites 5 and 6). The portable weighbridge was sited either in a lay-by or on a length of superseded road on the A and B-class roads. The unclassified roads posed a small problem in that there were no suitable lay-bys or lengths of superseded road. In these cases the portable weighbridge was set up in one lane of the single carriageway road in the direction of travel of the vehicles to be weighed. Cones were positioned along the centre line of the road effectively splitting it into two lanes and a traffic control system operated for all vehicles excluded from the weight survey. This procedure was made possible by the relatively low volume of traffic which used the roads and worked extremely well in each case.

A policeman was hired for each survey to stop the required commercial vehicles. The arrangement with the policeman was that he would stop the required commercial vehicles and then one member of the research team would ask the driver if he would be prepared to have his vehicle weighed. If the driver agreed he would then proceed onto the weighbridge and if he declined he was allowed to continue on his journey.

The portable weighbridge is of the static type which meant that the axles were weighed one at a time with the vehicle stationary for each measurement.

In total some 578 commercial vehicles were weighed with the portable weighbridge during these surveys and in that time only one vehicle refused to go into the lay-by to be weighed. On all roads, apart from the A38, the commercial traffic flow was such that sampling was not necessary and all commercial vehicles travelling in the appropriate direction were weighed.

The commercial traffic flow on the A38 was heavier than that on any other site and a sampling procedure, stopping every fifth vehicle, was implemented.

The time taken to weigh the commercial vehicles was proportional to
the number of axles; two-axle commercial vehicles were weighed in less than 30 seconds and four-axle commercial vehicles in less than 1 minute.

11.8.4 Estimation of Axle and Vehicle Damage Factors

The axle damage factor is the number of standard axles per commercial axle and was obtained for each site by summing the derived standard axle factors and dividing by the total number of axles weighed.

The vehicle damage factor is the number of standard axles per commercial vehicle obtained by multiplying the number of axles per commercial vehicle (obtained from the traffic count) by the axle damage factor (obtained from the vehicle weighing).

11.8.4.1 Axle and Vehicle Damage Factors

The axle and vehicle damage factors for each vehicle classification are presented by site in Tables 11.16 to 11.25 inclusive. These show, for the sites investigated substantial differences between the average vehicle damage factors for the various vehicle classifications. These differences could be due to the variation in the numbers of vehicles weighed and the influence of one or two fully laden vehicles on a small sample. There is also evidence from Tables 11.16 to 11.25 of variation in damage factor between sites for a particular vehicle classification. This variation is due to the difference in the type of use of vehicle within each classification.

Consider, for example, the values derived for the two and three axle rigid classifications for the roads at Ermington (Table 11.16) and Okehampton (Table 11.17).

<table>
<thead>
<tr>
<th></th>
<th>Two Axle Rigid</th>
<th>Three Axle Rigid</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ermington</td>
<td>0.46</td>
<td>3.10</td>
</tr>
<tr>
<td>Okehampton</td>
<td>0.75</td>
<td>1.52</td>
</tr>
</tbody>
</table>

A large number of two-axle distribution vehicles (generally relatively light) and fully laden three-axle brick lorries use the road at Ermington.
<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>NUMBER OF VEHICLES</th>
<th>AXLE DAMAGE FACTOR</th>
<th>VEHICLE DAMAGE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWO AXLE RIGID</td>
<td>23</td>
<td>0.23</td>
<td>0.46</td>
</tr>
<tr>
<td>THREE AXLE RIGID</td>
<td>4</td>
<td>1.03</td>
<td>3.10</td>
</tr>
<tr>
<td>FOUR AXLE RIGID</td>
<td>4</td>
<td>0.48</td>
<td>1.91</td>
</tr>
<tr>
<td>FOUR AXLE ARTIC</td>
<td>4</td>
<td>0.55</td>
<td>2.21</td>
</tr>
</tbody>
</table>

Table 11.16  B3210  Ermington

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>NUMBER OF VEHICLES</th>
<th>AXLE DAMAGE FACTOR</th>
<th>VEHICLE DAMAGE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWO AXLE RIGID</td>
<td>26</td>
<td>0.38</td>
<td>0.75</td>
</tr>
<tr>
<td>THREE AXLE RIGID</td>
<td>7</td>
<td>0.51</td>
<td>1.52</td>
</tr>
<tr>
<td>FOUR AXLE RIGID</td>
<td>1</td>
<td>0.06</td>
<td>0.23</td>
</tr>
<tr>
<td>THREE AXLE ARTIC</td>
<td>1</td>
<td>0.21</td>
<td>0.64</td>
</tr>
<tr>
<td>FOUR AXLE ARTIC</td>
<td>4</td>
<td>0.24</td>
<td>0.97</td>
</tr>
<tr>
<td>TWO AXLE BUS</td>
<td>6</td>
<td>0.07</td>
<td>0.13</td>
</tr>
</tbody>
</table>

Table 11.17  B3218  Okehampton

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>NUMBER OF VEHICLES</th>
<th>AXLE DAMAGE FACTOR</th>
<th>VEHICLE DAMAGE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWO AXLE RIGID</td>
<td>18</td>
<td>0.83</td>
<td>1.66</td>
</tr>
<tr>
<td>THREE AXLE RIGID</td>
<td>19</td>
<td>1.01</td>
<td>3.03</td>
</tr>
<tr>
<td>FOUR AXLE RIGID</td>
<td>19</td>
<td>0.89</td>
<td>3.56</td>
</tr>
<tr>
<td>FOUR AXLE ARTIC</td>
<td>6</td>
<td>0.97</td>
<td>3.88</td>
</tr>
<tr>
<td>TWO AXLE BUS</td>
<td>5</td>
<td>0.20</td>
<td>0.40</td>
</tr>
</tbody>
</table>

Table 11.18  Unclassified - Cornwood
<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>NUMBER OF VEHICLES</th>
<th>AXLE DAMAGE FACTOR</th>
<th>VEHICLE DAMAGE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWO AXLE RIGID</td>
<td>30</td>
<td>0.92</td>
<td>1.84</td>
</tr>
<tr>
<td>THREE AXLE RIGID</td>
<td>6</td>
<td>0.65</td>
<td>1.96</td>
</tr>
<tr>
<td>FOUR AXLE RIGID</td>
<td>7</td>
<td>0.24</td>
<td>0.95</td>
</tr>
<tr>
<td>THREE AXLE ARTIC</td>
<td>1</td>
<td>0.05</td>
<td>0.15</td>
</tr>
<tr>
<td>FOUR AXLE ARTIC</td>
<td>9</td>
<td>0.93</td>
<td>3.72</td>
</tr>
</tbody>
</table>

Table 11.19 Unclassified - Lee Moor

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>NUMBER OF VEHICLES</th>
<th>AXLE DAMAGE FACTOR</th>
<th>VEHICLE DAMAGE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWO AXLE RIGID</td>
<td>71</td>
<td>0.24</td>
<td>0.49</td>
</tr>
<tr>
<td>THREE AXLE RIGID</td>
<td>5</td>
<td>0.32</td>
<td>0.97</td>
</tr>
<tr>
<td>FOUR AXLE RIGID</td>
<td>4</td>
<td>0.40</td>
<td>1.61</td>
</tr>
<tr>
<td>THREE AXLE ARTIC</td>
<td>2</td>
<td>0.01</td>
<td>0.03</td>
</tr>
<tr>
<td>FOUR AXLE ARTIC</td>
<td>5</td>
<td>0.06</td>
<td>0.24</td>
</tr>
<tr>
<td>TWO AXLE BUS</td>
<td>4</td>
<td>0.04</td>
<td>0.08</td>
</tr>
</tbody>
</table>

Table 11.20 A381 Harberton
## Table 11.21 A379 Modbury - 24/10/79

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>NUMBER OF VEHICLES</th>
<th>AXLE DAMAGE FACTOR</th>
<th>VEHICLE DAMAGE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWO AXLE RIGID</td>
<td>33</td>
<td>0.18</td>
<td>0.35</td>
</tr>
<tr>
<td>THREE AXLE RIGID</td>
<td>1</td>
<td>0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>FOUR AXLE RIGID</td>
<td>1</td>
<td>0.25</td>
<td>1.00</td>
</tr>
<tr>
<td>TWO AXLE BUS</td>
<td>2</td>
<td>0.15</td>
<td>0.29</td>
</tr>
</tbody>
</table>

Overall vehicle damage factor = 0.36

## Table 11.22 A379 Modbury - 17/3/80

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>NUMBER OF VEHICLES</th>
<th>AXLE DAMAGE FACTOR</th>
<th>VEHICLE DAMAGE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWO AXLE RIGID</td>
<td>32</td>
<td>0.22</td>
<td>0.44</td>
</tr>
<tr>
<td>THREE AXLE RIGID</td>
<td>8</td>
<td>0.97</td>
<td>2.89</td>
</tr>
<tr>
<td>FOUR AXLE RIGID</td>
<td>2</td>
<td>1.01</td>
<td>4.06</td>
</tr>
<tr>
<td>THREE AXLE ARTIC</td>
<td>2</td>
<td>0.02</td>
<td>0.06</td>
</tr>
<tr>
<td>FOUR AXLE ARTIC</td>
<td>1</td>
<td>0.03</td>
<td>0.11</td>
</tr>
<tr>
<td>TWO AXLE BUS</td>
<td>4</td>
<td>0.04</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Overall vehicle damage factor = 0.94

## Table 11.23 A379 Modbury - 18/3/80

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>NUMBER OF VEHICLES</th>
<th>AXLE DAMAGE FACTOR</th>
<th>VEHICLE DAMAGE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWO AXLE RIGID</td>
<td>24</td>
<td>0.14</td>
<td>0.27</td>
</tr>
<tr>
<td>THREE AXLE RIGID</td>
<td>1</td>
<td>0.77</td>
<td>2.30</td>
</tr>
<tr>
<td>THREE AXLE ARTIC</td>
<td>2</td>
<td>0.13</td>
<td>0.40</td>
</tr>
<tr>
<td>FOUR AXLE ARTIC</td>
<td>3</td>
<td>0.13</td>
<td>0.53</td>
</tr>
<tr>
<td>TWO AXLE BUS</td>
<td>3</td>
<td>0.07</td>
<td>0.13</td>
</tr>
</tbody>
</table>

Overall vehicle damage factor = 0.38

## Table 11.24 A379 Modbury - 19/3/80

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>NUMBER OF VEHICLES</th>
<th>AXLE DAMAGE FACTOR</th>
<th>VEHICLE DAMAGE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWO AXLE RIGID</td>
<td>29</td>
<td>0.16</td>
<td>0.32</td>
</tr>
<tr>
<td>THREE AXLE RIGID</td>
<td>5</td>
<td>0.79</td>
<td>2.38</td>
</tr>
<tr>
<td>THREE AXLE ARTIC</td>
<td>3</td>
<td>0.42</td>
<td>1.27</td>
</tr>
<tr>
<td>TWO AXLE BUS</td>
<td>2</td>
<td>0.04</td>
<td>0.08</td>
</tr>
</tbody>
</table>

Overall vehicle damage factor = 0.65
<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>NUMBER OF VEHICLES</th>
<th>AXLE DAMAGE FACTOR</th>
<th>VEHICLE DAMAGE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWO AXLE RIGID</td>
<td>71</td>
<td>0.15</td>
<td>0.31</td>
</tr>
<tr>
<td>THREE AXLE RIGID</td>
<td>5</td>
<td>0.46</td>
<td>1.37</td>
</tr>
<tr>
<td>FOUR AXLE RIGID</td>
<td>6</td>
<td>0.22</td>
<td>0.88</td>
</tr>
<tr>
<td>THREE AXLE ARTIC</td>
<td>16</td>
<td>0.27</td>
<td>0.80</td>
</tr>
<tr>
<td>FOUR AXLE ARTIC</td>
<td>23</td>
<td>0.17</td>
<td>0.67</td>
</tr>
</tbody>
</table>

Table 11.25 A38 Plympton By-Pass

<table>
<thead>
<tr>
<th>SITE</th>
<th>NUMBER OF AXLES PER COMMERCIAL VEHICLES</th>
<th>AXLE DAMAGE FACTOR</th>
<th>VEHICLE DAMAGE FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>A381 HARBERTON</td>
<td>2.19</td>
<td>0.21</td>
<td>0.46</td>
</tr>
<tr>
<td>A397 MODBURY</td>
<td>2.27</td>
<td>0.26</td>
<td>0.59</td>
</tr>
<tr>
<td>B3210 ERMINGTON</td>
<td>2.56</td>
<td>0.43</td>
<td>1.10</td>
</tr>
<tr>
<td>B3218 OKEHAMPTON</td>
<td>2.28</td>
<td>0.33</td>
<td>0.76</td>
</tr>
<tr>
<td>UNCLASSIFIED CORNWOLD</td>
<td>3.00</td>
<td>0.84</td>
<td>2.52</td>
</tr>
<tr>
<td>UNCLASSIFIED LEE MOOR</td>
<td>2.72</td>
<td>0.74</td>
<td>2.01</td>
</tr>
<tr>
<td>A38</td>
<td>2.69</td>
<td>0.23</td>
<td>0.62</td>
</tr>
</tbody>
</table>

Table 11.26 Axle and Vehicle Damage Factors for each test site
The effect of these vehicle types on the damage factors for their particular classification can be seen; the damage factor at Ermington for two-axle rigid vehicles is nearly half that at Okehampton and the damage factor for three-axle rigid vehicles is nearly twice that at Okehampton.

The overall damage factors for each site have been calculated and are given in Table 11.26.

Figure 17 from LR910 has been reproduced as Figure 11.9 in this report and the axle damage factors for each site have been superimposed on this figure to allow a comparison to be made with the estimated values for 1979.

The two unclassified roads at Cornwood and Lee Moor yield damage factors nearly twice as high as any other road because of the high proportion of fully loaded vehicles which travel on them every day. The large number of vehicles weighed in each vehicle classification rules out the possibility of the damage factors being influenced by a few fully loaded vehicles. Their positions in Figure 11.9 relative to that estimated for roads carrying <250 vehicles reinforces the point made previously that it is vitally important to identify such 'special' roads and to carry out traffic volume and axle weight surveys.

The position of the Ermington site is probably slightly high because of the reason given previously concerning the influence of fully loaded vehicles on small weighing samples. An interesting point to note is that both A-class roads lie within the relevant band, both B-class roads within a band above the relevant one and both unclassified roads lie within a band three above the relevant one. These relative positions are possibly due to the trend line for <250 being based upon information from A-class roads with comparable axle load spectra and that it does not take into account the variability of axle load spectra on B and unclassified roads. This lack of relevant data is a good reason why use should be made of a portable weighbridge to establish realistic damage factors.

The axle load factor of 0.23 for the A38 is low for this category of road and therefore needs an explanation. As one of the main routes
Figure 11.9 The calculated axle damage factors superimposed upon the estimated changes in axle damage factors for four levels of the traffic, 1945 to 2005.

(after E.W.H. Currer and M.G.D. O'Connor)\(^{(7)}\)
into the South West the A38 carries in excess of 1000 commercial vehicles per day in each direction. This volume of traffic made it impracticable to weigh every vehicle and a sampling rate of every fifth vehicle was adopted. Problems of sticking to this sampling rate were encountered and the results indicated a bias towards the smaller, lighter commercial vehicles. An axle load factor of 0.23 is not a true reflection of the damage factor for this particular road.

11.8.4.2 The effect that the choice of day can have on the estimated value of the damage factor

In addition to the vehicle weight surveys already described, work was undertaken on a three consecutive day weight survey on the Modbury site. The main reason behind this was to determine whether, as with the traffic count, the choice of day greatly affected the damage factors obtained for each vehicle category.

The results obtained are presented in Tables 11.22 and 11.24 inclusive, and reference to them will indicate that the damage factors obtained for each classification were different for each of the three days. There was also a difference in the overall vehicle damage factors for the three days. The duration of each survey was not the same, due to a vehicle breaking down on 18th March 1980 in the lay-by and blocking off the entry to the weighbridge. This accounts for the small number of vehicles weighed on that day.

The results shown in both Tables 11.21 and 11.23 are for surveys undertaken on two separate Wednesdays, and although there is a difference between the damage factors obtained for each vehicle classification, the overall damage factors were virtually the same.

11.8.5 Improved procedure for determining Damage Factors from Traffic Counts and Portable Weighbridge Surveys

The weight survey work undertaken with the portable equipment, although based at times on small samples, has indicated the difficulty in deriving a damage factor appropriate to a wide range of roads.
classified in terms of the number of vehicles using them (i.e. <250). It has also highlighted the effect that the daily variation in numbers and types of vehicles can have on the estimated damage factor for a particular site.

One method of making an allowance for this effect is to firstly conduct an extensive traffic count over a five day period to determine the average daily number and type of vehicles in each vehicle classification. It may be possible to use the General Traffic Census data \(^{(68)}\) for this purpose.

The second step would be to undertake a one day vehicle weight survey and weigh the number of vehicles in each classification, as specified from the traffic count.

Data on the accuracy of short period weight surveys is limited but work undertaken in Sweden \(^{(69)}\) indicates that average damage factors from a one day sample are likely to be within 30 to 60 per cent of the true value.

It is felt that the damage factors derived using the proposed procedure in association with the portable weighbridge would be more accurate and that damage factors could be considered appropriate if they varied by more than 30 to 40 per cent from the published data. If they varied by less than 30 to 40 per cent from the published data they could be ignored and the published data used.

11.8.6 Calculating the 'slice in time' for each test site

The standard axle factors and the present daily flow of commercial vehicles have been calculated for each site, as detailed earlier in this section.

In order to use this information to define the 'slice in time' in terms of the cumulative number of standard axles, it was first necessary to determine a starting point in the life of the pavement; usually defined by the date on which the road was opened to traffic or the date of the last major strengthening or reconstruction.
It was realised at an early stage that the former did not apply because the majority of the test sites were undesigned and had been built up to meet the traffic needs by a series of successive strengthening measures.

It was therefore necessary to determine the date of the last major strengthening and use this as the starting point for the estimate of past traffic.

Consultations with the appropriate Divisional Surveyors indicated that records were generally only available for strengthening work undertaken over the last 20 years.

Sections which had been strengthened during this time were identified and for these sections the estimate of past traffic was based upon the date of completion of the strengthening work.

The lack of earlier records meant that a decision had to be made concerning the starting point for sections not strengthened during the last 20 years.

A start point of 1960 was assumed for these sections because it can be shown that the exclusion of traffic prior to this date from the estimate would not greatly affect the position of the 'slice in time' on the deflection and life chart.

Published data has shown that the standard axle factor has increased over the last twenty years. To take account of this in the estimates of past traffic use was made of the trend lines in Figure 18 in LR916(7) to determine the values for each year between 1980 and the year defined as the start point for estimating past traffic.

The 'slice in time' for each site was calculated in terms of the cumulative number of standard axles by summing, for each year in the past life of the pavement, the product of the standard axle factor and the commercial vehicle flow.

Past traffic has been calculated for 2%, 3% and 4% rates of growth in commercial traffic and are given in Table 11.27. Discussions with the TRRL resulted in figures appropriate to a 4% growth in commercial traffic.
<table>
<thead>
<tr>
<th>SITE</th>
<th>Description</th>
<th>4%</th>
<th>3%</th>
<th>2%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site 1 -</td>
<td>Totnes Hill Ch. 0.00 - 1.25</td>
<td>0.31 x 10^6</td>
<td>0.33 x 10^6</td>
<td>0.36 x 10^6</td>
</tr>
<tr>
<td>Site 1 -</td>
<td>Halwell to River Wash Bridge (Overlaid Section)</td>
<td>0.06 x 10^6</td>
<td>0.06 x 10^6</td>
<td>0.06 x 10^6</td>
</tr>
<tr>
<td>Site 1 -</td>
<td>Halwell to River Wash Bridge</td>
<td>0.37 x 10^6</td>
<td>0.40 x 10^6</td>
<td>0.43 x 10^6</td>
</tr>
<tr>
<td>Site 4 -</td>
<td>Ashbury Junc. to 'X' roads (Overlaid Section)</td>
<td>0.08 x 10^6</td>
<td>0.08 x 10^6</td>
<td>0.09 x 10^6</td>
</tr>
<tr>
<td>Site 4 -</td>
<td>Ashbury Junc. to 'X' roads</td>
<td>0.23 x 10^6</td>
<td>0.25 x 10^6</td>
<td>0.27 x 10^6</td>
</tr>
<tr>
<td>Site 5 -</td>
<td>Cornwood to Quarry Ch. 0.00 - 1.615</td>
<td>0.11 x 10^6</td>
<td>0.12 x 10^6</td>
<td>0.13 x 10^6</td>
</tr>
<tr>
<td>Site 5 -</td>
<td>Tolchmoor Gate to Cadover Bridge Ch. 3.0 - 7.0</td>
<td>0.27 x 10^6</td>
<td>0.29 x 10^6</td>
<td>0.32 x 10^6</td>
</tr>
</tbody>
</table>

Table 11.27 Estimated Past Traffic for each site, calculated for 2%, 3% and 4% growth rates.
traffic being adopted as the most likely for the test sites under consideration.

11.9 RELATING DEFLECTION TO THE CONDITION OF THE PAVEMENT

11.9.1 Introduction

The 'slice in time' for each site has been calculated from the data collected during the traffic surveys and expressed as a cumulative number of standard axles (details given in section 11.8.6).

It is then necessary to identify, for each slice in time, the deflection levels corresponding to pavements visually classified as being in a SOUND, CRITICAL and FAILED condition. This was expected to be achieved by a simple matching procedure between deflection and condition.

However, obtaining the required accuracy of match proved to be an important part of this investigation into the behaviour of lightly trafficked roads.

Several methods of relating deflection and condition were employed and it was found that the great variability in strength both along and across these lightly trafficked roads made it necessary to ensure that the condition of the pavement's surface was recorded at the exact point of each deflection measurement. The variations in distance which often arose as a consequence of the different methods used to locate the deflection measurements and the condition also contributed to the difficulties experienced.

As a result of time-consuming analyses of deflection and condition a procedure was developed for use with the Deflectograph that involved marking the position of the measuring tip on the pavement's surface for each deflection measurement to allow the condition to be recorded at that point.

The analysis of the deflection and condition measurements recorded using this procedure has resulted in a basic relationship being
developed between deflection and condition for lightly trafficked roads.

The slice in time approach has been used to compare the derived deflection and performance relationships with those published by TRRL.

The relationships derived from the experimental data collected during this project suggests that the behaviour of lightly trafficked roads with granular non-cementing and bituminous road-bases is similar to that presented in the charts in LR833.(35)

11.9.2 Matching Deflection and Visual Condition recorded at 10 m intervals

An attempt was made to relate individual standardised deflection measurements recorded at about 4 m intervals to the condition of the pavement's surface recorded at 10 m intervals. Previous work on designed roads had shown that a condition survey every 10 m was generally sufficient to reflect the overall condition of the pavement in relation to individual deflection measurements. This was not the case for the undesigned roads being investigated. It was impossible to discern a relationship between deflection and condition; similar deflection ranges were identified for all three (SOUND, CRITICAL and FAILED) pavement classifications.

The frequency of the condition measurements did not adequately reflect the rate of change of strength of the pavement and it was therefore decided to record conditions at 1 m intervals on reduced lengths of each site.

It was felt that the methods used to record the position of each deflection measurement and condition of the pavement's surface also contributed to the problem of matching these two parameters.

11.9.3 Matching Deflection and Visual Condition recorded at 1 m intervals

One metre visual condition surveys were recorded on the lengths of
road detailed in Table 11.28

<table>
<thead>
<tr>
<th>SITE</th>
<th>CHAINAGE</th>
<th>LENGTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site 1 - Totnes Hill</td>
<td>500 - 1000</td>
<td>500 m</td>
</tr>
<tr>
<td>Site 4 - Ashbury Junction to 'X' roads</td>
<td>3750 - 4250</td>
<td>500 m</td>
</tr>
<tr>
<td>Site 5 - Cornwood to Quarry</td>
<td>250 - 750</td>
<td>500 m</td>
</tr>
<tr>
<td>Site 5 - Tolchmoor Gate to Cadover Bridge</td>
<td>3000 - 3100</td>
<td>100 m</td>
</tr>
<tr>
<td>Site 5 - Tolchmoor Gate to Cadover Bridge</td>
<td>3300 - 3500</td>
<td>200 m</td>
</tr>
<tr>
<td>Site 5 - Tolchmoor Gate to Cadover Bridge</td>
<td>6600 - 6700</td>
<td>100 m</td>
</tr>
</tbody>
</table>

Table 11.28 Lengths visually assessed at 1 m intervals.

These lengths of road were selected from the 10 m visual condition survey sheets to include fairly long sections of each of the three surface condition classifications.

Because of the inadequacy of the fifth wheel as an accurate distance measuring device it was decided to determine the average spacing between the individual deflection measurements. This was done by dividing the distance between two chainage markers by the number of deflection readings within that distance.

11.9.3.1 Deflection Profiles

The deflection data for a particular length of road was presented as a deflection profile obtained by plotting each deflection against the chainage and joining the individual points with straight lines, Figure 11.10. Although this provided a continuous deflection profile for the length of road only the individual deflection points representing the actual measured values were compared to condition.

Deflection data from five Deflectograph surveys, representing a time
Figure 11.10 Deflection Profile
scale of about two years, was considered as being applicable to the same slice in time because most of the test sites were so lightly trafficked.

For analysis the deflection profiles for each survey were reproduced on tracing paper in order that they could be superimposed on top of one another.

11.9.3.2 Visual Condition Data

The information from the visual condition survey sheets was transformed onto graph paper with each pavement surface classification being represented by a different symbol, Figure 11.11. Presenting the data in this manner allowed the deflection profiles to be positioned on top of these condition sheets to determine the deflection values associated with each pavement condition classification.

11.9.3.3 Relating Deflection and Condition

The deflection profiles were placed on top of the condition survey sheets one at a time and moved a maximum of one deflection reading to the left or the right to find the most probable match with condition. Plus or minus one deflection reading was regarded as the greatest accuracy of interpretation of the results from the Deflectograph. This process was repeated for the remaining four other deflection profiles applicable to each particular length of road. Once having determined their most probable positions relative to the condition sheet the deflection profiles were superimposed on top of one another for comparison with condition.

Superimposing the deflection profiles in this way gave an indication of the general deflection levels associated with each particular length of road.

A general indication of deflection level was the best that could be hoped for because the position of the measuring beam on the road's surface was probably not the same from survey to survey.
Figure 11.11 Chart showing the method used to present the visual condition classification.
The deflection values associated with each pavement surface classification were determined from this method of matching the deflection profile and the condition and typical results are shown in Figures 11.12 and 11.13 for the test sites at Okehampton and Tolchmoor Gate. Similar relationships between deflection and condition were derived for the other sites.

The position of the critical condition curve for the appropriate slice in time was extracted from the relevant deflection performance chart in LR833 (35) and superimposed on Figures 11.12 and 11.13 for comparison purposes.

The results shown in Figures 11.12 and 11.13 are typical for all of the test sites and indicate similar deflection ranges for sound, critical and failed pavements.

As a consequence of the low deflection levels corresponding to critical conditions, the position of the critical condition curve, suggested by these results, was lower than indicated in LR833 (35).

It would appear from the results of the comparison between deflection and 1 metre condition survey data that the condition specific to each deflection value was not identified or that the deflection approach does not apply to lightly trafficked roads. The probable reason for this was that the chainage calculated for the deflection readings was different to that recorded during the visual condition survey. A difference in chainage of one or two metres, together with the great variation in deflection levels apparent on these undesigned roads, could result in the deflection and condition relationships suggested by Figures 11.12 and 11.13.

To try and solve this problem it was decided to record the visual condition at the exact point of each deflection measurement.

11.9.4 Use of the Deflection Beam to relate Deflection to Condition at the exact point of measurement

The deflection beam was used to measure deflections on sections of
Figure 11.12 Deflection levels corresponding to the SOUND and CRITICAL visual condition classification.
Figure 11.13 Deflection levels corresponding to the SOUND, CRITICAL and FAILED visual condition classifications.
Site 5B, Tolchmoor Gate to Cadover Bridge, and Site 1A, Totnes Hill. The visual condition of the pavement's surface at every measurement point was noted. Figures 11.14 & 11.15 show the deflection levels for pavements in a sound and critical condition.

Deflections were measured at a road temperature of about 4°C and were not temperature corrected because of the inaccuracies involved when correcting deflections measured at such low temperatures.

The standard deflection levels associated with the two sites were not therefore reflected in these results.

In both cases it can be seen that the deflection values associated with the critical conditions were higher than those associated with sound conditions, i.e. the more damaged and weaker the pavement section, the higher the deflection. This work with the Deflection Beam, although based upon a small sample and undertaken at very low pavement temperatures, has indicated that a relationship can be developed between deflection and condition for lightly trafficked roads provided that the two parameters are measured at the same position on the pavement's surface.

11.9.5 Matching Deflectograph Deflection and Visual Condition

Having used the Deflection Beam to show the importance of recording condition at the exact point of each deflection measurement, it was necessary to devise a method of marking the position of the measuring tip during subsequent work with the Deflectograph.

Various techniques were tried and the one most easily implemented was a hand held aerosol paint spray. A Deflectograph survey of each test site was undertaken using this modified operating procedure with great success. The paint marks at specific intervals along each test section were easily identifiable, allowing the condition of the pavement's surface to be assessed at these points by a person walking behind the Deflectograph.

Matching deflection and condition information, gathered in this way,
SITE 1A - TOTNES HILL

Visual Condition Classification

S1 | S2 | C3

Deflection (x 10^-2 mm)

180
160
140
120
100
80
60
40
20

Figure 11.14 Deflection Beam deflections corresponding to the SOUND and CRITICAL visual classifications.

SITE 5B - CADOVER BRIDGE

Visual Condition Classification

S1 | S2 | C4

Deflection (x 10^-2 mm)

180
160
140
120
100
80
60
40
20

Figure 11.15 Deflection Beam deflections corresponding to the SOUND and CRITICAL visual classifications.
has resulted in the identification of the deflection levels for the various pavement condition classifications, Figures 11.16,11.17 & 11.18. Reference to these Figures will show that the general deflection level associated with a failed pavement is greater than that associated with a critical pavement, which in turn is greater than that associated with a sound pavement.

Similar results were obtained for the other sites surveyed using this modified procedure.

Figures 11.16, 11.17, 11.18 indicate a basic relationship between deflection and condition, for lightly trafficked roads, the trend of which is similar to that derived for medium and heavily trafficked roads.

11.9.6 Using the Critical Deflection Range to locate the Position of the Critical Condition Curve

Relationships between deflection and condition have been determined for each test site indicating the deflection ranges for lightly trafficked pavements in a Sound, Critical and Failed condition. Combining this information with the past traffic levels for each site allowed the behaviour of lightly trafficked roads to be expressed using the slice in time approach.

11.9.6.1 Comparing Derived Relationships with those previously published

The behaviour of lightly trafficked roads was related to the published data by extracting the deflection ranges corresponding to critical conditions from figures 11.16 to 11.18 superimposing them for each slice in time on Figures 11.19 and 11.20.

Figures 11.19 and 11.20 show the relationship between critical life deflection and critical life for pavements with granular non-cementing and bituminous roadbases, respectively.

All of the sites investigated, with the exception of the one at Totnes, had granular non-cementing roadbases. The test site in Totnes had a
Figure 11.16 Deflection levels corresponding to the SOUND, CRITICAL and FAILED visual classifications.
Figure 11.17 Deflection levels corresponding to the SOUND, CRITICAL and FAILED visual classifications.
Figure 11.18 Deflection levels corresponding to the SOUND and CRITICAL visual classifications.
Figure 11.19 Critical deflection range for the test sites superimposed upon the relation between critical life deflection and critical life for pavements with non-cementing granular road bases.
Figure 11.20 Critical deflection range for the test site at Totnes superimposed upon the relation between critical life deflection and critical life for pavements with bituminous road bases.
bituminous road-base.

The deflection and performance relationships were derived by the TRRL from work undertaken on their full-scale pavement design experiments and were originally used to help define the position of the critical curve in the charts given in LR833. The extrapolated region (shown dotted) below one million cumulative standard axles represents the lightly trafficked roads being investigated by this project.

The following points can be drawn from Figures 11.19 and 11.20:

1. The range of critical deflections for the sites at Okehampton and Tolchmoor Gate are similar to those suggested by the TRRL data but have substantially higher and lower limiting values.

2. The range of critical deflections for the sites at Modbury and Lee Moor are below those suggested by the TRRL data.

3. The upper limiting value representing critical conditions for the Totnes site with a bituminous road-base lies within the range suggested by the TRRL data, but the lower limiting value lies outside this range.

In general the relative positions of the critical deflection ranges would suggest that the position of the critical condition curve for lightly trafficked roads lies below that indicated by the TRRL data.

As a possible explanation for the low deflection levels corresponding to pavements in a critical or failed condition consider the damage criteria used to define the conditions of the pavement's surface.

Deterioration shows itself as rutting or cracking of the pavement's surface and it is the magnitude of the damage which defines pavement condition.

A problem often encountered while trying to classify the condition of these undesigned lightly trafficked roads was whether the shape of the pavement's surface was due to general unevenness or genuine
rutting. The unevenness encountered extended both along and across the pavement sections. It is quite possible that at times the unevenness of the pavement's surface could have been mistaken for rutting, with the result that a relatively strong pavement was classified as being in a critical or failed condition. The strength of the pavement would be reflected in the magnitude of the deflection measurements and in this case low deflection values would be linked to pavements classified as being in critical or failed condition.

This problem was further complicated by the liberal use of surface dressings on lightly trafficked roads. Surface dressings can not only conceal any cracking of the pavement's surface but can at times give the impression of a rut due to the size and distribution of the stones.

Considerable care was required to ensure that the surface deformation was not due to the effect of general unevenness and/or surface dressing.

Non-structural deterioration in the form of small ruts in the surfacing only could result in the pavement condition being classified as critical and, when used to qualify the deflection, would indicate an artificially low deflection level associated with critical conditions.

11.9.7 Using the Top of the SOUND Deflection Range to locate the Position of the Critical Condition Curve

To overcome the possible influence of surface characteristics (unevenness, surface dressing) and non-structural deterioration on the criteria used to define pavement condition, attention was turned to an analysis of the deflection measurements recorded on pavements in a SOUND condition. It was felt that an approach based upon the deflection levels at the top of the sound range would give a more reliable indication of the position of the critical condition curve because these values were less likely to be influenced by the damage criteria used to classify pavement condition.

Deflection levels at the top of the sound range represent pavement sections beginning to show signs of deterioration, i.e. approaching critical conditions.
The deflection ranges appropriate to pavements in a sound condition have been extracted from Figures 11.16, 11.17 and 11.18 and superimposed for each slice in time on Figures 11.21 and 11.22.

Reference to Figures 11.21 and 11.22 will show that in all cases, apart from one, the top of the sound deflection range lies within the band of critical deflections suggested by the TRRL data. These relative positions of the limiting deflection values would indicate that the behaviour of lightly trafficked roads with granular non-cementing and bituminous road-bases is similar to that indicated by the deflection and performance relationships published in LR833. (35)

11.10 DISCUSSION

The aim of this work was to develop relationships between deflection and performance for lightly trafficked roads and use these relationships to validate the position of the critical condition curves given in the deflection and life charts in LR833. (35)

The project covered a three-year study, which was insufficient time to develop full deflection histories relating deflection and performance for the pavement sections considered.

A new methodology had therefore to be developed for investigating the behaviour of flexible pavements based upon data gathered over a limited time scale.

The method adopted allows the deflection and performance relationships to be considered as a slice in time from their complete deflection histories.

To use this approach it was necessary to determine the deflection ranges corresponding to pavements in a SOUND, CRITICAL and FAILED condition by relating deflection to the condition of the pavement's surface defined in terms of the depth of rutting and extent of cracking in the wheel-paths.

A feature of this work on lightly trafficked roads was the need to match
Figure 11.21 Sound deflection range for the test sites superimposed upon the relation between critical life deflection and critical life for pavements with non-cementing granular road bases.
Figure 11.22 Sound deflection range for the test site at Totnes superimposed upon the relation between critical life deflection and critical life for pavements with bituminous road bases.
deflection to the condition at the exact point of measurement. This was only necessary to develop the basic relationship between deflection and condition with sufficient accuracy to define the position of the critical condition curve. Such accuracy of match between deflection and condition is not necessary for routine deflection surveys.

The slice in time was defined by the cumulative number of standard axles carried by each test site since last major strengthening.

The critical condition curves presented in the charts in LR833 for each road-base type cover a band of deflection levels and it was initially thought that the accuracy of the position of these curves could be assessed by superimposing the deflection and performance relationships for pavements in a critical condition onto the appropriate chart.

This was achieved using the slice in time approach, with the result that the critical deflection range either covered a similar range to that suggested by the TRRL data, but with substantially higher and lower limiting values, or covered a range below that suggested by the TRRL data.

The relative positions of these critical deflection ranges indicated that the critical condition curve, for pavements with granular non-cementing and bituminous road-bases, lay below that presented in the relevant charts in LR833 (35).

It was felt that the low deflections corresponding to pavements in a critical condition may be a consequence of either

(i) surface unevenness and/or loss of chippings from surface dressing

or

(ii) non-structural deterioration

on the criteria used to visually define pavement condition.

To keep any such influence to a minimum, attention was turned to the use of the deflection level corresponding to the top of the SOUND
deflection range as the basis for the estimate of the position of the critical condition curve. The deflection levels corresponding to the top of the SOUND deflection range represents pavements beginning to show signs of visual deterioration and whose condition is also reflected in the magnitude of the deflection measurement.

Deflection levels higher than these on pavements also showing signs of visual deterioration must therefore represent pavements in a critical condition.

The deflection ranges appropriate to pavements in a SOUND condition were superimposed, for each slice in time, on the chart showing the deflection and performance relationship for the relevant road-base type. In all cases, apart from one, the deflection level corresponding to the top of the SOUND deflection range fell within the band of deflections suggested by the published data. The relative positions of these limiting deflection values would indicate that the position of the critical condition curve is within the range shown by the curves in the published data.(35)

The deflection and performance relationships derived from the experimental investigations suggests that the behaviour of lightly trafficked roads with granular non-cementing and bituminous road-bases is similar to that presented in the relevant charts in LR833.(35)

11.11 CONCLUSIONS

The work carried out has resulted in:-

1. the development of a new methodology for investigating the behaviour of flexible pavements based upon data gathered over a limited time scale.

This approach has further applications in that it can be used to 'calibrate' the existing evaluation and strengthening design method for use in other countries with different climates. Calibration could be achieved by developing deflection and

355
performance relationships over a limited period of time and using the slice in time approach to compare an estimate of the position of the critical condition curve based on these relationships with the position suggested in the method. Excessive differences in the position of the critical condition curve would require an adjustment to be made to any estimate of remaining life.

2. the establishment of deflection and performance relationships for lightly trafficked roads and the subsequent validation of the relationships given in the charts in LR833 describing the behaviour of lightly trafficked roads.

3. the setting up of a large experimental data base containing information on pavement condition, construction and strength, including curvature, maximum deflection, visual deterioration, in-situ moduli, pavement thicknesses and strengths.

The extension of the existing evaluation and strengthening design method for use on lightly trafficked roads is dealt with in the next chapter.
12 EXTENSION OF THE FLEXIBLE PAVEMENT EVALUATION AND OVERLAY DESIGN PROCEDURE TO INCLUDE THE BEHAVIOURAL DATA GATHERED ON THE LIGHTLY TRAFFICKED TEST SECTIONS

12.1 INTRODUCTION

This chapter contains the extended deflection and performance and overlay design charts for pavements with granular non-cementing and bituminous roadbases.

Details are given of the methods used to extend the existing relationships to include the behavioural data gathered on the lightly trafficked test sections.

12.2 DEFLECTION AND PERFORMANCE RELATIONSHIPS

The experimental data collected during this project has allowed the definition of critical deflection and critical life relationships for lightly trafficked roads with granular non-cementing and bituminous roadbases. These relationships have been shown to be similar to those suggested by the data collected on the TRRL's full-scale design experiments (Figures 11.21, 11.22).

The critical life deflection and critical life relationships were used, together with relationships between early life deflection and critical life, to construct the deflection and performance charts given in LR83. The deflection and performance relationships for pavements with granular non-cementing roadbases is given in Figure 12.1. This chart can be used, together with measurements of maximum deflection and estimates of past traffic, to predict the remaining life in a pavement.

The lines, shown in this chart, defining critical conditions were extrapolated for the lower traffic levels and were shown as dashed lines.

On the basis of the evidence presented in Figures 11.21 and 11.22 it is now felt that these lines can be shown as solid down to a traffic
Fig. 12.1 Relation between standard deflection and life for pavements with non-cementing granular road bases. (after Kennedy and Lister)
level of $0.2 \times 10^6$ cumulative standard axles, for pavements with granular non-cementing and bituminous roadbases.

The solid lines are indicative of the increased confidence in the ability of the relationships shown in these charts to accurately represent the likely behaviour of lightly trafficked roads.

The deflection and performance relationships for pavements with granular non-cementing and bituminous roadbases are presented in Figures 12.2 and 12.3 respectively.

The reduced time scale of this project has meant that it was not possible to construct full deflection histories for the lightly trafficked test sections. This means that both the shape and the position of the deflection trend lines shown in the original charts for lightly trafficked roads must be assumed correct because of the lack of contrary evidence.

The 'fanning out' of the critical condition lines to represent a range of probabilities of achieving a given life is also assumed correct because their relative positions could not be validated using the information collected during this investigation.
Figure 12.2 Deflection and life relationships for pavements with granular non-cementing roadbases.
Figure 12.3 Deflection and life relationships for pavements with bituminous roadbases.

Cumulative Standard Axles (x 10^6)

Probability of achieving life
- 0.25
- 0.50
- 0.75
- 0.90

Critical Condition Curves

Standard Deflection (x 10^-2 m)
12.3 OVERLAY DESIGN CHARTS

The investigations undertaken on the lightly trafficked test sections has shown that deflection levels in excess of $120 \times 10^{-2}$ mm can be recorded on pavements in a sound condition.

It follows that if the residual life of these roads is less than that required, the overlay design charts presented in LR833 should be capable of being used to design overlays to strengthen them.

However, in the majority of cases the overlay design charts can only be used to design overlays for pavements with present deflection levels up to about $120 \times 10^{-2}$ mm. It was therefore necessary to extend the overlay design charts to accommodate present deflection levels in excess of this value.

The overlay design charts were extended using:

(i) the overlay reduction chart, Figure 12.4;

(ii) the early life deflection and critical life relationships for overlaid pavements. (An example is given in Figure 12.5 for pavements with granular roadbases).

These relationships were derived by TRRL from performance data gathered on their full-scale experiments.

The extended overlay design charts are presented in Figures 12.6 and 12.7 for pavements with granular non-cementing roadbases, and Figures 12.8 and 12.9 for pavements with bituminous roadbases.
Figure 12.4 The reduction in deflection achieved by rolled asphalt overlays of different thickness.
Figure 12.5 Relation between early life deflection and critical life for pavements with granular bases overlaid with rolled asphalt.
Figure 12.6 Overlay design chart for pavements with non-cementing granular road bases (0.50 probability)
Figure 12.7 Overlay design chart for pavements with non-cementing granular road bases (0.90 probability)
Figure 12.8 Overlay design chart for pavements with bituminous road bases (0.50 probability)
Figure 12.9 Overlay design charts for pavements with bituminous road bases (0.90 probability)
REFERENCES


18. SALT, G.T. and SZATKOWSKI, W.S. 'A guide to levels of skidding resistance for roads'. Transport and Road Research Laboratory, LR510, Crowthorne.


24. LISTER, N.W. 'A Deflection Beam for investigating the behaviour of pavements under load', Road Research Laboratory Note RN/3842/NWL, 1960 (Unpublished).


27. BARENHOLT, E. 'Bearing capacity investigation by Deflectograph', Fifteenth Permanent Int. Ass. of Road Congresses, World Road Congress, Mexico City, 1975.


33. Washington State Highways Commission. 'Pavement deflection and pavement overlay criteria, Research Project, Y - 1202, Phase 1, Dept. of Highways, USA.


44. DEHLEN, G.L. 'A simple instrument for measuring the curvature induced in a road surfacing by a wheel load'. Civil Engineer in South Africa, Vol. 4, No. 9, Sept. 1962, p 182-194.


46. RUSSAM, K. and BAKER, A.B. 'Using a curvature meter to measure transient deflections of road surfaces'. Civil Engineering, Nov. 1964, 59, (700), 1427.


50. ODEMARK, N. 'Investigations as to the elastic properties of soils and the design of pavements according to the theory of elasticity', Statens Vaginst, Stockholm Medd 77, 1949.


52. MAJIDZADEH, K. 'Dynamic deflection study for pavement condition investigation'. Ohio Dept. of Transportation, Columbus, Ohio, June 1974.

53. Pafec Ltd. 'Pafec 75, theory and results', Nottingham, England.

54. KENNEDY, C.K. et al. IN PRIVATE.


63. PAGE, J. and MAYHEW, H.C. IN PRIVATE.


74. O'FLAHERTY, C.A. 'Highways and traffic - Volume 1', Edward Arnold, Chapter 2, pages 50-83.
I.C. Butler, Research Fellow, Department of Civil Engineering, Plymouth Polytechnic.
C.K. Kennedy, Head of Department of Civil Engineering, Plymouth Polytechnic, United Kingdom.

Abstract

Measurements of maximum deflection recorded by a deflection beam or a Deflectograph have been used as the basic input for assessing pavement performance and overlay design. The methods also require information on construction and at present this is obtained by coring and from trial pits. The duality of information required by design methods has limited the introduction of real time processing and, as a result, reduced the basic advantage of having closely spaced measurements by the time delay created between survey and design recommendation.

This paper describes investigations into the use of deflected shape measurements to characterise the pavement construction. Details are presented of theoretical studies, using a finite element approach, together with in situ measurements made with a Deflectograph on a wide range of pavement types.

Resulting from these investigations a method of estimating the thickness of bound material in a pavement structure is presented.

The accuracy of the proposed method is assessed in relation to the quality of the measurements, the existence of localised 'soft spots' in the soil subgrade and granular layers of the pavement and localised cracking of the bound layers.

The Use of Maximum Deflection and Deflected Shape Measurements for Assessing Pavement Performance

1.0 Introduction

Measurements of maximum deflection recorded by a Deflection Beam or a Deflectograph are a basic input for assessing pavement performance and overlay design\(^{(1)}\). The methods also require information on construction and this is usually obtained by coring and from trial pits.

At present most Deflectographs measure and record only the maximum deflection response of a pavement. The deflection measurements collected are subsequently processed using computers that are normal remote from the road being surveyed.

A more suitable approach would be for the deflection measurements to be processed on the machine as they are recorded to allow immediate identification of suspect areas. Provision of real-time analysis would encourage closer contact between those taking the measurements and those responsible for making decisions on the basis of the measurements.

A requirement of an on board processing system would be a continuous assessment of the thickness of the bituminous layer of the pavement being surveyed.

An indication of the thickness of the bituminous layer is important because bitumen is temperature susceptible and the degree of temperature susceptibility is a function of the proportion of the total pavement stiffness contributed by the bound layer.

The TRRL structural maintenance design method\(^{(2)}\) incorporates a number of temperature and deflection relationships for pavements of various bituminous layer thicknesses, which can be used to standardise deflection measurements obtained at any pavement temperature within the range 5-30°C to equivalent values at 20°C. A knowledge of the thickness of the bituminous layer means that the appropriate relationship can be selected.

This paper describes investigations into the use of deflected shape measurements to characterise the pavement construction. Deflected shape is defined in terms of a differential deflection at a specific distance from the position of maximum deflection. The differential deflection at a particular point on the deflection dish is calculated as the difference between the maximum deflection and the ordinate deflection at that point. Details are presented of theoretical studies, using a finite element approach, together with practical studies using in-situ measurements of maximum deflection and deflected shape made with a Deflectograph on a wide range of
pavement types.

Resulting from these investigations a method of estimating the thickness of bound material in a pavement structure is presented.

The accuracy of the proposed method is assessed in relation to the quality of the measurements, the existence of localised soft spots in the granular layers and soil subgrade and localised cracking of the bound layers.

2.0 The Pavement Model

The finite element model developed represents a longitudinal pavement section, 12 metres in length, loaded at its mid-point by a load equal to that applied to the road surface by each dual wheel assembly of the Deflectograph. A distance of 12 metres was selected because it approximates to the length of road influenced by the action of the loaded wheels on all but the stiffest of pavement structures.

The symmetry of the model means that it can be sub-divided at the point of application of the load and subsequent analyses undertaken on half of the original model.

Physically reducing the size of the model, results in a considerable saving in the computer time required to complete the analysis.

The model, Figure 1, was sub-divided into a number of rectangular elements connected at their nodal points. The nature of the finite element idealisation means that, in general, the accuracy of the solution increases with the number of elements taken \(^{(3)}\).

A graded division into elements was adopted to allow a more detailed study to be made in the region under the load.

Such a selective distribution of elements is efficient and can lead to economy in solution time without any loss of accuracy.

Boundary conditions were such that the bottom of the model was constrained from moving in both the horizontal and vertical directions, the sides constrained in the horizontal direction only and the surface of the model free to move in both directions.

The external load acting on the actual pavement structure was replaced by an equivalent system of forces acting at the element nodes. Care was taken, with the selection of the finite element mesh, to ensure that nodes occurred at the points of application of the forces.

2.1 Calibration of the Model

Before the finite element model could be used to investigate the ability of deflected shape measurements to characterise the pavement construction, it was first necessary to ensure that the response of the theoretical model was similar to the response of an actual
Initially it was envisaged that this would be achieved by reproducing the response of a pavement structure as measured by a Deflectograph.

However, experimental and theoretical studies have demonstrated that the deflections measured under a rolling wheel, either by a Deflection Beam or a Deflectograph, cannot be readily depicted by multi layer elastic theory and that the inclusion of non-linear behaviour does not resolve the difference.

To overcome these problems absolute deflections measured with displacement gauges on to pavement structures, one with a granular roadbase, and one with a bituminous bound roadbase were obtained from instrumented road sections. These measurements defined the pavement response to be modelled. Of the parameters controlling the response of each layer, stiffness, \( E \), poisson's ratio, \( v \) and thickness, \( h \), to applied load only the bound layer, the granular roadbase and the granular subbase thicknesses were known for the structures investigated.

In developing the model, values for the elastic parameters (\( E \) and \( v \)) of the individual layers and the subgrade were initially deduced from typical published values for each material type. These values were subsequently modified within closely defined limits until the response of the theoretical model was similar to that of the actual pavement structure.

A theoretical pavement model has been developed that accurately predicts the response under load for a pavement containing a granular roadbase, Figure 2, and using similar values for layer stiffness and poisson's ratio, also for an alternative structure with a bituminous roadbase.

2.2 Prediction of Bituminous Layer Thickness

The model has been used to determine the relationship between the deflected shape of the pavement's surface and thickness of the bituminous layer. This relatively simple relationship between deflected shape and bituminous layer thickness could have formed the basis of a bituminous layer thickness estimation procedure if it could have been assumed that the strengths and thicknesses of the other pavement layers and subgrade remained constant along a length of road.

However, variations in these values do occur in practice and it was therefore necessary to quantify the possible effect of each likely change on the deflected shape.

Analysis of the deflected shapes produced by the model has resulted in a number of charts showing the relationship between
deflected shape, defined as the differential deflection at various distances from the position of maximum deflection, and the maximum deflection for pavements of various thicknesses of bituminous layer.

Differential deflections were calculated as the difference between the maximum deflection and ordinate deflections measured at specific distances from the position of the maximum deflection. These charts enabled the position of the differential deflection most influenced by a change in thickness of bituminous layer to be identified and this was found to correspond to an offset 200 mm from the point of maximum deflection. Figure 3 shows the chart for an offset of 200 mm and demonstrates and quantifies the influence of changes in the stiffness of the bituminous layer and resilient moduli of the granular roadbase and subgrade, on the deflected shape.

The slopes of these lines indicate that the stiffness of the bituminous layer has a greater influence on the deflected shape (differential deflection) than on the maximum deflection, whereas the opposite is true for the modulus of the subgrade.

The strong influence of bound layer stiffness on differential deflection is not important because equivalent thickness at a single standard stiffness can be derived i.e. 100mm of a bituminous layer with a stiffness of $5.0 \times 10^9 \text{ N/m}^2$ is equivalent to (say) 80mm of material with a stiffness of $1 \times 10^{10} \text{ N/m}^2$. The influence of thickness and stiffness of the granular layer and stiffness of the subgrade on differential deflection is less strong but this will affect any estimates of bound layer thickness that are made on the basis of differential deflection. Quantification of any errors that may be introduced in the estimates as a result of changes in the properties of the granular and soil layers are considered in the next section.

Charts similar to that presented in Figure 3 could be used in conjunction with measurements of deflected shape, as recorded by a Deflectograph and corrected to absolute values, to estimate the thickness of the bituminous layer in the structure tested.

2.3 The Accuracy of the Estimate of Layer Thickness

The accuracy of the proposed method was assessed by quantifying the effect of change in either the strength or thickness of each of the pavement layers and subgrade on the estimate of bituminous layer thickness.

Charts were constructed for this purpose and a typical one, showing the effect of roadbase modulus, is presented in Figure 4. The relationship shown suggests that increasing the modulus of the roadbase by a third would result only in about a 10 per cent increase in the estimate of the thickness of the bituminous layers. A
substantial reduction in roadbase modulus, of about 66 per cent, results in about a 15 to 30 per cent reduction in the estimate of bituminous layer thicknesses.

In general, for maximum typical ranges of layer moduli and thicknesses, the bituminous layer thickness can be estimated to within ± 10-15 per cent.

The effect of soft spots in the granular subbase and cracking in the bound layers has also been studied using the model.

The magnitude of errors introduced in the estimate of bituminous layer thickness by soft spots in the granular subbase is dependent upon the size and position of the soft spot relative to the differential deflection used to define deflected shape. Small weak areas, CBR less than 1 per cent, up to 500mm long result in only small errors in the estimate of bound layer thickness and can be disregarded.

The effect of cracking in a bound layer was investigated using the model for a pavement with a bituminous roadbase. Individual vertical cracks were introduced, at various locations, either in the top or bottom half of the bituminous roadbase.

A single vertical crack resulted in an underestimate of bituminous layer thickness; the magnitude of the underestimate was generally less than 10 per cent and was again largely dependent upon the position of the crack relative to the differential deflection used to define deflected shape.

3.0 Site Investigations

In situ measurements were made with a Deflectograph on a wide range of pavement types and used to investigate a relationship between the deflected shape and thickness of the bituminous layer.

A Deflectograph recorded the response of the pavement structure as a maximum deflection and at ten pre-selected ordinate deflections.

3.1 Construction thicknesses

Prior to the analysis all deflected shape measurements were temperature corrected to equivalent values at 20°C using previously derived relationships between deflection and temperature.(2). The temperature correction procedures significantly affect the magnitude of the maximum deflection but modify the measured shape of the deflection dish only by small amounts.

Bound layer thicknesses were determined primarily using conventional coring equipment. Additional information was obtained using the non destructive wave propagation technique at a single frequency to give a continuous estimate of bound layer thickness. Comparison of the results, with cores extracted at specific points along the section of road, suggested that the single frequency technique was capable of predicting bound layer thickness to within ± 10%.
3.2 Deflected shape measurements

The in-situ deflected shape measurements were analysed using linear regression techniques to develop relationships between deflected shape, expressed in terms of the differential deflection 200mm from the point of maximum deflection, and the maximum deflection. The results obtained covered a range of pavement types which in general were substantially different from those used to develop the model. The differential deflection 200mm from the point of maximum deflection was used in the analysis because the results from the finite element model indicated this would be the offset most influenced by change in the thickness of the bituminous layer.

The relationships derived, Figure 5, are similar to those derived theoretically, but their numerical values differ from those predicted probably due to the non absolute nature of the measurements recorded by the Deflectograph, material variability in the constituent layers of the pavement and the variations in the layer thicknesses as suggested by the wave propagation results.

Further work is continuing to determine the pavement response predicted by the finite element model for pavements with similar structures to those tested in-situ. Additional work on instrumented road structures is required to validate the estimation procedure over a wider range of pavement types. However, in the interim deflection measurements recorded by a Deflectograph can be used in conjunction with Figure 5 to provide the first estimate of bound layer thickness.

4.0 Conclusions

Deflectograph measurements of maximum deflection and deflected shape can be used to estimate the thickness of bituminous bound layers. A design chart is presented for this purpose based on data collected from in-service pavements.

A finite element model has been developed that can accurately predict the maximum deflection and deflected shape response of real pavements under load.

This model has been used to investigate the accuracy of estimating bound layer thickness from deflected shape measurements and indicates that for typical ranges of layer stiffness the estimates obtained will normally be within ±10-15 per cent of the true value.

5.0 Acknowledgements

Work at Plymouth Polytechnic on the development of the deflection method for structural maintenance is supported by the Transport and Road Research Laboratory but any views expressed are not necessarily those of the Transport and Road Research Laboratory or of the Department of the Environment or of the Department of Transport.
6.0 References


Figure 1. Finite element mesh used for analysis of pavement structures.
Figure 2. Comparison of the actual and theoretical response of a pavement with a granular roadbase.

Figure 3. The influence of change in the stiffness of the bituminous layer and resilient moduli of the granular roadbase and subgrade on the deflected shape of pavement structures with 50, 100, 150, and 300 mm of bituminous material.
Figure 4. Chart for assessing the effect of change in roadway modulus on the estimate of bituminous layer thickness.

Figure 5. Chart showing relationship between differential deflection and maximum deflection derived from actual deflection shape measurements recorded on pavement structure with 40mm and 100mm of bituminous material.
The Calibration of an Existing Design Method for New Conditions

C K Kennedy  Head of Department  Department of Civil Engineering
R Pick  Research Assistant  Plymouth Polytechnic
I C Butler  Research Fellow  England
B W Ferne  Project Officer  Transport and Road Research Laboratory


crithorne, England.

Abstract

Effective use of the large sums of money now being spent worldwide in strengthening roads requires a design system capable of matching spending to needs; it should establish priorities for work and also the nature and extent of strengthening on the roads selected for treatment. A number of complete design systems are now available that satisfy these requirements but which, in general, have been developed within specific design situations.

This paper describes a procedure for calibrating existing methods to suit local conditions and existing local maintenance standards. The procedure was developed as part of a research project aimed at validating the use of the TRRL method on lightly trafficked roads in the United Kingdom. Some examples of the data collected and assessment procedures used on the project are given. A more general calibration procedure is then described that can, within a period of one to two years, effectively assess the accuracy with which an existing well-proven maintenance design method might be applied to a new set of operating conditions.

1. Introduction

Effective use of the large sums of money now being spent worldwide in strengthening roads requires a design system capable of matching spending to needs; it should establish priorities for work and also the nature and extent of strengthening on the roads selected for treatment.

A number of complete design systems are now available that satisfy these requirements but which, in general, have been developed to satisfy specific design situations. These methods are the result of many years of research effort and are based on either

(i) empirical performance relations and design criteria developed from performance studies on in-service pavements
or (ii) analytical studies of pavement behaviour combined with empirical design criteria developed from in-service pavements and laboratory investigations.

The current design methods therefore, have a limited range of direct application because of the input of empirical data covering a necessarily restricted range of environmental conditions and material types. Potential users must decide on the basis of very limited evidence which method is most likely to satisfy their requirements. The choice is usually based on a subjective assessment followed by a long process of validation and modification to provide a working system.

This paper describes a procedure for calibrating existing methods to suit local conditions and existing local maintenance standards. The procedure is related specifically to the Transport and Road Research Laboratory (TRRL) design method (1) (2) (3) but may be appropriate for use in conjunction with other methods.

The procedure was developed as part of a research project aimed at validating the use of the TRRL method on lightly trafficked roads in the United Kingdom. The roads investigated were largely undesigned in any formal sense, being built-up through successive maintenance over many years.

Examples of data collected during the project are presented. Details of the assessment procedure are described together with estimates of the time required to undertake such a study.

2. Development of the TRRL design method

The decision was first taken by TRRL in the middle 1950's to attempt performance forecasting of in-service pavements on the basis of measured deflection. In order to gain widespread acceptance by the large number of Highway Authorities responsible for pavement strengthening, the aim was to develop a method based primarily on a straightforward technique of in-situ measurement. A programme of testing was undertaken on a wide range of combinations of pavement materials and thicknesses in over 340 experimental sections built as part of the national road network.

2.1 The influence of pavement temperature

Temperature influences the stiffness of bituminous materials and hence the deflection of flexible pavements. Correction charts are therefore required to reduce deflection measurements made at different road temperatures to equivalent deflections at a standard temperature. These correction charts based on experimental evidence have been developed for a wide range of bituminous material types and thicknesses (2) (3); an example is given in Fig. 1.
2.2 The relation between deflection and performance

The equivalent, or standard, deflection values were used to construct deflection histories for the experimental sections. Deflection histories relate standard deflection to pavement performance under known traffic expressed in terms of deformation in the wheel-paths and any cracking that developed. By comparison of deflection and deformation behaviour a critical stage in the life of a pavement can be identified.

The critical condition is defined as that point in the total life of a pavement just before the accumulation of damage begins to escalate, when some structural integrity remains and therefore the life of the pavement can still be extended economically by strengthening the existing structure with an overlay (2). In the United Kingdom, providing the accumulated damage is structural rather than superficial in origin, it has been found that the onset of critical conditions normally correspond to a rut depth of about 10 mm with some surface cracking.

Beyond critical conditions deformation and deflection begin to increase more rapidly leading to failure and the need for reconstruction of the pavement.

By combining large numbers of deflection histories collected over a 20 year period of measurement, relations were developed between deflection and life, expressed in terms of cumulative standard axles, Fig 2. These design charts were developed from pavements in which permanent deformation was the primary form of deterioration. Limited evidence of their use for predicting the remaining life of pavements that deteriorate through fatigue cracking is also available from the United Kingdom (1) and Australia (4).

2.3 Overlay design

When a pavement approaches a critical condition, its life can be extended by the application of a bituminous overlay. To specify the thickness of overlay, it is necessary to establish both the reduction in deflection brought about by overlays of different thicknesses and also the deflection-life relations of the overlaid pavements. This information was obtained from TRRL full-scale experimental sections supplemented by evidence from pilot-scale trials and consolidated into overlay design charts, Fig. 3. (3)

3. Extension of the method to lightly trafficked roads

The TRRL design method is based on measurements of deflection and pavement condition obtained primarily from medium to heavily trafficked roads. The lack of information on lightly trafficked roads was reflected in the design charts by showing the relations in this section with dashed lines on Fig. 2. To validate the accuracy of these projections the following additional information was required.
Temperature - deflection relations for the common types of materials used on lightly trafficked roads and further information on the position of the critical condition curve on the deflection-life chart.

Deflection-life relations for overlaid pavements were not studied because work by TRRL had established that, although overlaid pavements generally had longer lives than new pavements with the same deflection level, the differences in life were small (1). Therefore the relations defined for new pavements could be safely applied to overlaid pavements.

Further evidence of the reduction in deflection brought about by the application of overlays was not considered necessary because the evidence on which the main design document (3) was based covers a wide range of bituminous mixes and because similar data has been reported elsewhere (5) for other types of mix.

3.1 Data collection

The majority of the deflection measurements were obtained with a Deflectograph, the remainder with a Deflection Beam. (6) (7)

To enable the required data to be collected in a short period of time the development of complete deflection histories covering the life of sections of road from construction to failure was impractical. The concept of studying a 'slice in time' of a deflection history was established as a viable alternative procedure.

3.2 The slice-in-time approach

Following a single survey of a given site the varying deflection levels observed along the length of the road were linked to a visual condition survey; this defined the ranges of deflection levels corresponding to sound, critical and failed conditions, Fig 4. By combining the results from comparable sites that have experienced different cumulative traffic levels it was possible to construct the line representing the onset of critical conditions, Fig 4.

Typical results showing the upper limit of sound conditions using this approach are shown in Fig. 5. These results were taken from sites with non-cementing granular bases on a range of subgrade types and strengths.

In using this approach it is necessary to define the level of past traffic. It is also necessary to compare individual deflection measurements with the condition of the road at precisely the point of measurement. The level of surface deterioration recorded by condition surveys will also include all non-structurally associated deterioration and the correspondence between lengths of road classified as being in a critical condition or worse and the predicted position of the critical condition curves will be more variable than those shown for sound conditions.
3.3 Estimation of past traffic levels

Past traffic levels for each section of road were established from commercial vehicle counts, vehicle weight surveys using a portable weighbridge and estimates of past growth rate and past changes in vehicle damage factor (8). To obtain the best estimate of traffic loading it was found necessary to classify the commercial vehicle count by type and then, using vehicle weight surveys, determine for each type an average vehicle damage factor (8). The average factor for each type of vehicle was then combined in direct proportion to the number of vehicles recorded in each type from the commercial vehicle counts to provide an average vehicle damage factor for the site. Twelve-hour commercial vehicle count surveys on each of five days were combined with eight hour weight surveys on each of three days.

3.4 Extension of the design charts

The results of the study validated the position of the critical condition design curves, shown dashed in Fig. 2, for lightly trafficked roads for traffic down to 0.2 million standard axles and also permitted some extension to lower traffic levels. On the basis of the data collected some of the overlay design charts have also been extended.

It should be noted that the trend in deflection level with time has not been defined by the procedure adopted. This aspect of the design charts can be verified only by long term studies on a small selection of sites. In the present investigations it has been assumed that the general pattern of trend lines defined for medium to heavily trafficked roads apply equally to lightly-trafficked roads. Although a somewhat inexact approach, this will provide a more realistic estimate than making the assumption contained in many other design procedures, that deflection remains constant throughout the life of the road.

4. Recommended calibration procedure

Calibration of an existing design procedure requires a comprehensive appraisal to be undertaken of a representative sample of the total road network. The total length of road included in the sample will depend on local conditions but it should include sections representing the range of predominant environmental conditions, subgrade types, forms of flexible construction and types of traffic loading. For each of these classifications a range of pavement conditions from sound through critical to failed must be identified. To provide a statistically significant quantity of deflection and condition information every section representing a particular combination of conditions should have a minimum length of about 0.5 km.

To satisfy these conditions may often involve a total survey length of 300-1000 km. Such a programme will take about one to two years to complete,
including about six months of survey work and site testing, providing automatic deflection measurement and analysis procedures are available.

Detailed evidence of construction type and layer thickness of the test sections must be established. This will normally be achieved in a three-stage programme of coring. The first stage, with a coarse spacing of cores, (about 5 km intervals may be satisfactory), backed up with construction records where available, will identify potential test lengths. The second stage involves taking cores at 0.5 km intervals along the potential test lengths to enable the final selection of sections. A final stage, undertaken during the analysis period, may be required in which additional cores are taken from within the test sections in positions suggested by anomalous deflection and condition measurements.

Before the routine deflection survey work can begin it is necessary to define those periods when deflection measurements are likely to be significant in terms of the evaluation of pavement performance and when temperature profiles within the pavement and subgrade conditions are consistent enough to give reproducible deflection measurements. Therefore, following the identification of the main test lengths by coring, a selection of shorter temperature test sections is made, each around 25 m in length and covering the major construction material and subgrade types. Deflection beam measurements (6) (7) are then obtained at several points within each section at intervals throughout the anticipated testing season. Once the testing period is defined routine deflection testing of the total survey length can commence together with the survey of road condition. It is important to ensure that the condition of the road at precisely the point of deflection measurement is recorded. At intervals during the routine survey period, deflection beam measurements are continued at the points within the temperature test sections. These will enable temperature correction procedures to be developed. Deflection measurements on the temperature test sections, together with measurements on a wider range of control lengths monitored as part of the routine survey, will provide additional information on seasonal effects.

Parallel traffic studies can use either the procedures detailed in Section 3.3 or other techniques suited to local circumstances.

Finally, the information collected is used to develop deflection-life relations for the range of pavements studied. In determining the position of the critical condition curve local maintenance standards can be applied to the condition data when identifying the appropriate levels of deflection.

The information collected will, at a minimum, provide the evidence necessary to set priorities for maintenance of the lengths of pavement surveyed and, as further deflection surveys are undertaken, for other parts of the network. In addition, the data collected should either validate the direct use of existing design methods or form the basis of a modified design method.
directly related to local maintenance standards and to the specific existing maintenance criteria, traffic loading and material types covered by the investigation.

5. Conclusions

A detailed calibration procedure has been identified that can, within a period of one to two years effectively assess the accuracy with which an existing well proven highway maintenance design method might be applied to a new set of operating conditions. The procedure has been used to extend the TRRL design method to lightly trafficked roads in the United Kingdom.

6. Acknowledgements

The work described in this paper forms part of the programme of the Transport and Road Research Laboratory and the paper is published by permission of the Director.

References


Crown Copyright 1982. Any views expressed in this Paper are not necessarily those of the Department of the Environment or of the Department of Transport. Extracts from the text may be reproduced, except for Commercial purposes, provided the source is acknowledged. Reproduced by permission of Her Britannic Majesty's Stationery Office.

Fig. 1 Relation between deflection and temperature for pavements with less than 135mm of bituminous material of which less than 75mm is dense bituminous material.
Fig. 2  RELATION BETWEEN STANDARD DEFLECTION AND LIFE FOR PAVEMENTS WITH NON-CEMENTING GRANULAR ROAD BASES

Fig. 3  OVERLAY DESIGN CHART FOR PAVEMENTS WITH NON-CEMENTING GRANULAR ROAD BASES (0.50 PROBABILITY)
Fig. 4  THE SLICE-IN-TIME APPROACH

Fig. 5  RANGE OF DEFLECTION MEASURED ON SOUND PAVEMENT AT SIX LIGHTLY TRAFFICKED SITES