The design of bituminous concrete mixes

by

J N Preston B.Eng

Thesis submitted to the University of Nottingham for the degree of Doctor of Philisophy.

May 1991.

UNIVERSITY OF NOTTINGHAM

DEPARTMENT OF CIVIL ENGINEERING

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ABSTRACT

The highway industry in the United Kingdom relies on empirical specifications for control of the materials used in the bound layers of flexible pavements. The main aim of this research project was to develop a rational and practical mix design procedure for bituminous roadbase and basecourse materials. The approach which has been followed throughout the project represents a significant departure from conventional procedures and introduces a format for performance based end-product specifications.

The new mix design procedure, which has evolved from the research, concentrates on the principal influential variables which effect mix performance. These have been considered as the aggregate gradation, the binder content and the compaction level of the final mixture for a given grade of binder.

The importance of the method of specimen manufacture has been recognised, and the Percentage Refusal Density test apparatus was employed to address this problem. This equipment facilitates a kneading action to be imparted to the bituminous mixture and can accommodate large aggregates up to 37mm in size.

Material performance has been quantified through the analysis of volumetric proportions, which take account of aggregate packing characteristics and the mechanical properties of elastic stiffness and deformation resistance, measured using the Nottingham Asphalt Mix Tester. Examination of the results of these analyses, facilitates the identification of the optimum mix formulation for a given source of material.

ACKNOWLEDGEMENTS

The author would like to thank all the people who have provided help and advice throughout the duration of this research project. In particular he would like to express his gratitude to the following :

Mobil Oil Co. Ltd, the National Asphalt Pavement Association of the United States of America, and the Science and Engineering Research Council of the United Kingdom, for the financial sponsorship which facilitated the execution of the research.

Bardon Hill Quarry, Tarmac's Dene Quarry and Mobil Oil Co. Ltd, for the generous donations of materials, and Northamptonshire and Shropshire County Councils for the provision of road cores.

Professor Stephen Brown, for the provision of the excellent laboratory facilities at Nottingham University, and for his enthusiastic supervision throughout the project.

Mr Keith Cooper, who provided an inexhaustible wealth of knowledge of bituminous materials, and a stimulating source of innovative ideas.

Dr Chris Bell of Oregon State University who helped initiate the ideas which later developed into the mix design procedure.

Professor Peter Pell for numerous useful discussions and advice.

All the members of the Pavement Research Group, with whom it has been a pleasure to work. Particular thanks are due to Mr Geoff Rowe, the human encyclopedia on asphaltic materials, who provided great assistance during the writing of this thesis.

The jolly band of technicians, without whom all research in this department would take twice as long, with notable thanks due to Mr Eshan Sharegh, who was my reliable aide in the laboratory.

The unsung heroines who comprise the secretarial staff of the Civil Engineering Department, for the typing of this dissertation, and Mr J Smith for the tracings.

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CHAPTER ONE

INTRODUCTION

In recent years the highway infrastructure of a country has begun to play an increasingly important roll within the economy and as more and more freight is transported by road, the demands placed on the performance of the highway system are correspondingly severe. Maintenance programs on principal highways involve, not only the cost of materials and labour for reconstruction of a section of road, but also the huge cost associated with delayed journey times and uneconomical use of fuel.

Roads must be considered as civil engineering structures and pavement performance can only be optimised through a sequential design procedure which forms a structural analysis of the highway in terms of traffic loading and environmental conditions. The materials constituting the road layers must also be designed rationally to ensure that the resulting structure is capable of satisfying the design requirements.

Despite advances in engineering technology, the design of bituminous paving materials has remained empirically based. The complex nature of the material itself concerning the response to applied loading, demands that any approach to the design of bituminous materials must be a simplification, otherwise the problem will be too involved.

1.1 HISTORICAL REVIEW

Pavement engineering in the United Kingdom has been in existence since Roman times, when a network of stone carriageways was constructed to provide lines of communication between cities and encampments. Although some of their work is still visible today, the Roman roadbuilders had to perform maintenance operations on sections of road. Archeological findings have shown that some Roman roads have been patched and overlaid several times, and replacement lengths laid alongside the original. The excavation of Vindolanda just south of Hadrian's Wall uncovered a wooden tablet upon which a letter was inscribed by Octavius around AD115. The contents of the letter described the unacceptable state of the roads which had resulted in the delay of a consignment of cattle hides and grain from Catterick. He suggested that a pack of mules would facilitate an improved form of transportation.

The Romans left Britain in the fifth century and there was no comparable effort given to the nation's highways until the late 18th Century, when two men, working independently, began to apply engineering principles to the construction of roads.

The undoubted primogenitors of pavement engineering and material analysis in this country were Thomas Telford and John Loudon Macadam. Telford's constructions required the preparation of a level subgrade on which hand placed stones were laid broadest end downwards, forming a foundation layer. The surface interstices of this layer were filled with fine chippings and the central 5.5 metres of road was then covered in two layers of stone of about 100mm and 50mm thickness. Finally the whole pavement was covered with a binding layer of gravel approximately 40mm thick, the compaction of which being left to the action of weather and traffic. Cross drains were often laid below the hand placed stones at about 90 metres intervals discharging into side ditches.

Macadam's constructions adhered to the philosophy that a road should comprise clean broken stone "which must be so prepared and laid as to unite by its own angles into a firm, compact impenetrable body". His foundation was shaped to follow the camber of the road surface to ensure good side drainage, and a uniform constuctional depth over the width of the road. Macadam considered that two 100 to 150mm layers of 75mm broken stone would satisfy the structural requirements of the pavement on top of which a finishing layer of angular broken stone was placed and rammed into the inserstices of the lower layer.

Both Telford and Macadam had strict specifications concerning the size, shape and quality of the stone used for highway construction, between them inaugurating the engineering principles of road construction which still apply today.

The introduction of the use of rock asphalt as a cosmetic treatment on city streets finally led to the evolution of dense bituminous mixtures, of which the aggregate composition was based on the principles of Macadam and Telford. The eventual development of 'Tarmacadams' was empirical, with recommendations concerning binder contents evolving from engineering experience.

Current U.K practice employs such materials for the main structural layer of flexible pavements and specifies their composition through recipe methods. The revised edition of BS 4987 published in 1988 (1) still adheres to this format, without any recommendations concerning mix design or performance based criteria.

1.1.1 Overseas Developments

Heavy duty asphaltic mixtures have been used in the USA since 1930, and innovations in machine laying at the same time facilitated extensive paving operations to be performed. For the structural layers of asphalt pavements, the aggregate gradations were based on a maximum density curve (2), with a maximum stone size of approximately 18mm. These fine graded mixtures differed considerably from the materials used in the United Kingdom at that time, and were known as 'sheet asphalts'. A series of failures however led the engineers to realise that a specific quantity of asphalt must be mixed with the aggregate in order to produce mixture which exhibited satisfactory performance. One of the earliest methods of mix design was developed by Hubbard and Field during the 1920's (3), and was applied to the 'sheet asphalts' of the time. The Hubbard Field test employed a punching shear mechanism, on specimens formed by a combination of hand tamping and direct compression, at a temperature of 60°C. The maximum load developed under the test conditions was reported as a stability number, values of which were plotted on charts against the asphalt cement content. The test constituted an attempt at rational mixture design and proved satisfactory for the designs of the day.

Work performed in California during the 1930's by Francis Hveem resulted in the Hveem stability test which quantified mix formulations of asphaltic materials through the application of a compressive load on a specimen whilst simultaneously maintaining a confining stress. The test, which is still used by some agencies in the United States, assesses a series of mix formulations over a range of binder contents and makes recommendations based on stability numbers and the volumetric proportions of test specimens.

The change in aircraft technology during the second World War led to increased loadings and tyre pressures from military aircraft, and an improvement to the Hubbard Field test was required in order that airfield pavements could be designed and perform satisfactorily under the new conditions. The Marshall method, (5) which evolved from this era, has subsequently dominated mix design work worldwide. The objective is to establish an optimum binder content for a fixed aggregate gradation through the application of a mechanical test and analysis of the volumetric proportions of the specimens.

1.2 CURRENT IDEOLOGY

Continously graded bituminous materials are extensively used for pavement roadbase layers in the United Kingdom and many other parts of the world. U.K practice is governed by recipe specifications, which have been developed empirically as previously described. Recipe specifications are convenient to use but, in the absence of any performance based criteria and engineering analysis, cannnot be applied to explain a section of failed material. When a wide grading envelope is combined with a range of binder contents varying from 3.5% to 4.7%, to form a material specification, the possible combinations of the two components of stone and bitumen may result in mix formulations which exhibit substantially different mechanical properties and yet all comply with the advocated specification. It is also inappropriate to propose one grading envelope and one target binder content to cover mixtures containing aggregates and binders from different sources. An article in the 'New Civil Engineer' from January 1990, reported on the dissatisfaction of Cleveland County Council, with the recipe mixed materials of BS4987 : "Recipe mixed materials are not the answer to modern day traffic problems. With recipes there is no engineering input, it is a total compromise."

Such reports emphasize the increasing awareness amongst engineers of the shortcomings of recipe approaches and the need to quantify materials in term of their fundamental properties. A survey performed in 1990 indicated that a total of 350 mixes were programmed into asphalt plants, two thirds of which were coated Macadams and one third rolled asphalt materials (4). Such a figure reflects the confusion which has been generated within the industry concerning the applications of bituminous materials and underlines the need for a rational approach to mixture specification.

In countries where roadbase and basecourse materials are designed by procedures such as the Marshall method, there exists widespread recognition of the inadequacies of these procedures, and an acceptance that a more fundamental approach is required. This is particulary true in the U.S.A where Marshall design dominates the production of asphalt concrete mixes, yet various modes of pavement distress are regularly manifested.

1.3 OBJECTIVES OF THE RESEARCH

The preceding discussion has outlined the need for an investigation into the development of a rational mix design procedure for the main structural layer of flexible pavements. The main objective of the project was the development of a procedure which would lead to the optimisation of available resources in the production of continosly graded binuminous materials for use in roadbase and basecourse layers of flexible pavements and the quantification of the material properties through relevent end-product testing. The second aspect of the research objective necessitated the development of practical test methods to quantify the elastic stiffness and deformation resistance characteristics of mixes.

The work used current U.K practice for 28mm Dense Roadbase Macadams incorporating 100 pen bitumen as the starting point. The use of modified binders was not addressed as part of this investigation. Close collaboration with material suppliers was maintained throughout the duration of the project, facilitating an evaluation of the effect of different aggregate types on the bituminous mixtures under scrutiny.

This thesis describes the methodology which was applied to the development of a rational mix design procedure for dense bituminous concretes and introduces a philosophy which signifies a departure from previous thinking. A pragmatic approach was sustained in all aspects of the work in an effort to produce a practical and implementable design method. Asphalt technology has achieved a status far in excess of that executed in practice and this project has endeavoured to use and extend the finding of previous research to establish a format for design of road base materials.

The sequence of chapters in this thesis describes the approach, and influential factors which dictated the route followed by the research. The principal conclusions which have arisen from the investigation are documented in chapter 12, and the potential implications of the research findings are discussed in chapter 13. It is hoped that the design procedure which has evolved from the project represents a significant advance in the field of mix design although there remains considerable work to be done before full implementation. Chapter 14 outlines the areas where future research should be concentrated in order that a full and comprehensive understanding of the problem may be achieved.

CHAPTER TWO

DEVELOPMENTS TO THE STATE OF THE ART

2.1 EXISTING METHODS

Bitumen and tar bound pavement layers have dominated the construction of principal highways for the majority of the twentieth century. Associated investigations into mixture compositions led to the empirical development of mix design procedures which attempted to quantify mixture properties through some form of mechanical test. These developments are heavily rooted in the United States of America, where the established design methods are still in use today, but their influence has migrated all over the world, including some European countries. The United Kingdom has remained detached from external influence concerning the composition of roadbase materials, and still relies on recipe specifications which are largely based on experience.

Of the design methods available, two have emerged as those most commonly used by asphalt paving technologists throughout the last four decades. The current specifications regarding the Hveem and Marshall methods of mix design (5) have evolved empirically, and are required to produce a mixture which provides:

(a) sufficient bitumen to ensure a durable pavement.

(b) sufficient mix stability to satisfy the demands of traffic without distortion or displacement.

(c) sufficient voids in the final compacted mix to allow for a slight amount of additional compaction due to traffic loading without flushing, and loss of stability, yet sufficient voids to prevent the ingress of air and moisture.

(d) sufficient workability to permit the efficient placement of the mix.

2.1.1 The Marshall method of mix design

If a highway engineer with a knowledge of paving materials were asked what was the most significant development concerning the design of bituminous concrete mixtures, the probable response would be "the Marshall Test". The introduction of this procedure coincided with a need for a mix design method for asphaltic concrete to cope with increasing wheel loads and tyre pressures of military aircraft during a frantic period of airfield pavement construction in the USA throughout the later stages of the second World War.

The US Army Corps of Engineers selected the testing device which had been developed by Bruce G Marshall of the Mississippi State Highway Department, and employed the apparatus in the role of a mix design tool. Cylindrical specimens of asphaltic material, 100mm in diameter by 60mm high, were subject to a constant strain compression test at a temperature of 60°C. Measurements of the maximum load required to generate structural failure of the specimen, and the amount of deformation exhibited by the sample during the loading process were taken during the test. These two parameters became known as the Marshall stability and Marshall flow respectively (3).

Specimens of material were manufactured over a range of asphalt contents and from the test results the Corps of Engineers plotted bulk density, stability, flow and percentage voids in the mix, against the asphalt content. The design asphalt content corresponded to the mix formulations averaged from the maximum stability curve, the maximum density curve, and those mixtures satisfying criteria applied to the void contents which were based on values from existing pavements.

The simplicity and ease of performing the Marshall test helped the method gain a status of great popularity, and the procedure soon became extensively used for the design of highway pavements. The test was standardized in 1958 (5).

2.1.2 The Hveem method of mix design

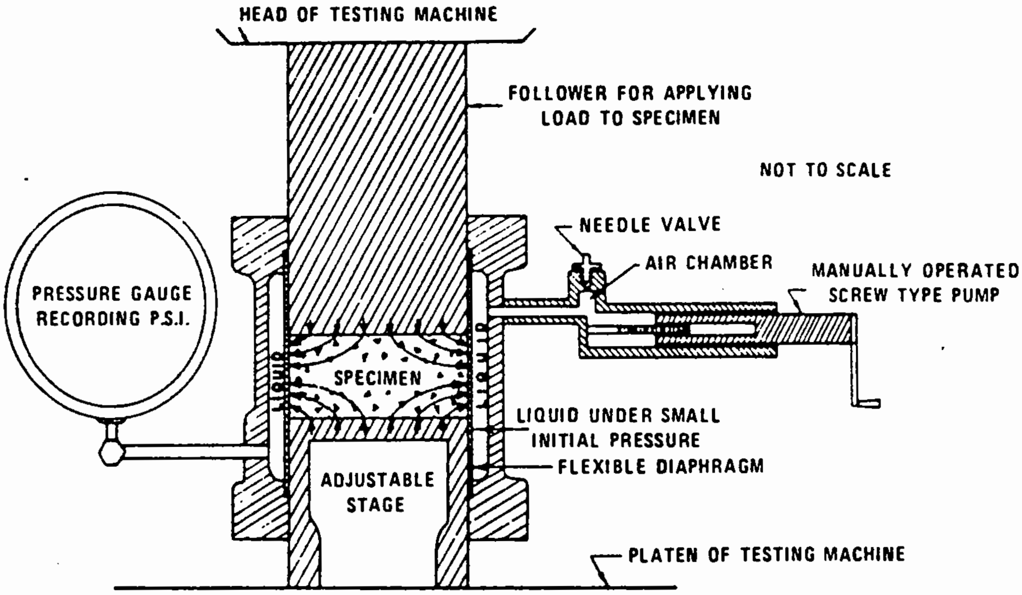
Although different in execution to the Marshall test, the Hveem method of mix design provides another procedure for determining the optimum binder content for a fixed aggregate gradation. The method is based on the relationship between the surface area of the aggregate particles within the gradation, and the quantity of asphalt required to provide a film of certain thickness on the aggregate particles. The introduction of the Centrifuge Kerosine Equivalent (CKE) test (6) allowed a rational procedure to be applied to the estimation of aggregate surface area and subsequent calculation of optimum asphalt content. The resulting binder content provided the datum for the range of asphalt contents at which specimens should be manufactured, and subject to a mechanical test.

Triaxial loading is applied to an asphaltic specimen where a compressive axial stress is superimposed on a compressive confining stress. The apparatus used for the test is called the Hveem stabilometer which is shown diagramatically in figure 2.1. The specimens which exhibit the greatest stabilometer values above a minimum criterion are then visually assessed to examine flushing effects, with the design binder content representing the highest asphalt content satisfying the stabilometer test which does not show any signs of binder flushing. A swell test is then performed on companion specimens to determine the resistance of the compacted specimens to water. The present day status of the Hveem procedure had fully evolved by 1959, and was incorporated into the design standards (5).

The Hveem method of mix design exists in the shadow of the Marshall test, as depicted by figure 2.2 (7), with 76% of highway agencies in the USA practising Marshall, and negligible execution of the Hveem method elsewhere in the world. Both methods apply a mechanical test to a specimen of the bituminous mix, and design judgements are made on the basis of these results, the philosophy of the design content selection being shown in figure 2.3.

2.2 DEVELOPMENTS TO TEST METHODS

The fundamentals of the Marshall and Hveem mix design methods were established in the late 1950's, and have really seem little change in the ensuing thirty years, despite significant changes in traffic volumes and axle loadings. The basic compaction and test procedures have remained intact, with only minor modifications introduced subsequent to the



NOTE–The specimen is given lateral support by the flexible sidewall, which transmits horizontal pressure to the liquid. The magnitude of the pressure can be read on the gauge.

Figure 2.1 Diagrammatic representation of the Hveem Stabilometer

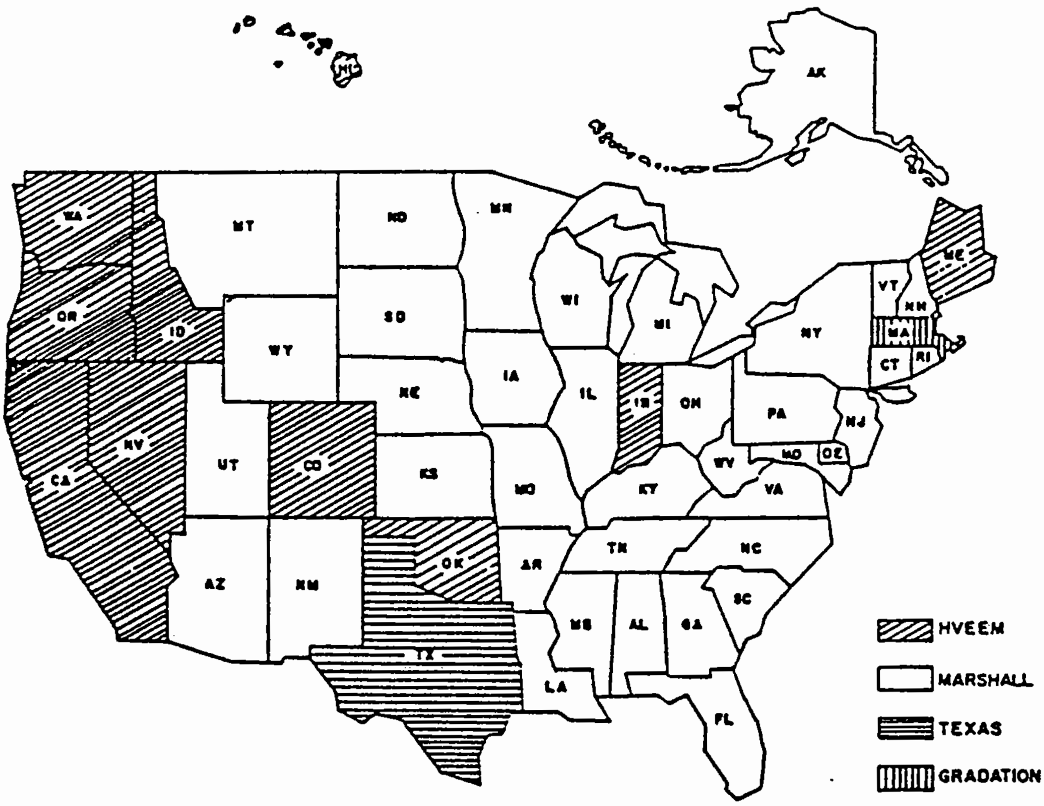


Figure 2.2 Illustration showing the dominance of the Marshall Test in the USA

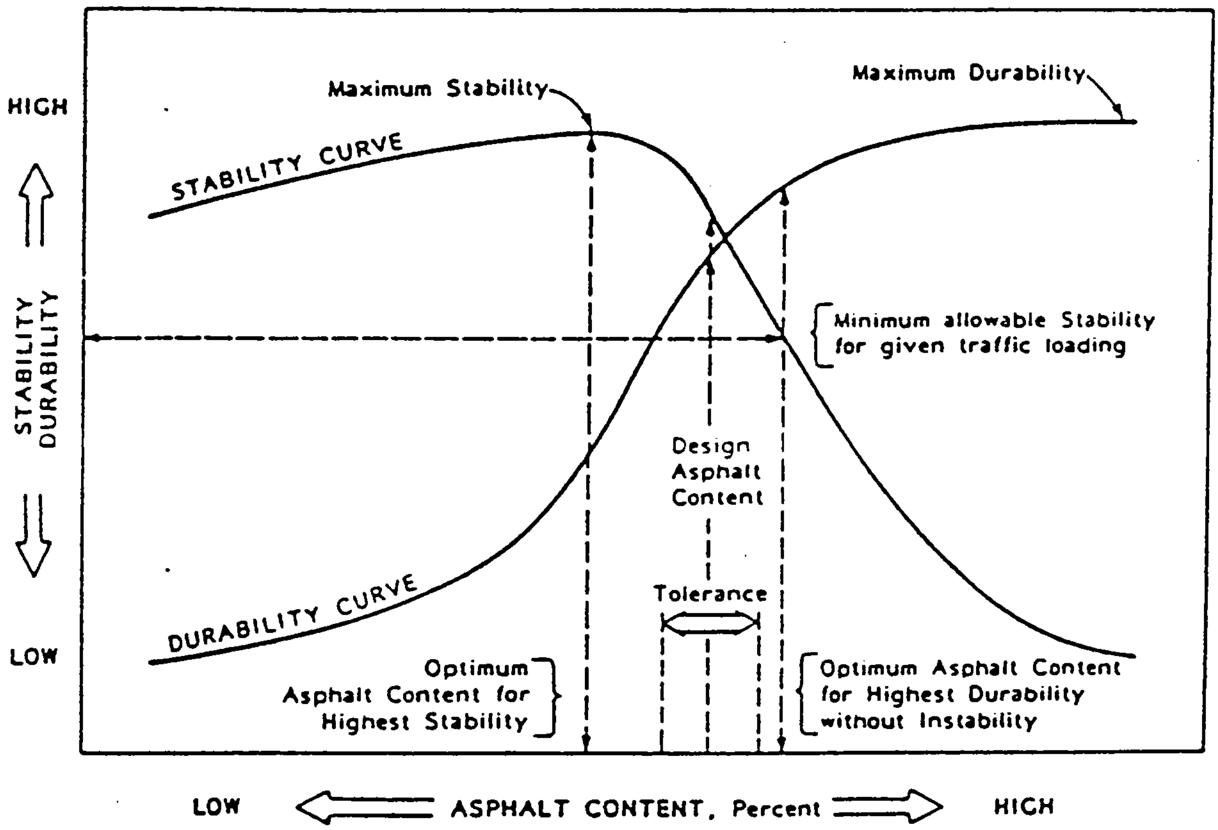


Figure 2.3 Philosophy of the selection of design binder content for asphaltic concretes (after Monismith). (ref 17)

application of the procedures to a wide range of mixtures and traffic conditions, resulting in the enlargement of the data base. Mixing and compaction temperatures were related to asphalt viscosity rather than being fixed numbers, and the severity of traffic was catergorized into light, medium and heavy. To cope with variations in traffic loadings, the level of compaction of the specimens in the Marshall test, was divided into three categories according to the number of blows from the drop hammer.

Probably the most significant change from the Corps of Engineers original method was the substitution of a criterion on per cent voids filled with asphalt, to a voids in mixed aggregate criterion (VMA) (3). Despite the introduction of this parameter, only 16 out of 38 states in North America adhere to this criterion of a minimum VMA in the mix (7). The majority of developments to existing methods have concentrated on the Marshall procedure, and the design of wearing course mixtures, leaving roadbase technology to stagnate. Visible deformations in the wheel tracks of United Kingdom rolled asphalt wearing courses led to the introduction of the Marshall method into the 1973 edition of BS 594 (8) for the selection of design binder content, whilst empirical recipes continued to specify roadbase mixtures. The Shell Organisation investigated a new interpretation of Marshall test results by considering the value of Marshall stability divided by the Marshall flow (9). The resulting Marshall 'Quotient' is generally taken as a measure of mix stiffness, as it is determined from a division of a load and a deformation, but the conditions under which the values are obtained means that the quotient is not representative of an elastic stiffness but more analogous to a viscous stiffness.

2.2.1 Use of the Marshall quotient

The implementation of the quotient as a method of analysing the material properties is broadly European based, with little interest expressed in the parameter in the USA. Although not included in a British Standard, several local highway authorities impose limitations to the Marshall quotient as a criterion for the design of rolled asphalt wearing courses.

Lees (10) proposed an amalgamation between British and American practices, and implied that "immediate adjustments in Marshall procedures and interpretation could be made which it is suggested would be beneficial". Lees very astutely identified the differences in the range of acceptable mixes resulting from a Marshall test when the quotient parameter was introduced as a limiting factor. Figure 2.4 illustrates Lees' argument, and identifies ranges of mix-formulations which currently satisfy criteria and yet will probably exhibit inferior performance to other mixes meeting the same criteria, and mix types which current specification rejects, and yet would probably show satisfactory performance.

This type of analysis shows a rational approach to the interpretation of mix design data, and tries to introduce a greater understanding of the mechanics involved in bituminous surfacing materials. Lees extended his investigations to propose Marshall test criteria to acknowledge traffic levels and pavement thickness, which are shown in figure 2.5. However, this type of approach remains totally reliant upon a design procedure which has become inappropriate and unsatisfactory by which to optimise resources for the use of materials in pavement construction, and more fundamental modifications are required.

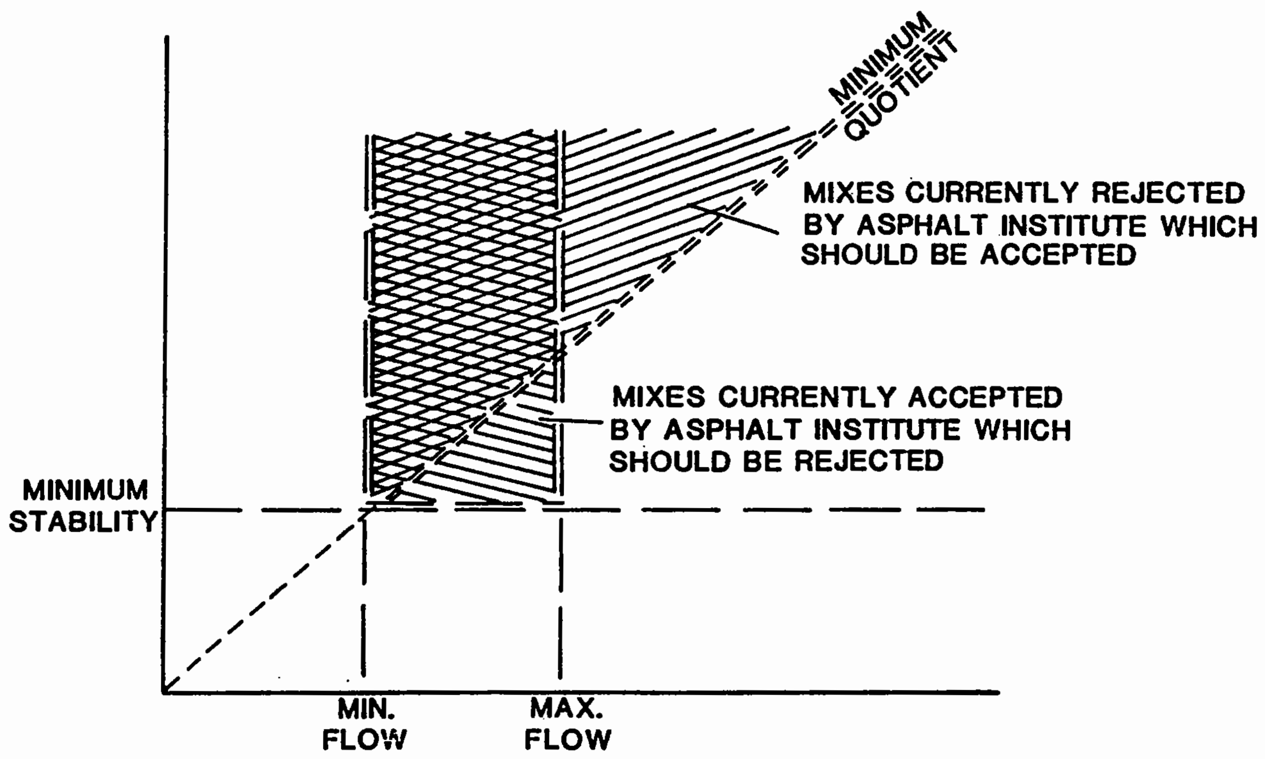


Figure 2.4 Modification to the criteria applied to the Marshall Test, by the introduction of minimum quotient (after Lees)

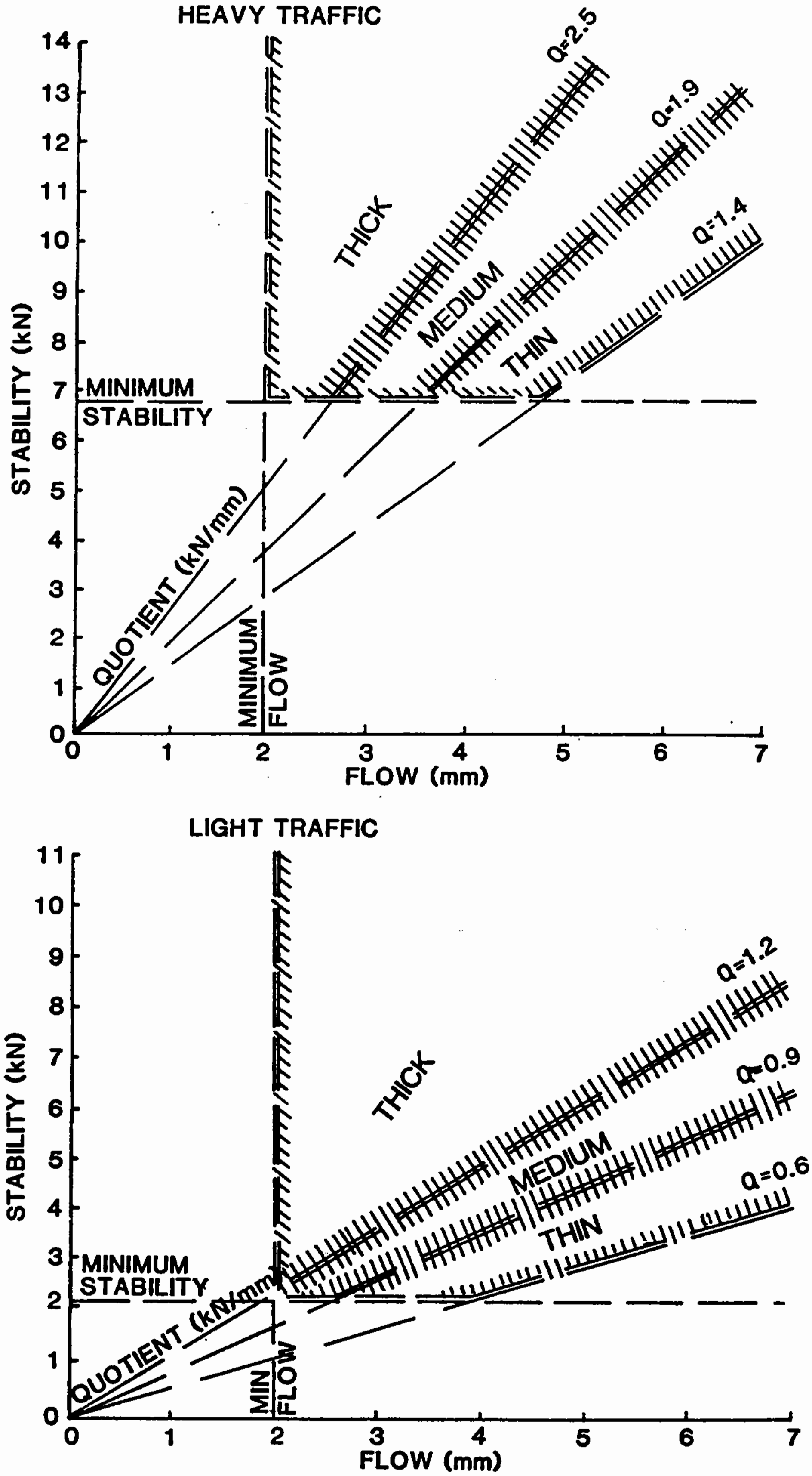


Figure 2.5 Marshall design criteria to accommodate different traffic levels and pavement thickness. (after Lees)

2.2.2 Large stone asphalt mixes

The structural strength of bituminous bound roadbase materials is dependent upon the mechanical interlock between the aggregate particles, which necessitates the formation of a tight stone skeleton. British practice has traditionally employed a maximum particle size of 37mm, which in the absence of a mixture design procedure has not created problems, but agencies specifying mixes through the Marshall test are constrained because of the dimensions of the mould. The Marshall apparatus (11) consists of a 100mm diameter compaction mould, which is intended for a stone of maximum size 25mm, therefore inhibiting the design of materials comprising larger aggregates. There has been a recent trend in the USA to produce 'large stone asphalt mixes', which contain aggregates of up to 50mm in size, and therefore cannot be designed using conventional Marshall procedures.

The solution to this problem has been to increase the dimensions of the Marshall apparatus, based on a mould diameter of 150mm (9). The development to the equipment is considered satisfactory to deal with gradations containing aggregate particles up to 50mm maximum size, and has been used by the Kentucky Department of Highways, and the Pennsylvania Department of Transportation as a mix design tool (12). The method adheres to the Marshall procedure, but has the following significant differences:

1) The mass of the drop hammer is 10.2kg which requires a mechanical operation.

2) The mould has an internal diameter of 150mm and a height of 95mm.

3) The number of blows to sustain the same level of compaction in a 100mm diameter specimen is increased by 50%.

The trials of this modified Marshall procedure by the aforementioned agencies have led to the recommendation of this approach for the design of bituminous materials incorporating aggregates with a top size greater than 25mm.

Although these modifications to the Marshall test may produce a more appropriate procedure for the design of continuously graded materials with a large maximum stone size, there remains a fundamental flaw in the philosophy. The Marshall test is an unsatisfactory procedure by which to design dense bituminous concrete. It relies upon an unrepresentative specimen fabrication process, and applies an arbitrary mechanical test to evaluate different mix formulations. Any design method which is an extension of, or based on the Marshall method, utilises an imperfect datum and hence extends the erroneousness of the procedure. However, despite the general acceptance of this amongst researchers and personnel within the highway industry, "most agencies are reluctant to buy new equipment, and tend to prefer and utilize existing equipment and methodologies" (9).

2.3 THE NEED FOR A NEW METHOD OF MIX DESIGN

The probable reason why the Marshall and Hveem methods of mix design, and the United Kingdom recipe mixtures gave apparently sterling service for many years was that many highways did not receive sufficient loading to cause distress to the integrity of the pavement. A road must be considered as a structure, and distress mechanisms will be manifested only if design parameters are exceeded. The increasing frequency and severity of axle loadings during the last two decades have exposed the limitations of existing design methods and made engineers realise that a more rational approach is required.

Commentaries on the shortcomings of existing design methods, and the lack of correlation between design and field performance have been published (13,14) drawing attention to the need for a more fundamental approach.

The fundamental philosophy of a mix design procedure for any material is the manufacture of a representative sample of that material, and an application of mechanical tests to evaluate whether the material adheres to design criteria. Although existing methodologies vaguely follow these guidelines, neither of the above requirements are satisfied. The reproduction of site material in the laboratory has proven to be very difficult, but it cannot be achieved from the action of a vertical drop hammer imparting a load to the mix as in the case of the Marshall test. Secondly the quantification of material properties must equate to the mechanical properties which dictate field performance.

The application of a Marshall stability test or a Hveem stabilometer test, may give some indication of a material strength, but fails to address the fundamental engineering properties. The use of recipe specifications do not provide any means of assessing material properties, thus preventing an engineering analysis of a failure.

The mechanical properties of bituminous concretes effecting pavement performance have been identified as the following:

1 Mixture stiffness (elastic)
2 Resistance to permanent deformation
3 Resistance to fatigue cracking
4 Durability
5 Low temperature fracture characteristics (mainly a US and continental phenomenon)

A mix design method for bituminous highway materials must quantify mix formulations in terms of those properties, which necessitates the use of different test equipment from the apparatus associated with Marshall and Hveem. Such test methods must comprise inexpensive equipment which facilitates rapid, and easy testing of mix specimens, and a straightforward interpretation of data. The development of a new mix design procedure must incorporate such tests so the performance characteristics of materials may be evaluated. The recent trend in areas of the USA of adjusting the criteria of Marshall and Hveem designs in order to improve mixture performance does not constitute an answer to the problem (15).

2.3.1 New design methodologies

The mix design symposium held by the Association of Asphalt Paving Technologists in 1985 introduced the philosophy of applying state of the art knowledge, and procedures of asphalt technology into the role of a mix design method. The paper by Monismith et al (16) emphasized the significant mixture properties which must be addressed through a mix design procedure, and recommended test methods which could be employed to quantify material performance. Although the paper publicised a recognition of the necessary format of the quantification subsystem in a design method, it failed to address the problem of specimen manufacture and the influence of the aggregate grading. A subsequent publication by Monismith (17) outlining a comprehensive asphalt concrete mixture design system took account of the factors which had been omitted previously, and recommended a form of kneading compaction for the preparation of laboratory samples. The mix design system is illustrated in figure 2.6 where the general framework is divided into a series of subsystems which evaluate the salient parameters associated with pavement performance. The final mix formulation advocated from the procedure must achieve a desirable balance between the distress modes under consideration for a specific pavement application.

A similar methodology has resulted from investigations performed by the Texas State Department of Highways and Public Transportation (SDHPT) (18), where the Texas gyratory-shear method of compaction is used in specimen fabrication, and three modes of distress : permanent deformation, fatigue cracking and low temperature cracking, quantify mixtures through a series of failure criteria.

These types of approaches undoubtedly constitute a significant advance from the traditional Marshall and Hveem methods of mix design. The suggested procedures assess mix formulations in terms of end product performance tests, and apply criteria which may be varied to allow for regional conditions. Although environmental differences are

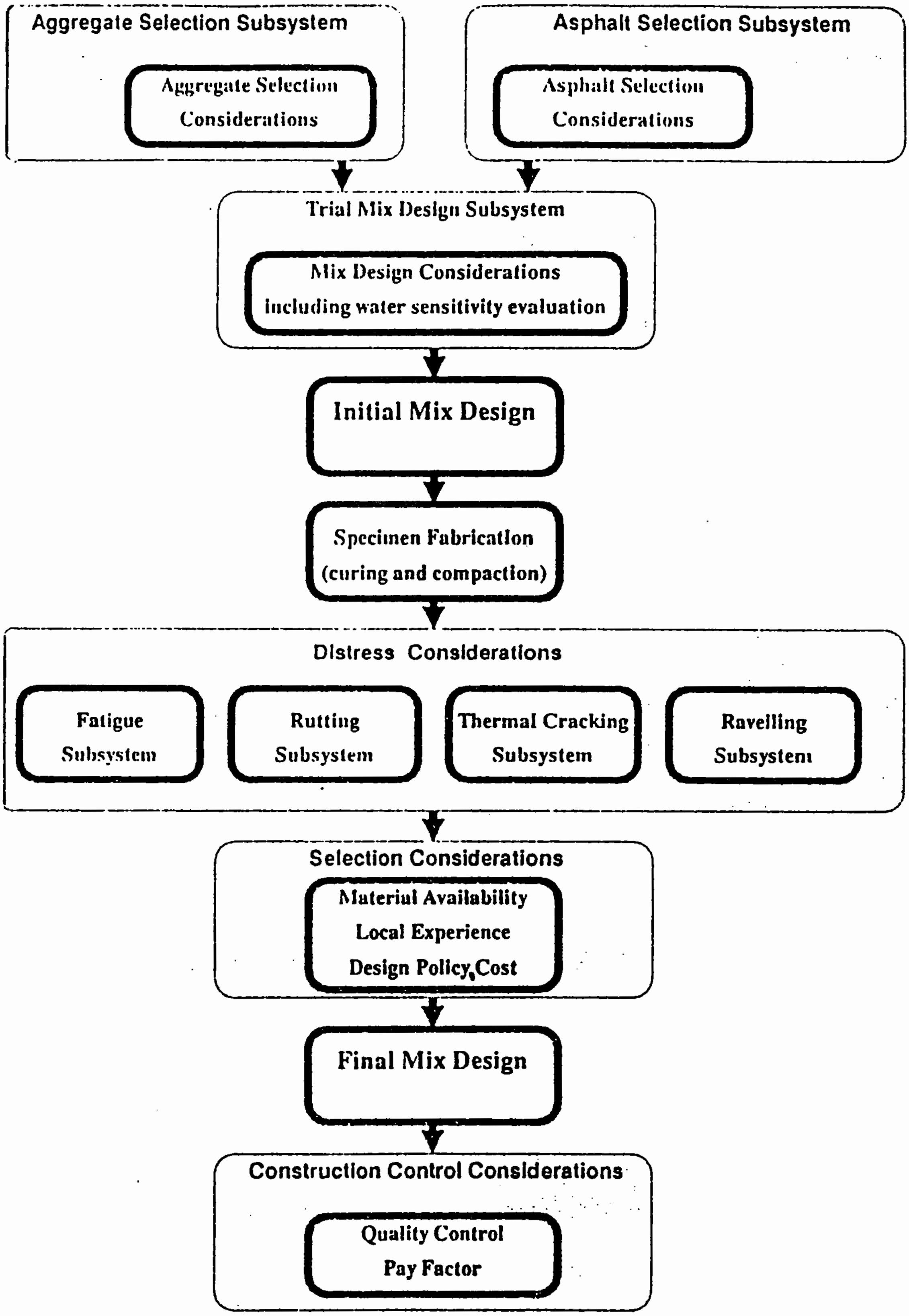


Figure 2.6 A comprehensive design system for asphaltic concrete.
(after Monismith)

incorporated within the regional variations, there is insufficient attention paid to the changes in aggregate properties and the influence of different gradations which occur with aggregates from different sources. Roadbase materials are combinations of crushed rock and bitumen, with each component having strong influences on the mix properties. A quantitive evaluation of variations to both of these components is a fundamental requirement if an optimisation of material resources is to be achieved.

The absence of mix design research in the United Kingdom is very apparent, and is reflected by the continuing dominance of recipe specifications for roadbase materials. The general availability of good quality aggregates within the UK, has led to the production of satisfactory pavement materials, but research has shown that material performance can be improved through small changes to current practice and through rational mix design (14). Recent investigations by Cabrera et al (19) and Elliot (20), have proposed design methods for UK practice, but a comprehensive and implementable procedure awaits development.

2.3.2 The A.A.M.A.S and S.H.R.P investigations

The realisation that the methodology applied to mix design practices, requires updating through technology, has led to the concentrated studies of the asphalt-aggregate mixture analysis system (AAMAS), and the Strategic Highway Research Program (SHRP). Both studies have been initiated in the USA and receive the benefits of substantial financial backing to try and guarantee results from a very large co-ordinated research effort.

2.3.2.1 The A.A.M.A.S project

The objective of the AAMAS project is to develop an asphalt-aggregate mixture analysis system for the laboratory evaluation of asphalt concrete mixtures based on performance related criterias. This anticipates the determination of the in-place properties from plant mixed and roadway placed materials during the mixture design stage, which must relate to the critical parameters affecting pavement distress. A flow chart depicting the step by step procedure of AAMAS is shown in figure 2.7.

The initial status of AAMAS consisted of three primary phases (21) firstly the mixture design phase, secondly a mixture compaction/conditioning phase, and finally the integrated mixture evaluation phase. Phase 1 builds on the current methods of Marshall and Hveem procedures to provide an initial mixture design, which is evaluated in terms of engineering properties of the resultant mixture. The selected mix progresses to phase 2 where the properties and characteristics of the in place mixtures are duplicated in the laboratory. This involves compacting the materials in the laboratory to attain the same characteristics of field cores both immediately after compaction, and after several years in service. This subsystem alone requires a compaction study, and investigation into the age hardening of the mixture, and an assessment of moisture damage.

The final phase is an investigation into the performance of mechanical tests which must be reliable, reproduceable, efficient and simple to execute. The initial thinking within the AAMAS study concentrated on the standard resilient modulus (elastic stiffness) and permanent deformation (creep) tests, although new test methods may result from the

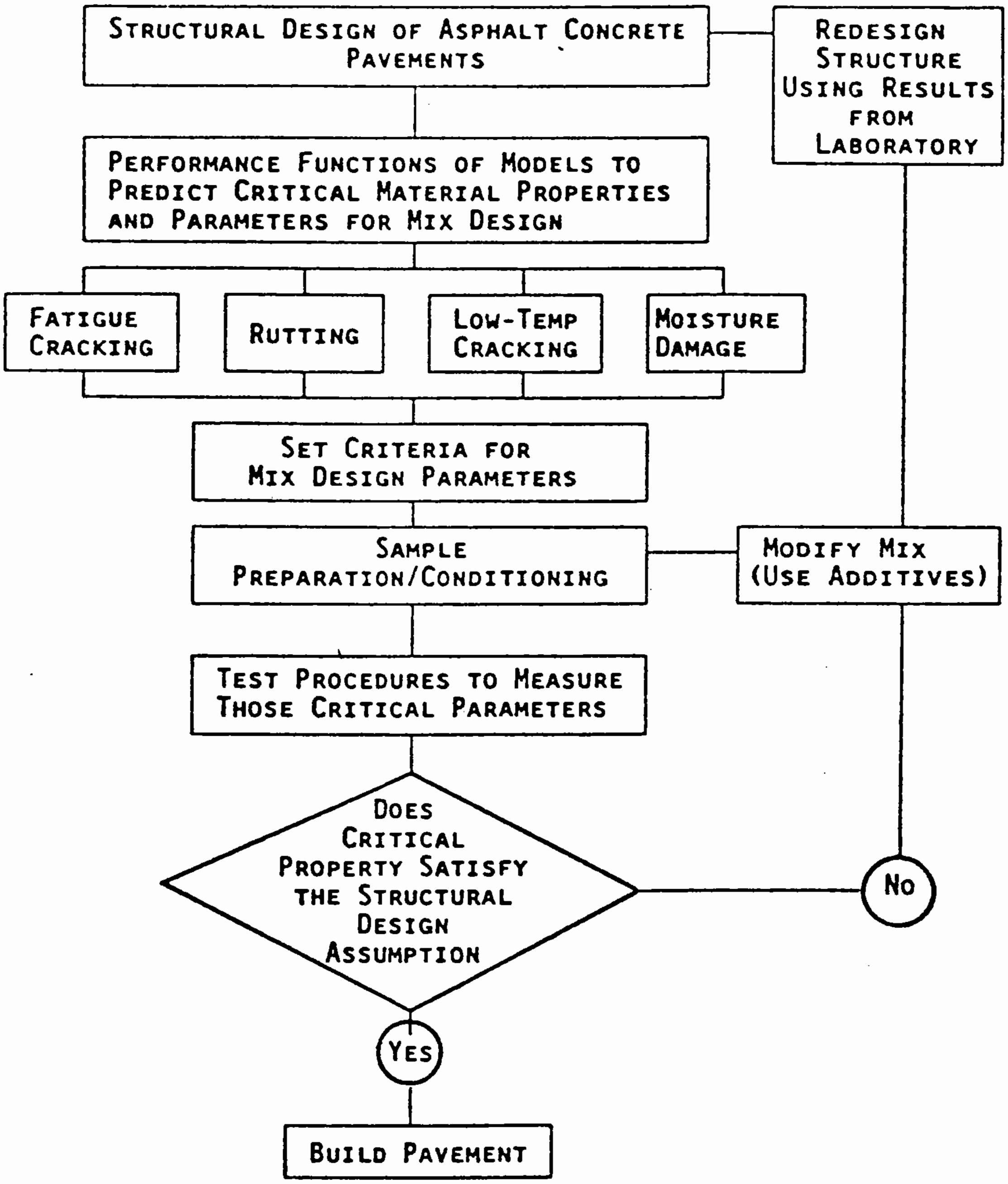


Figure 2.7 Outline of the Asphalt-Aggregate Mixture Analysis System (AAMAS) (after Von Quintas) (71)

investigations. Several state highway agencies are involved in the AAMAS project, facilitating the construction of field sections, which allows the generation of a data bank of information concerning phase 2 of the study philosophy, and provides a means of assessing the practicalities of mixing, laying and compacting new materials.

2.3.2.2 The Strategic Highway Research Program

The Strategic Highway Research Program is another concentrated effort targeting on improving specifications for bituminous binders and asphalt-aggregate mixtures, and introducing accelerated performance related tests to support the new specifications (22). The results of the program may then be implemented through an asphalt-aggregate mixture analysis system. The work is divided into six major contracts, figure 2.8, which are supplemented by a number of smaller supporting contracts. The main focus of the asphalt program is the effect of the composition and properties of the bituminous binder on pavement performance, which is addressed in the A-002 series of contracts. It is anticipated that the results of this part of the program will lead to new tests for the characterisation of binders and a greater understanding of the overall influence of the role of the asphalt within the final mixture. The asphalt-properties are being related to the in-service mixture performance through the A-003A contract, with mix performance being assessed through the distress modes of fatigue, deformation resistance, thermal cracking and durability associated problems.

It is this aspect which has the greatest implications with respect to a mix design procedure, as the results of the investigations from the SHRP

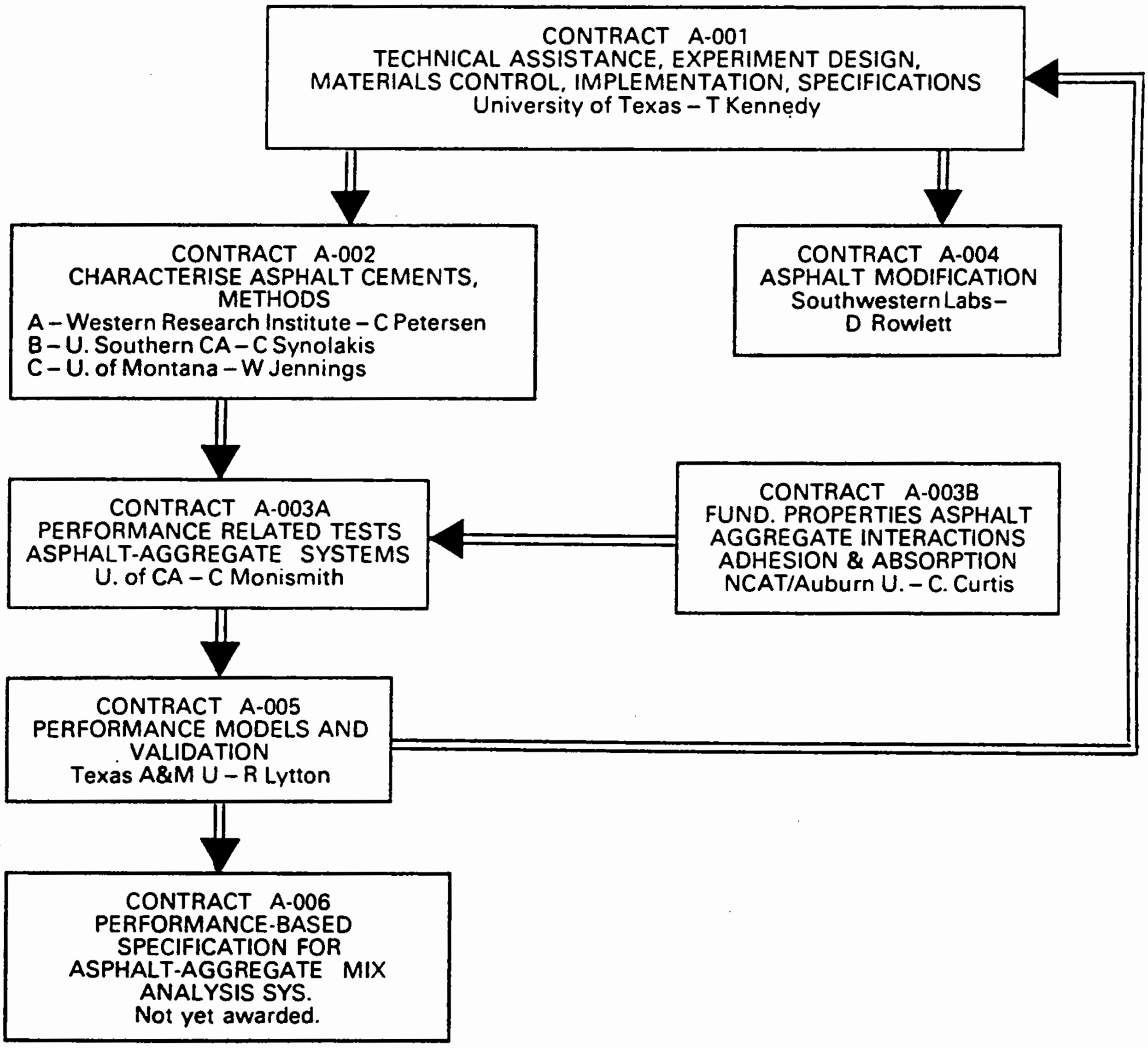


Figure 2.8 Outline of the Strategic Highway Research Program (SHRP) indicating specific areas of research.

A003-A contract will form the basis of recommended test methods for the evaluation of the fundamental properties of bituminous paving materials. For each distress mode, a number of candidate tests are being assessed from which the eventual laboratory test procedure will be selected. Attention must be given to test practicalities in terms of requirements for specimen manufacture, ease of test performance and cost of test apparatus, if the recommended procedures are to be implemented as routine test methods. Field simulation tests are also being performed to generate further data on material properties, and results from the full scale pavement sections, constructed as part of AAMAS, will also be incorporated within the SHRP study.

The A 003-B contract forms an investigation into the intimate relationship between asphalts and aggregates, aiming at modelling the chemistry of the aggregate-asphalt bond, and developing tests for evaluating adhesion and absorption.

These preliminary investigations require validating before a series of performance based specifications for bituminous materials may be implemented through an asphalt-aggregate mixture analysis system.

2.4 DISCUSSION

Despite widespread recognition of the limitations of existing design methods, bituminous paving materials are still specified predominantly through recipes or the Marshall and Hveem procedures. Most research has concentrated on different interpretations of results from the current design approaches, or recommendations of minor changes to these imperfect tests. It is evident that a completely new approach is required which, firstly manufactures representative samples of site material in the laboratory, and secondly, quantifies the mechanical properties of the material pertaining to in-service distress modes through a series of performance related tests. The on-going research efforts associated with AAMAS and SHRP are addressing the second aspect concerning the development of laboratory tests for material evaluation, but are failing to direct sufficient attention to the first part of a mix design method involving making changes to the parameters of mixture constituents, and the manufacturing of laboratory specimens. The development of AAMAS is still using Marshall and Hveem as a datum for fabricating laboratory material. The need for a new approach to the design of dense bituminous concrete is very apparent, and must be developed practically. It must employ state of the art asphalt technology and yet maintain a simple procedural format. Emphasis must change from the single parameter of the optimisation of a binder content, to include optimising the aggregate grading and optimising the level of compaction of the final mixture.

Laboratory procedures must be made as simple as possible, and test methods should facilitate rapid and easy evaluations of material performance. In the absence of satisfactory tests in current use, a new mix design method must include test equipment, which at present is not commonly found in highway laboratories. The proposed introduction of new equipment necessitates a comprehensive test apparatus, capable of quantifying fundamental material properties, which is seen as economically viable by the highway industry. For UK practice, deformation resistance qualities and the elastic stiffness of materials should be measured utilising simple test methods such as creep and indirect tension (23), with fatigue resistance estimated through the application of a prediction model, in the absence of an appropriate test. It is clear that there exists a need for a rational approach for the design of dense bituminous concrete, which can be applied to a variety of conditions by adjusting the criteria of the fundamental engineering properties of the material to satisfy the requirements dictated by the individual case.

CHAPTER 3

THE DESIGN OF AGGREGATE GRADINGS

3.1 INTRODUCTION

Bituminous materials used in highway engineering are simply binary combinations of particles of mineral aggregates, and a binding agent which is usually a penetration grade bitumen. The majority of work associated with the design of bituminous mixtures has addressed the problem of identifying on optimum binder content, with much less effort concentrated on the influence of the aggregate gradation.

The structural strength of asphaltic concretes and coated macadams relies primarily on the friction, and mechanical interlock between aggregate particles. The addition of a binding agent to the gradation, such as bitumen, provides a lubricant which enables the material to become workable, allowing ease of compaction, and contributes to the final mixture properties.

The quantity of binder introduced to the gradation is critical, as too much, or too little will adversely effect the mixture properties, but it must be realised that similar consequences may result from variations in the aggregate grading.

The development of aggregate gradings for use in roadbase materials has been empirical throughout practice in the United Kingdom, resulting in the envelopes currently specified in BS 4987 (1). These gradings form very dense matrices of stone particles, and are analagous to the type of gradations which have been developed for asphaltic concretes in the United States of America. The grading used in asphaltic concretes was developed through a philosophy which aimed to maximise the density of the mineral aggregate, and is based upon a gradation curve suggested by Fuller and Thomson in 1907 (2).

3.2 THE FULLER CURVE

Historically, the best known system which describes continuously graded material is the 'Fuller' curve, which was established from the results of laboratory experimentation. A mathematical relationship was formulated relating adjacent particle sizes, and led to the development of a series of curves represented by the equation :

$$P = \left(\frac{d}{D}\right)^{n} \tag{3.1}$$

Where P = percentage of material passing a sieve of size dmm

D = maximum particle size (mm)

n = an exponent between zero and 1

By selecting a value of the exponent between 0.4 and 0.5, the aggregate gradation will achieve maximum densification (24), and it is this format of the Fuller curve which has become the standard grading for asphaltic concretes. The n = 0.45 curve forms the target gradation for use in asphaltic concretes, irrespective of aggregate type, and is generally considered to be the optimum curve for a continuously graded system of rock particles. However, it has been proposed that an individual gradation cannot represent all aggregate types, because of the different properties exhibited by different rock types (25, 26). Therefore, it should be necessary to design a gradation according to the source material.

3.3 GRADING DESIGN

3.3.1 Two component systems

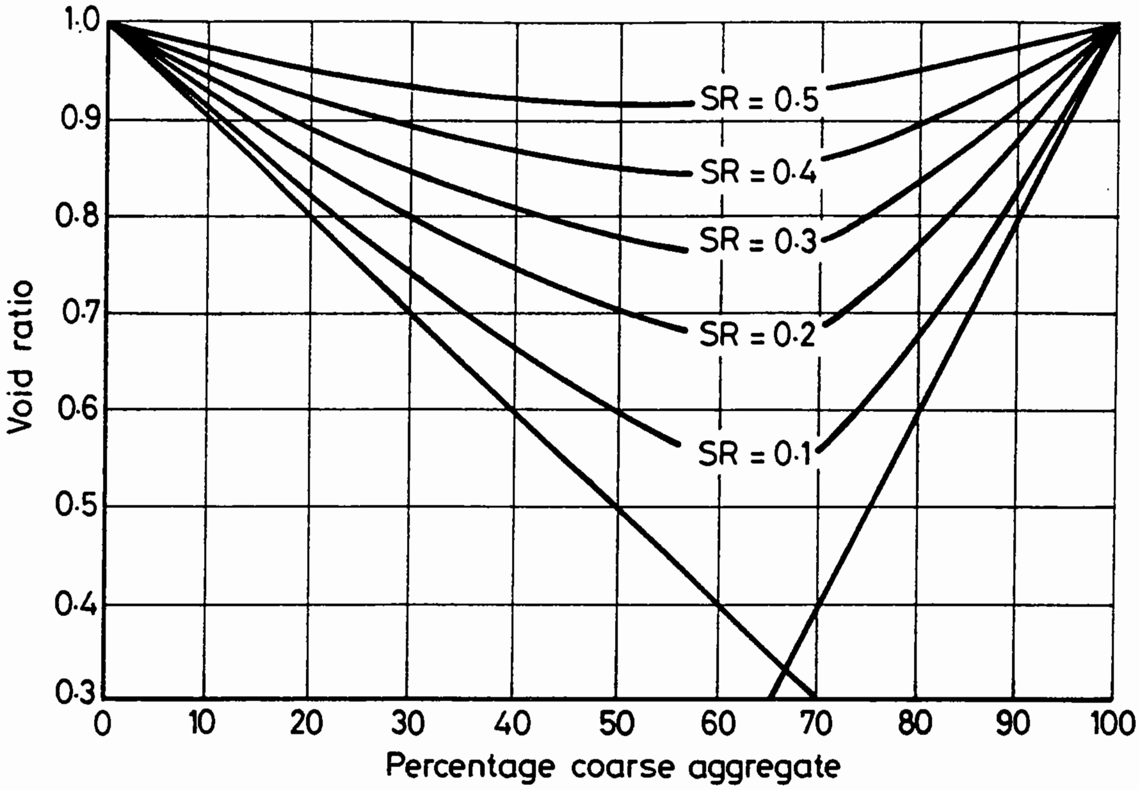
For any given system comprising two broken solids, each of single size, the void structure of the system will vary according to the combination of the two components. The first concentrated effort to study the effect of varying the combinations was made by Furnas in 1928 (27), who investigated the influence on voids of the proportion by volume of the coarse component of a binary mixture. Furnas demonstrated that a definite minimum level of voids could be achieved when a specific percentage of the coarse component was mixed with the fine component. The void content of this mixture was directly related to the size ratio of the two components, with the voids decreasing as the size ratio decreased, figure 3.1. A similar approach by powers (28) reconsidered the problem by expressing the void contents of the two components as void ratios, and plotting the relationship of the void structure within binary combinations on a void ratio chart.

The void ratio can be obtained from the equation :

Vr = \f(VMA,1 - V   3.2

where Vr = void ratio

VMA = voids in mineral aggregate



SR = Size ratio (small component/large component)

Figure 3.1 The effect of size ratio on the void ratio of binary aggregate mixtures. (after Furnas) (27)

The construction of the specific void ratio diagram is considered in Appendix A.

The specific void ratio diagram provides a facility of obtaining the optimum blend for lowest voids of a two component system. The only knowledge required of the mix constituents prior to blending is the void content of each component at the state of compaction at which blending would commence, and the size ratio relative to each other.

3.3.2 Multi-component systems

In order to use the specific void ratio diagram as a tool by which a continuous aggregate grading may be developed, the two component analysis must be extended into a system containing five, six or more constituents. This can theoretically be achieved by selecting the two coarsest components, determining the optimum blend, then considering this blend as a new 'two part' coarse component which is then combinded with a third finer constituent. New void ratios are calculated and the optimum 'three component' blend is achieved. Repetition of this process allows the introduction of a number of fine components finalising in a continuously graded mixture at maximum density. This is shown diagramatically in figure 3.2. Lees had used this procedure successfully for the design of gradings for asphaltic concrete wearing course mixes, and the same approach was adopted for roadbase gradings.

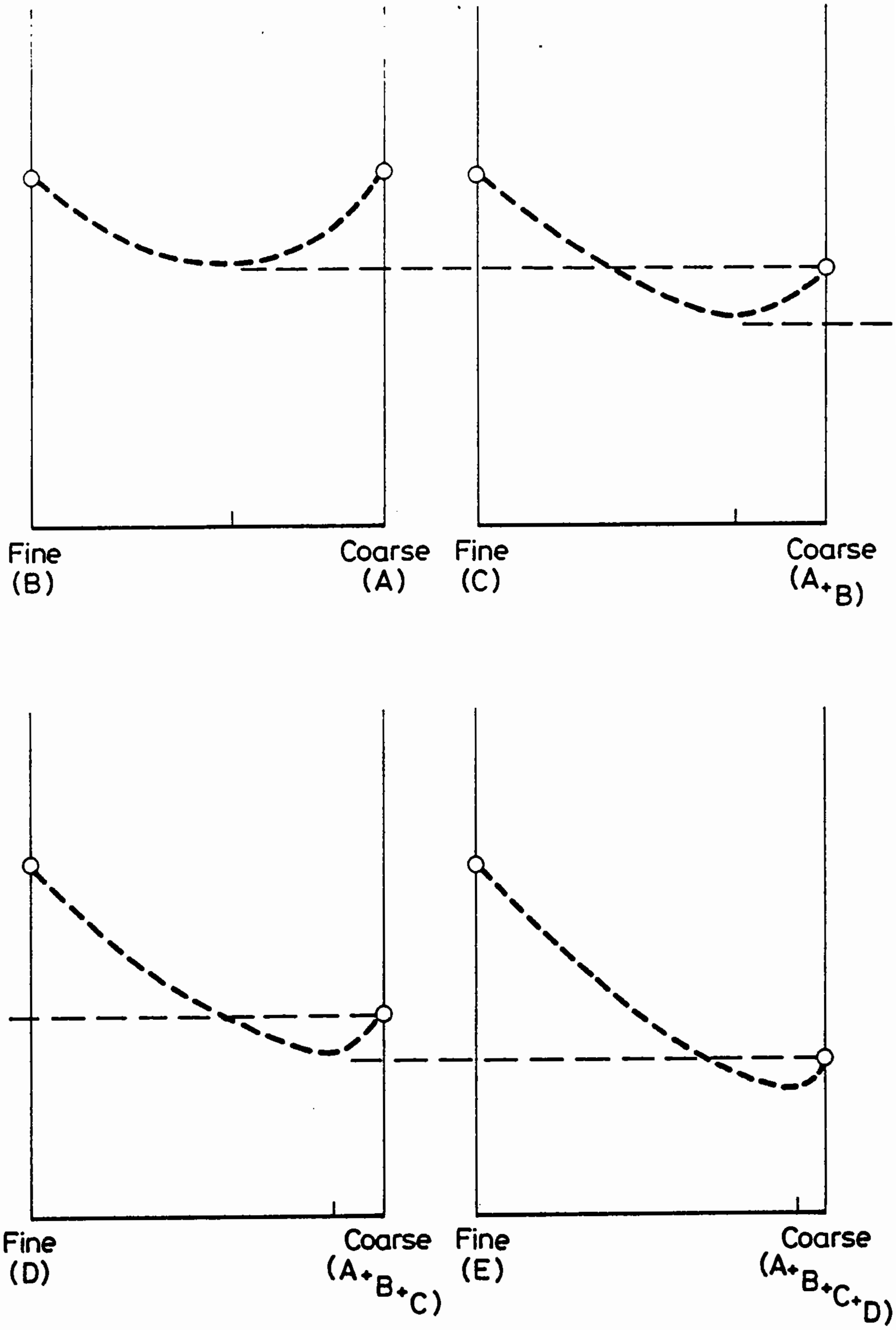


Figure 3.2 Illustration of the extension of a two component mix design into a multi component system. (after Lees) (26)

3.3.3 Design of a continuous grading for a basecourse layer.

By considering an aggregate from one source, it was anticipated that a design grading aimed at maximum density could be obtained from a sequential series of tests on the dry aggregate as previously described.

The material used in the tests was obtained from Bardon Hill Quarry in Leicestershire. The rock is a porphyritic andersite of igneous derivation which is classified as a granite within the construction industry. The rock is angular and carries little dust, figure 3.3 shows a photograph of the passing 20mm size fraction. Material was taken from the following bin sizes : Passing 28mm, 20mm, 14mm, 10mm, 6.3mm, 3.35mm and 0.075mm. Therefore, in order to design a continuous gradation, a seven component system must be developed.

3.3.3.1 Aggregate compaction tests

The apparatus employed for the operation of blending two aggregate size fractions, was an aggregate compaction cylinder and a vibrating table. The cylinder was of internal diameter 163mm and a height of 300mm. A plunger which formed a sliding fit within the cylinder was located centrally by a guide spindle, figure 3.4. The cylinder was then attached to the vibrating table, which was oscillated by an electric motor operating at mains frequency.

The principle of operation was the transmission of vibrations from the table to the aggregate matrix held within the compaction cylinder. The agitation of particles would cause a restructuring of the matrix resulting in

Fig 3.3.

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3.4

30 - 31.

Figure 3.3 Granite aggregate (Bardon Hill) passing 20mm retained 14mm.

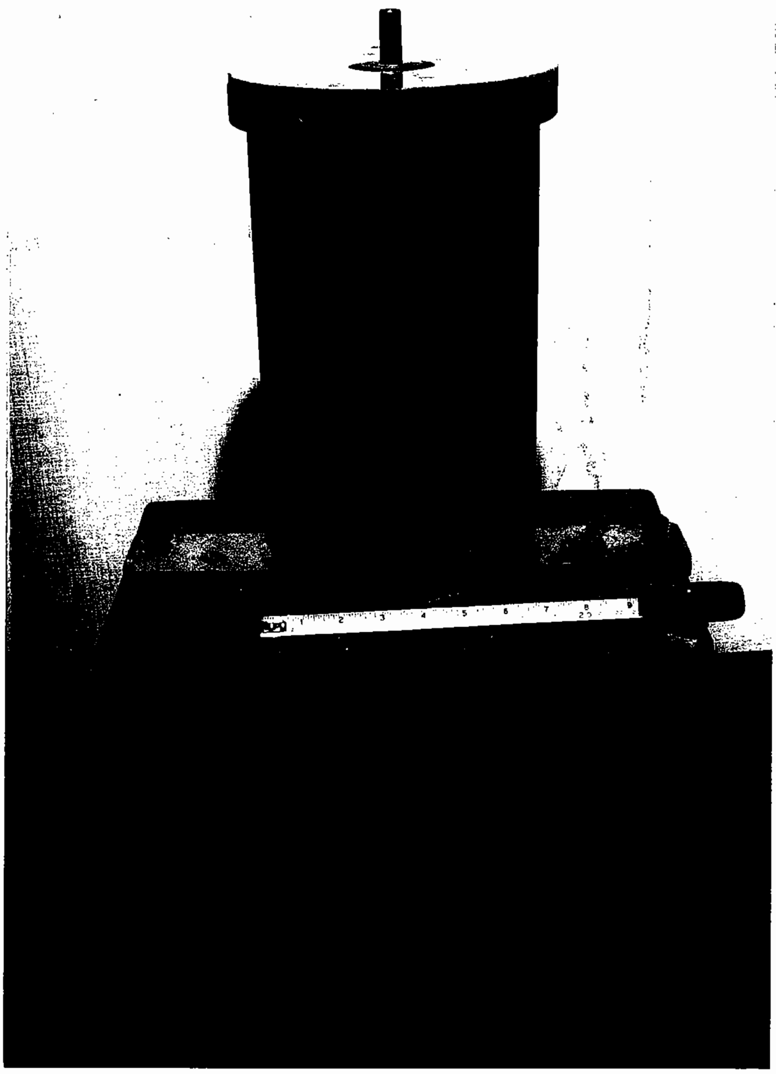


Figure 3.4 Small aggregate compaction cylinder.

a compacted mass of aggregate. The plunger should contribute a nominal surcharge, and prevent the loss of fine particles during the compaction process. Measurement of the depth of compacted aggregate was facilitated by a calibration on the spindle of the plunger.

Vibration was applied for a duration of forty five seconds for each operation. Initial experimentation indicated that there was no measurable increase in the compacted density of the aggregate following approximately thirty seconds agitation. Visible inspection of the procedure suggested that the majority of the restructuring process was quasi-instantaneous, with approximately 90% of compaction occurring inside the first few seconds of applied vibration.

Initially, void contents of each bin size were measured in order to obtain the datum void ratios for each size fractions. If the crushing process at the quarry created different sized rock fragments with identical angularities and flakiness indices, then each bin size should have the same void content when in a compacted state.

This is analagous to a fixed volume of ball bearings of diameter 10mm having the same void content as the same volume of 2mm diameter balls. The results are shown in table 3.1

Table 3.1 Void Ratio's of Material from Quarry bin sizes

Aggregate size fraction (mm)	28	20	14	10	6.3	3.35
Void Content (%)	49	47	45	43	43	48
Void Ratio $\frac{VMA}{1 - VMA}$	0.96	0.88	0.82	0.75	0.75	0.92

It can be seen that the larger size fractions exhibit a higher void ratio than the intermediate sizes. This is a result of the confining effect of the compaction cylinder preventing the aggregate from packing in its natural state. It was noted that this may create a problem when blending size fractions incorporating the larger sizes. There is also an increase in the void ratio for the smallest size fraction, material passing the 3.35mm sieve. This was considered to be due to the wide gradation inherent in this bin size, which contains seven sieve sizes. The material from this bin size was sieved into individual sieve sizes, and void ratio measurements of each size were taken.

Table 3.2 Void Ratio's of individual size fractions taken from material passing the 3.35mm sieve.

Sieve Size (mm)	3.35	2.36	1.18	0.6	0.3	0.15	0.075
Void Content (%)	43	43	43	43	43	40	38
Void Ratio $\frac{VMA}{1 - VMA}$	0.75	0.75	0.75	0.75	0.75	0.66	0.61

Table 3.2 confirms that the increased void ratio of 0.92 in table 3.1 is due to dilation of the aggregate particles through a random distribution of different sizes. Each individual size fraction has the same void ratio suggesting that the packing characteristics of each bin size from the quarry are the same. The very fine material (passing 0.15mm and passing 0.075mm) exhibited lower void ratios than the other sieve sizes. During the compaction tests it could be seen that for the finest material, a pumping action between the internal cylinder wall and the plunger took place causing a small quantity of the material to escape from the cylinder. Therefore when measuring the depth of the compacted material, an artifically low value was observed, resulting in the calculation of a depressed void content.

3.3.3.2 Bin size blending

Having obtained the void ratios of the material from each quarry bin size, a blending program commenced which would systemmatically incorporate each bin size to result in a continuous gradation. From the compaction tests using the 28mm aggregate where the edge effects from the cylinder had a significant influence on the void ratio of the material it was decided to try and design a grading with a maximum stone size of 20mm.

The procedure concentrated on size ratios of 0.5 or higher, commencing with a blend of 20mm and 14mm aggregate. 10mm aggregate would then be introduced to the blend giving the lowest void ratio, and the procedure as shown in figure 3.2 would be followed until all sizes were included. The initial blending tests highlighted the limitations of a small diameter cylinder. Figure 3.5 shows how it was impossible to distinguish an optimum blend for minimum void ratio when using these size fractions. Repeated tests gave the same spread of results.

To proceed with the testing a new compaction cylinder was manufactured, maintaining the same configuration as the original model but having larger dimensions. A section of steel pipe of external diameter, 260mm long was reamed out to give an internal diameter of 308mm and an internal plunger was machined to give a sliding fit, figure 3.6. Void ratios of the different bin sizes were obtained using the new apparatus.

Table 3.3 shows that the increased capacity of the compaction cylinder enabled all the bin sizes to be satisfactorily compacted.

Table 3.3 Void ratios of Quarry bin sizes using large Compaction cylinder

Aggregate Size Fraction (mm)	28	20	14	10	6.3	3.35
Void Content (%)	43	42	42	41	42	42
Void Ratio $\frac{VMA}{1 - VMA}$	0.75	0.72	0.72	0.69	0.72	0.72

Table 3.3 shows that the increased capacity of the compaction cylinder enabled all the bin sizes to be satisfactorily compacted.

The sequence of blending a binary mix which was adopted throughout, was to commence with a 20% coarse and 80% fine aggregate combination,

Fig. 3.6

34-35

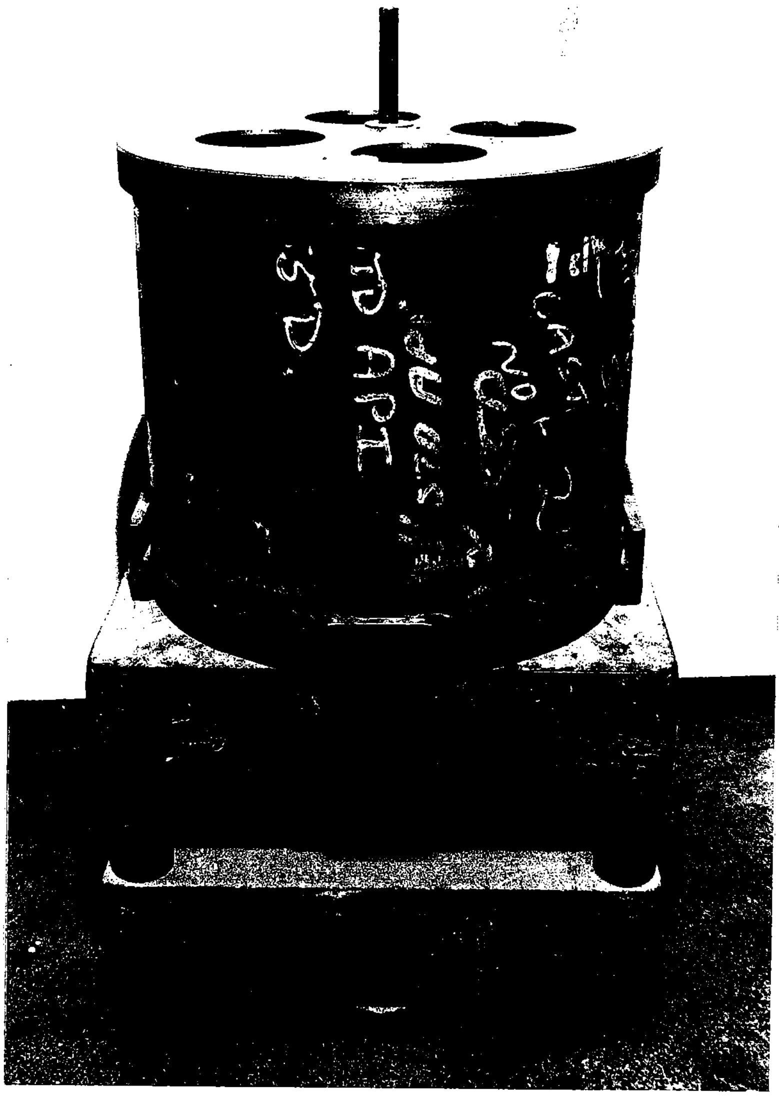


Figure 3.6 Large aggregate compaction cylinder

working through at 10% increments to a 20% fine and 80% coarse aggregate mix. Each mix was introduced into the cylinder in several layers so that whichever component represented the least percentage, it would be evenly distributed throughout the other aggregate, i.e. the blend was given a pre-mix to achieve a degree of homogenity prior to vibration.

A fundamental problem associated with this approach to the design of basecourse gradation soon became evident. Although the prelimary mix of 28mm and 20mm aggregate gave an optimum mix composition of 50% of each component (figure 3.7) the introduction of the 14mm bin size to the blend began to yield indistinguishable results in that it was impossible to identify the optimum blend. The mixes always involved high size ratios between the two components, resulting in void ratio relationships approaching the horizontal line representing a size ratio of 1. Adherance to the idealised method proposed by Lees in figure 3.2 could not be made using the chosen approach. The problem cannot be overcome by commencing with a low size ratio, for example blending 28mm aggregate with 6.3mm aggregate, as this results in a mixture with a mean effective particle diameter of about 16mm. Therefore when a third aggregate is intoduced to the blend, the new size ratio will be increased resulting in difficulty in assessing the minimum void ratio.

3.3.4 A model grading

One of the factors which influenced the imprecise results obtained from the dry aggregate compaction tests was considered to be the packing characteristics of irregular shaped rock particles. Ishai and Tons (29) attempted to quantify this parameter by the concept of packing volume to define the geometric irregularity of aggregate particles. The packing volume was defined as the volume a rock particle occupies is a mass of single sized particles and was termed the percent specific rugosity with values approaching zero representing uniform smooth spheres. The value of precent specific rugosity was obtained from a pouring test where a single sized aggregate of unknown packing characteristics was compared against a standard, (glass beads) with known packing characteristics. Each particle type was poured through a funnel into a container, and from knowlege of the material specific gravities, the packing density was found and related to the specific rugosity value given by :

$$Srv = 100 \left[1 - \frac{Gp}{Gap}\right] \qquad 3.3$$

Where Srv = specific rugosity value

Gp = packing specific gravity (from pouring test)

Gap = apparent specific gravity

In general the values of Srv reflect the irregularity of the aggregate particles, those which are angular, elongated or flaky usually possess higher values of Srv. These aggregates will give rise to higher VMA's in a compacted state (30) and will therefore be most difficult to use in dry aggregate compaction tests. In the context of designing an aggregate gradation, angular aggregates, or those possessing a high value of specific rugosity have been shown to give high Marshall stability values in a bituminous mix (31, 32). Hence a well designed grading using such an aggregate should produce a high quality bituminous paving mixture assuming the binder content could be optimised.

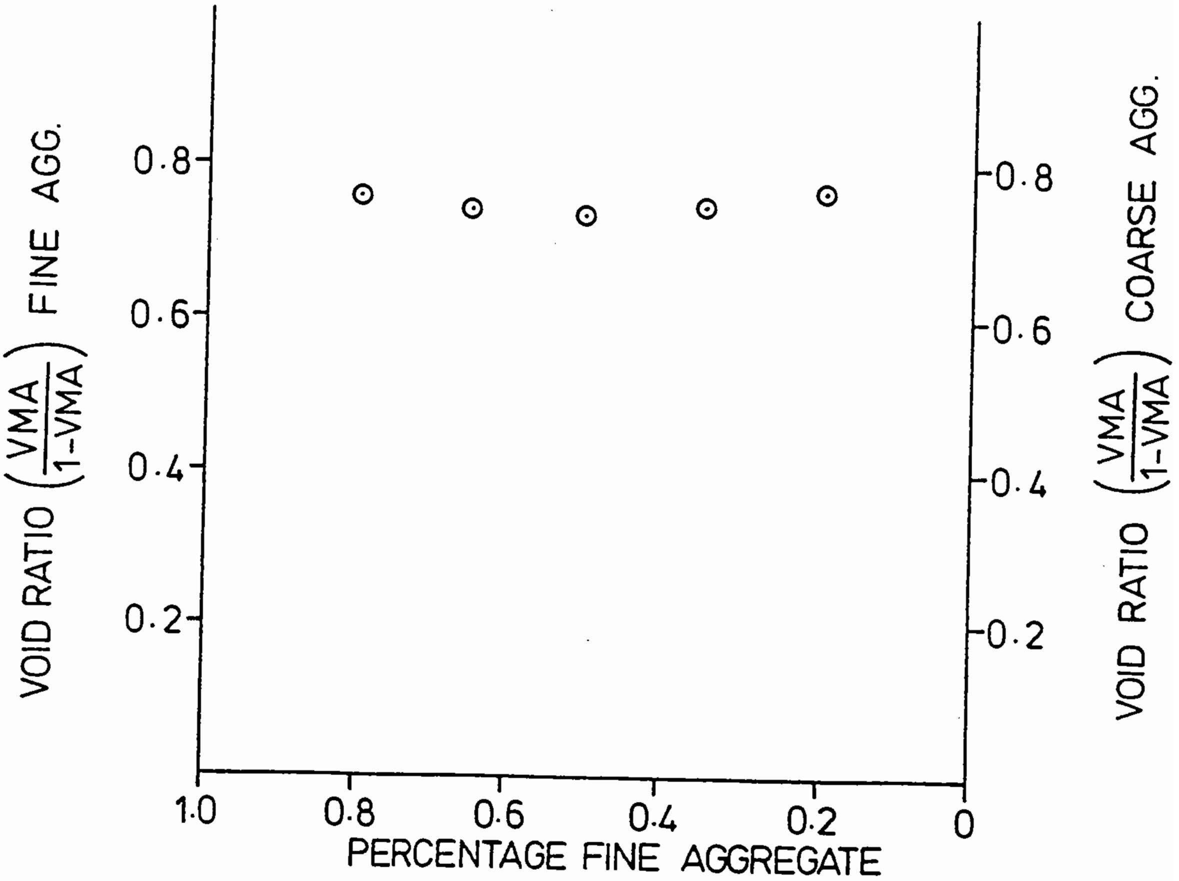


Figure 3.7 Void ratio plot showing relationship between binary combinations of coarse aggregate component (28mm) and fine aggregate component (20mm) using the large compaction apparatus.

By selecting a material which has a specific rugosity value of zero and blending different sizes of this material, it may be possible to develop a model for continuously graded aggregates. This approach was investigated using Lees' method and utilising spherical glass beads of known diameter and specific gravity. Results for a varity of size ratio combinations could be obtained from the vibrating cylinder apparatus which could be used as a datum to which aggregate particles could be related.

The spheres used were "Ballotini Beads", small glass balls which were imperfect spheres but, when randomly placed on a suface giving a layer thickness of one diameter and agitated, would exhibit a hexagonal packing arrangement as true spheres. The beads had the following diameters : 1mm 2mm 3mm 4mm 5mm and 10mm. Figure 3.8 shows a photograph three different bead sizes. Void contents of each bead size were obtained using the large compaction cylinder. All sizes exhibited similar packing characteristics with a typical void content of 36%, about 6% lower than that obtained from the granite aggregate ideal tetrahedral packing of uniform spheres can be shown to result in a void content of 25.9% (33) demonstrating the randomness with which the single sized ballotini pack together. The initial blending programme using the beads gave promising results when combinations of low size ratios were used (figure 3.9a) but when high size ratios of spheres were compacted the problem of an indistinct minimum void ratio recurred. Only after repeating the test a number of times could a relationship be seen for a combination of 10mm and 5mm beads, (figure 3.9b) which showed very little deviation from the size ratio = 1 line.

Fig 3.8.

37-38

before Fig 3.9b

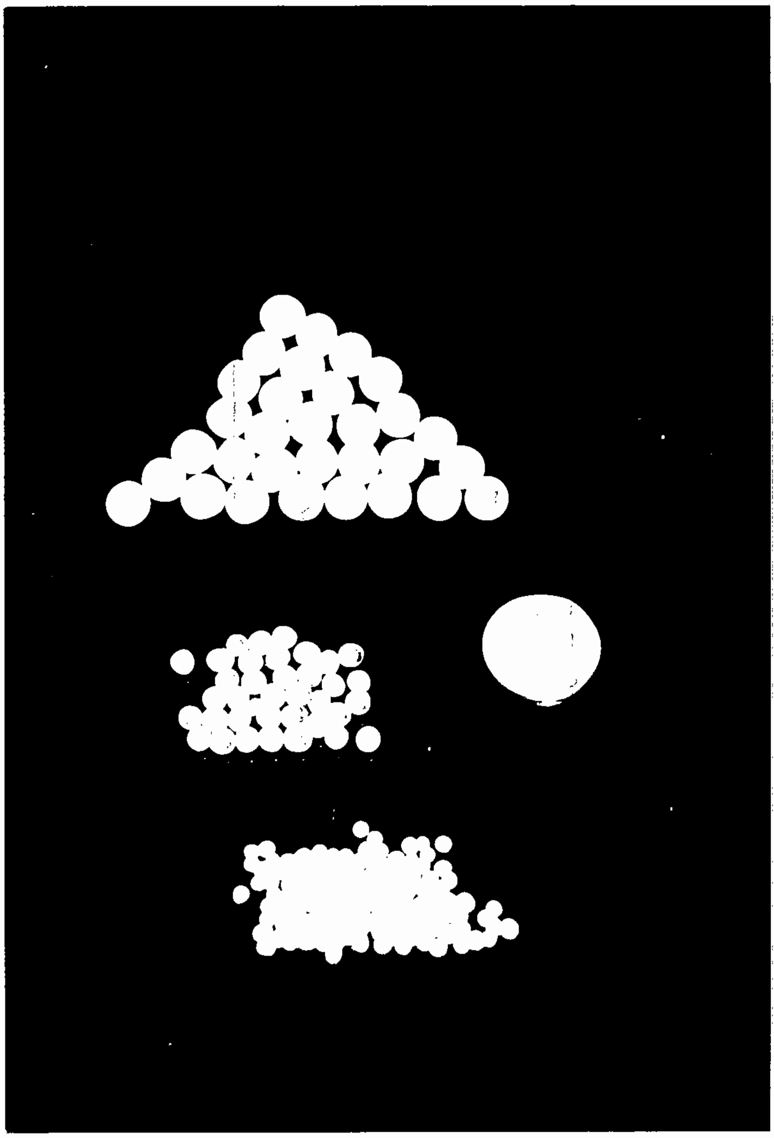


Figure 3.8 Glass beads for use in the development of model grading.

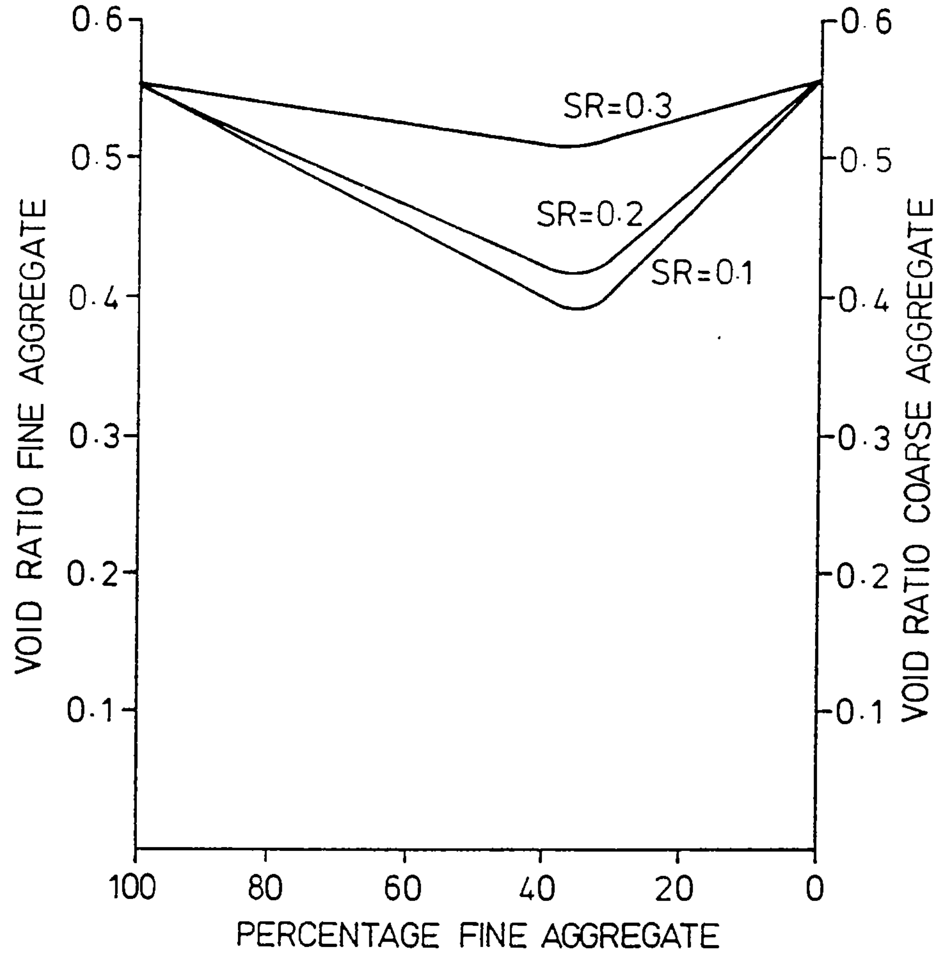


Figure 3.9a Void ratio plot of binary combinations of glass beads for size ratios of 0.1, 0.2 and 0.3.

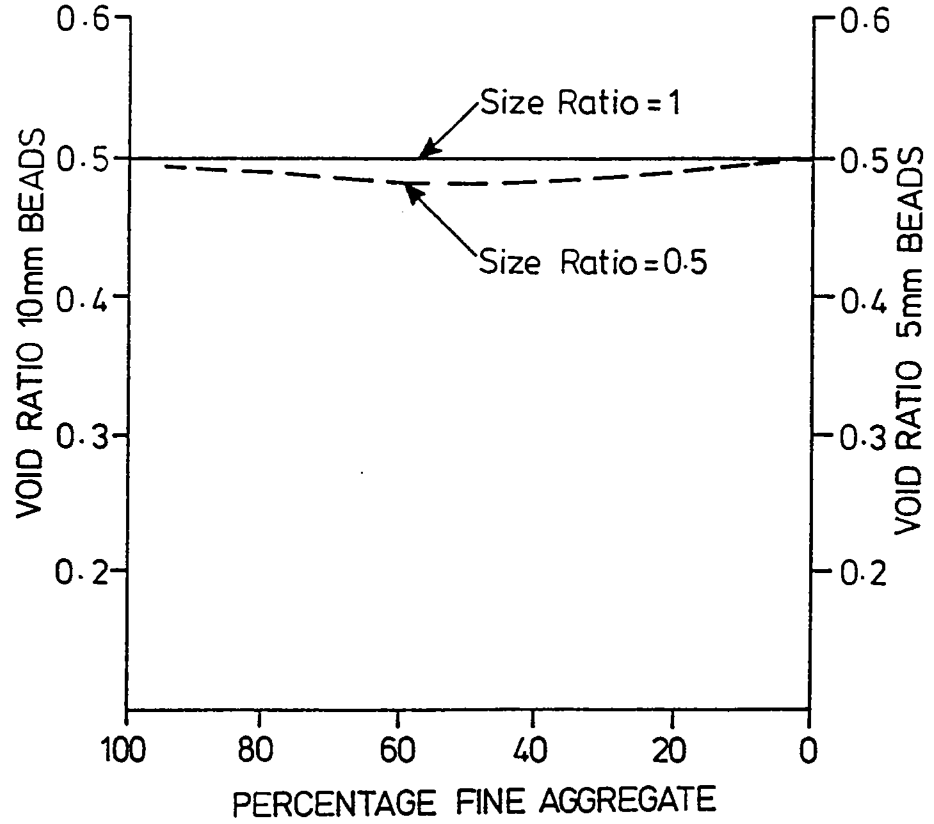


Figure 3.9b Void ratio plot of binary combination of glass beads for size ratio of 0.5.

3.3.5 Frequency response

A possible reason for the difficulty in identifying minimum void ratios, particulary in blends with high size ratios, was that full compaction was not achieved with the apparatus used. When a continuous grading to BS 4987 table 3 was vibrated in the cylindrical mould, it was found that there was a shallow hollow in the compacted material at one side of the cylinder. It was concluded that the eccentricity of the motor caused a steady migration of material to one side of the cylinder resulting in a falsely high value for the measured bulk volume of the aggregate.

Lees carried out many tests quite successfully to obtain void ratios of compacted aggregate using similar equipment to that employed at Nottingham. The fundamental difference between the two sets of experiments was that Lees' based his testing on much thinner layers of 40mm, comparted to the 100mm depths of aggregate representative of basecourses used in this project. Lees also commenced his compaction procedure with a high amplitude vibration, which agitated the full depth of the aggregate, then reduced the amplitude through the compaction sequence until finally there was just sufficient vibration to agitate small particles on the surface of the layer. The apparatus used in the investigation performed as part of this project had a fixed vibration amplitude preventing the adoption of Lees' method. In order to change the amplitude of the vibration different equipment would be required capable of transmitting an agitation to 15kg of loose aggregate. The combined mass of the steel cylinder and the aggregate generated such inertia that a substantial amplitude would be required to vibrate the stone matrix. A more rational solution would be to adjust the frequency of vibration to the resonant frequency of the aggregate particles. An investigation into this theory was performed using an Instron servo hydraulic testing machine, and it was found that the greatest particle agitation occurred at around 115Hz. Optimum frequencies for compaction of bituminous materials lies between 35 and 50Hz (34) with an amplitude of 0.4 to 0.8mm, which is substantially different from the resonant response observed for dry aggregate, although a bituminous binder would damp the particle oscillations. In order to vibrate a large cylinder containing a mass of aggregate of 15kg at 115Hz would require a sophisticated and expensive piece of apparatus, the development of which was beyond the scope of this project. The mechanical problems asociated with high amplitude, high frequency vibrations are many, particulary the limitations of flow of hydraulic oil through a controlling servo-valve suggesting that such a rational approach to dry aggregate compaction tests would not be practical, and be commercialy unviable.

3.4 AN ASSESSMENT OF CONTINUOUS GRADATIONS

Brown and Cooper (14) investigated the effect on the mechanical properties of bituminous mixes through modest changes to the grading specifications (1) and concluded that significant improvements to UK recipe mixes could be achieved by relatively small adjustments to the specifications. These primarily involved the substition of a harder grade of binder to the existing grading envelope, which successfully improved the elastic stiffness and the fatigue life of the mix. Deformation resistance was found to be unchanged by such modification. Recognition of the role of the aggregate and the importance of VMA led to further research into improving the performance of roadbase mixtures through grading design (35).

Cooper et al investigated nine transitional aggregate gradings of 20mm maximum particle size from hot rolled asphalts to open textured macadams. The gradings were quantified by the values of VMA for each mix and by the percentage of material passing the 0.6mm sieve. This conveniently divided each gradation into a binary mixture of rock particles. From this datum, plotting the nine mixes on a void ratio diagram showed the mix with a 30% fines content to have minimum VMA (figure 3.10). Earlier investigations by Hveem (25) included an assessment of a series of gradings which had exhibited good performance in the field. Hveem stated that "all of these gradations were developed by different individuals working independently and separated by considerable distance and time, and each one respresents the most ideal combination which was developed after a great many trials and consideration of other combinations". Hveem observed that for all of these ideal gradings 31% of the material passed a sieve of size 0.031 times the maximum sieve size of the gradations.

3.4.1 The concept of an "Equivalent Fines Content"

From the observations made by Hveem, Cooper et al concluded that if a series of individual aggregate gradings pass through a common intersection point on a size ratio grading curve, i.e. sieve sizes expressed as a fraction of the largest size, then those gradings should have similar VMAs close to the minimum possible. This would provide a means of predicting the VMA of a mix from the aggregate grading. A similar

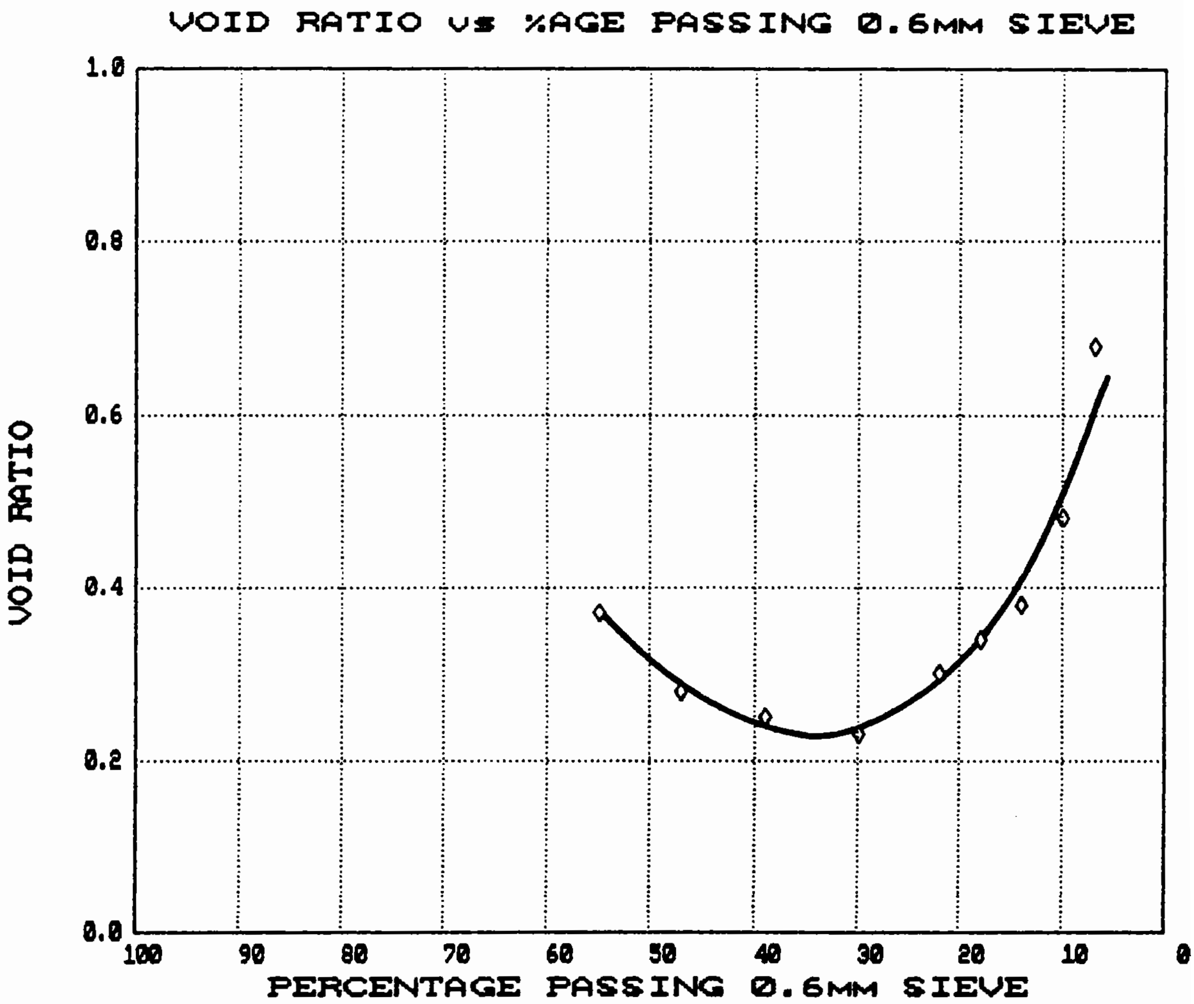


Figure 3.10 Variation of void ratio with percentage passing the 0.6mm sieve for the nine transitional mixes. (after Cooper et al)

approach had been made by Hudson and Davies (36) who made a range of gradings with different maximum aggregate sizes. They found that the VMA of the mixes was linked to the quantity of material passing a sieve having a size equal to 0.032 times the maximum size, and that Marshall stability values followed a similar association. The value corresponding to the quantity of material passing such a sieve has been termed the "equivalent fines content", the effect of which is shown by the results of Hudson and Davies' work in figure 3.11. The plot shows that minimum VMA and maximum Marshall stability are coincident with an equivalent fines content of about 20%.

Brown et al performed a comparison between continuous and gap gradings whilst maintaining an equivalent fines content of 30% for each paricular gradation. It was found that the VMA for each grading remained constant for a given compactive effort, confirming the relationship between volumetric proportions and the grading intercept point. The gradations investigated, and the measured VMAs are shown in figure 3.12. The effect of an optimum equivalent fines content on the mechanical properties of bituminous mixes was then given consideration. A series of mixes was manufactured using a range of aggregate types, filler contents and binder contents at different gradations, and it was found that the minimum VMA coincided with an equivalent fines content of 30%. Performance was indicated by elastic stiffness values of the mixes which were measured using a repeated load triaxial test, and these were found to peak at the same equivalent fines content.

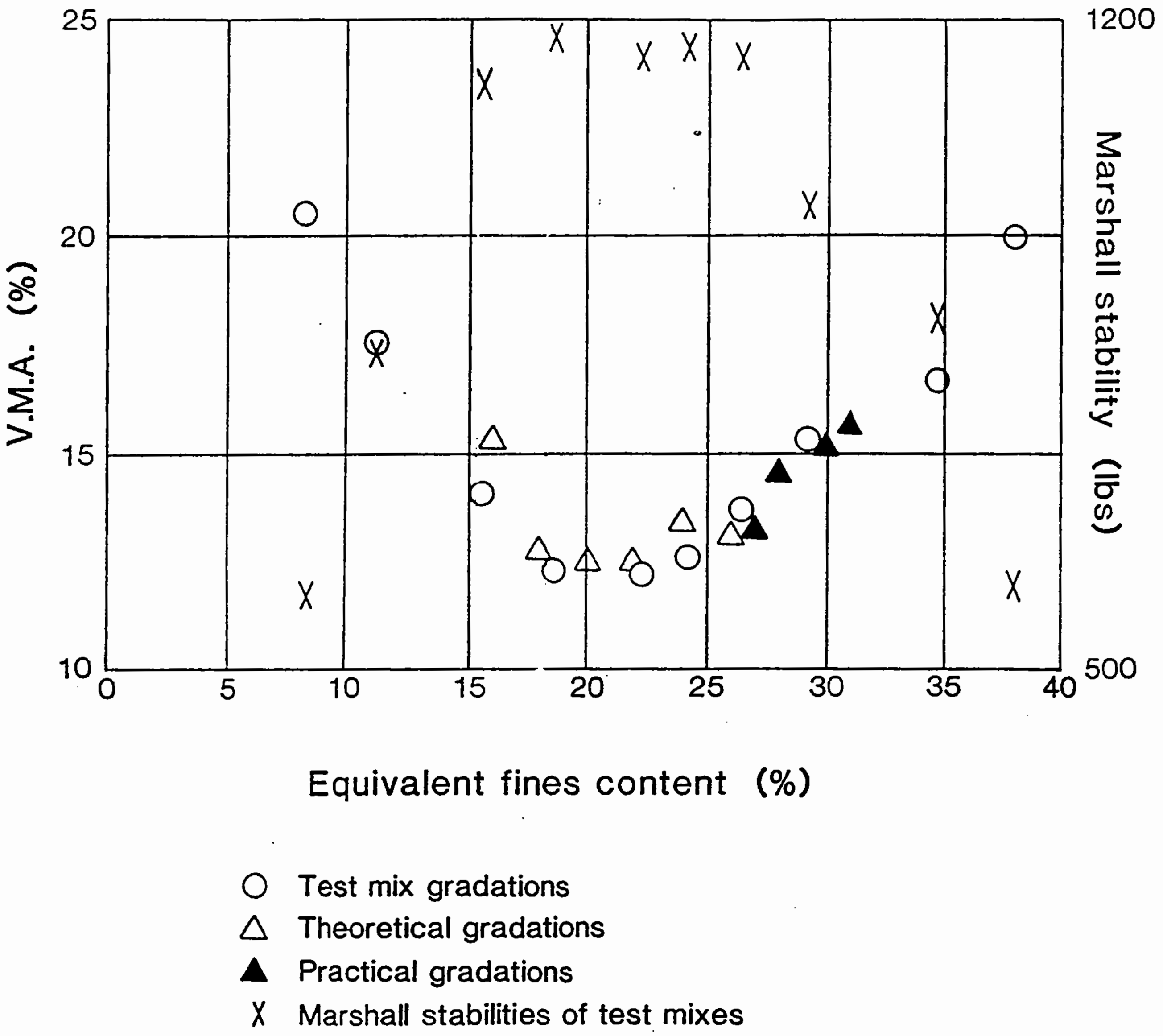


Figure 3.11 Variation of VMA and Marshall stability with equivalent fines content. (after Hudson and Davies)

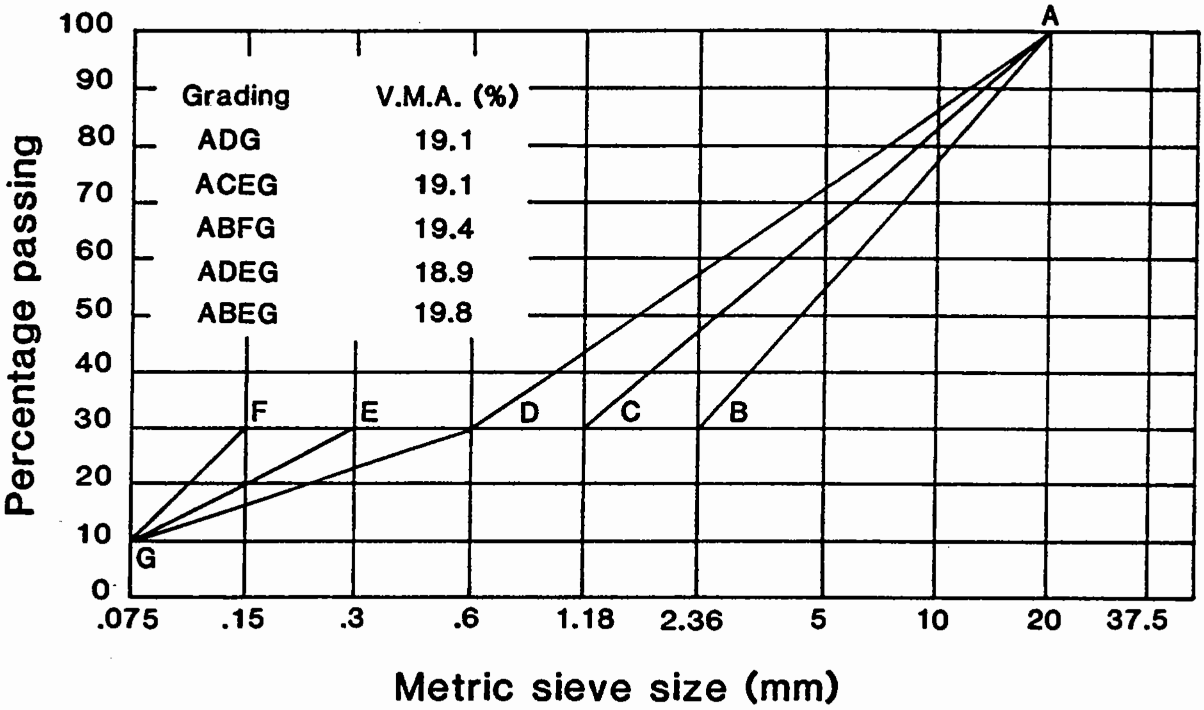


Figure 3.12 The effect on VMA of using various gap gradings having a common intersection point. (after Cooper et al)

3.4.1.1 Pracical implications

This chapter has dealt with the role of the aggregate grading in a bituminous mixture and the influence on the mechanical properties which dictate the ultimate performance of the mix. Although empirical practice has resulted in continuous gradations which form a dense matrix of rock paricles with a low VMA, there still remains to be shortcomings in the attention and scrutiny which is given to this part of the mix. BS 4987 specifies two checks on the gradation for material passing the 3.35mm sieve. These checks occur at the 300 micron and 75 micron sieves, both of which accomodate wide tolerances, resulting in minimal attention being given to approximately 35% of the rock in the grading.

The attempts at designing an aggregate grading to specific volumetric proportions through a series of dry compaction tests, principally investigated by Lees (26) and by the author as part of this project, either yielded unsatisfactory results or gave a procedure which did not lend itself to practical implemenation within industry.

The concept of an "equivalent fines content", characterising a grading intercept point would provide a technique for quantifying a particular gradation interms of minimum VMA. The work by Cooper et al indicated a relationship between VMA and permanent deformation of the total mix, and work elsewhere (37) suggests that maximum deformation resistance is associated with a minimum VMA. It could be proposed that the existence of maximum density gradings, such as the Fuller curve are all that is necessary to achieve an aggregate gradation with a near minimum VMA, but as described earlier the situation is complicated by aggregates from independant sources having different packing characteristics. The problem is further complicated by the degree of compaction of the material and the thickness of the layer under consideration. This becomes significant in the coarse component of the mix, for as layer thickness is reduced the packing of the coarse fraction becomes inhibited increasing its' void ratio thus increasing the percentage of fines required for minimum void ratio.

These factors highlight the importance of careful consideration of the aggregate grading. It is not satisfactory to simplify the situation and utilise wide grading envelopes, or permanently target on the Fuller curve, as these approaches do not cater for the variable properties of aggregates from different sources. Brown et al concluded that for a given aggregate, a series of gradings should be made up with equivalent fines content values incrementally increased from 20% to 30%, and the VMAs at each value obtained. The gradation with an equivalent fines content corresponding to minimum VMA should be selected and the mixing plant target on equivalent fines content made at 90% of the value obtained in the laboratory. This would ensure that the mix is not susceptible to over compaction in the field.

The thickness of the asphalt layer should also receive consideration in laboratory work and care should be taken to ensure that in-situ conditions are reproduced in terms of specimen dimensions compared with layer thicknesses in the field.

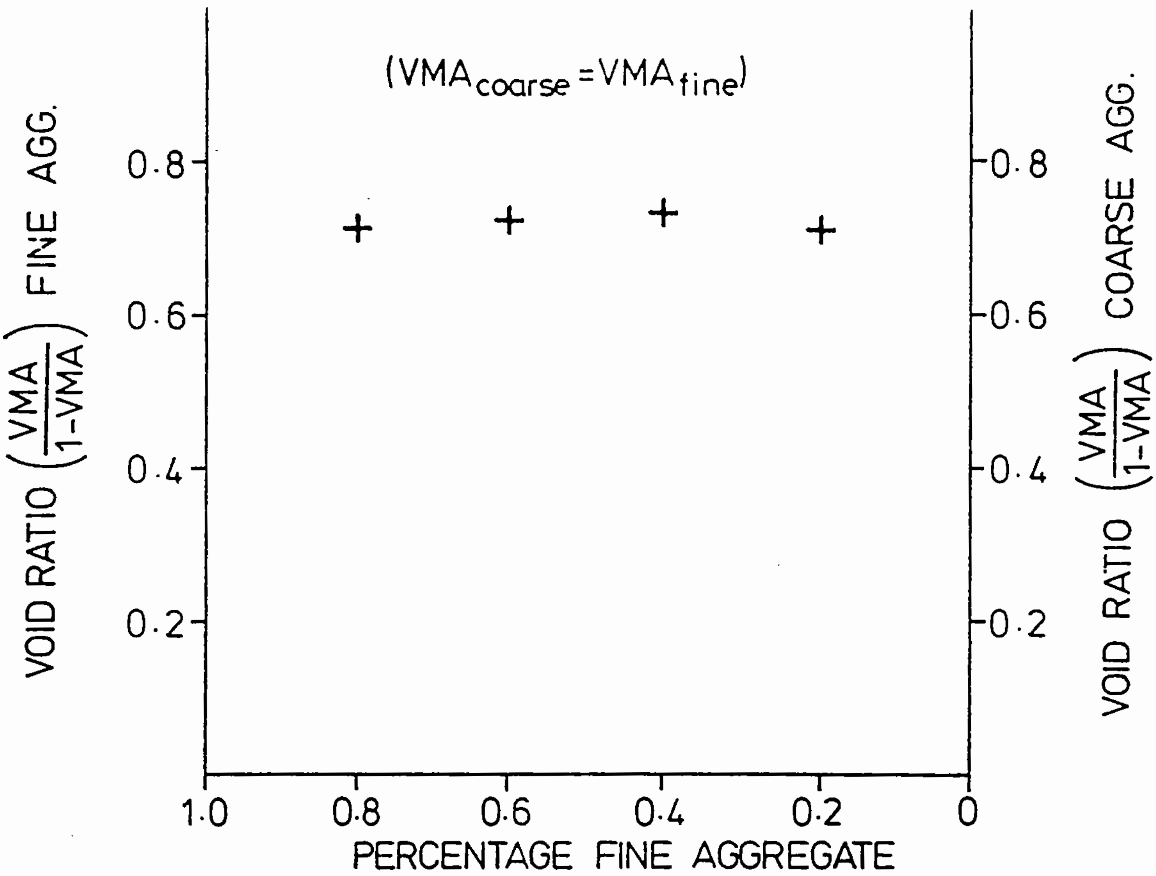


Figure 3.5 Void ratio plot showing relationship between binary combinations of coarse aggregate component (20mm) and fine aggregate component (20mm) using the large compaction apparatus

CHAPTER FOUR

THE MECHANICAL PROPERTIES OF BITUMINOUS MIXTURES

4.1 GENERAL

The purpose of designing any material, be it reinforced concrete, steel, plastics or bituminous concrete, is to ensure that the final product of the production process is capable of satisfying the demands placed on that material throughout its working life. This requires isolating the salient parameters which may cause the material distress, and identifying the inherent material properties which must be enhanced to prevent any distress mode reaching failure point.

The principal mechanisms which form the primary distress modes of bituminous pavements are the accumulation of permanent strains under application of loading, and the repeated tensile strains, which are greatest at the base of the asphaltic layers. The two modes manifest themselves as vertical deformations and cracks, respectively. The vertical deformations occur in the wheel tracks of the carriageway lanes which are predominantly used by commercial vehicles, and in an advanced state, the ruts can be clearly identified by the naked eye.

The cracking phenomenon is generally the result of repeated applications of a tensile strain at the underside of the bound material, and the subsequent propagation of small cracks to the road surface. This can be seen as 'alligator cracking', or an interwoven series of discontinuities on the highway surface if allowed to develop. Some cracks can be initiated on the road surface through a combination of rolling induced cracks, thermal contraction of the surface material, and stress generated through traffic action. These cracks may subsequently propagate downwards through the wearing course layers contributing to further pavement distress.

Even with high quality workmanship at the construction stage, and rehabilitation and maintenance programmes, all well trafficked roads would eventually exhibit one, or both of these distress modes. In UK practice a road is classified as having failed when a measured rut depth of 20mm, or extensive wheel track cracking can be observed (38), which would necessitate pavement reconstruction. To avoid this senario, a critical condition has been defined as a 10mm rut, or the first signs of wheel track cracking (39), which would signify a rehabilitation programme rather than full reconstruction. The art of mix design of bituminous mixtures is to mobilise the material properties to their optimum performance in order to combat the aforementioned distress modes. There are principally three mechanical properties which have been identified as being of paramount importance for the performance of bituminous materials:

1) Elastic Stiffness
2) Resistance to fatigue cracking
3) Resistance to permanent deformation.

To this list must be added durability and workability which, although not fundamental mechanical properties, still contribute to the performance of the pavement. Mix characteristics must be quantified in terms of these properties, and a knowledge of the effect of mix variables on the properties is necessary.

4.2 ELASTIC STIFFNESS

The elastic stiffness of a bituminous layer indicates the ability of the layer to spread load, and reduce the stress per unit area transmitted to the underlying layer. As a generalisation, the elastic stiffness can be thought of as analagous to the Young's Modulus of a perfectly elastic material such as steel.

The value of stiffness of a bituminous material is complicated because bitumen exhibits visco-elastic and thermo-plastic characteristics, which in turn, influences the behaviour of the mix. Whereas an elastic material will have a fixed response to a load of given magnitude, a bituminous material will have a variable response, dependent upon the time of loading and the temperature of the material during the loading process. The elastic stiffness of a bituminous mixture represents the stress-strain relationship of the material under loading conditions which may be considered to generate a predominantly elastic response.

4.2.1 Binder properties

The properties of bituminous mixtures are largely dependent upon the properties of the binder. Therefore, it is necessary to introduce some information concerning the characterisation and rheological properties of bitumen. The bitumens which are used in highway engineering are classified by hardness, or penetration grade, which are obtained through empirical tests. In the United Kingdom, BS 2000 pt 49 (40) describes the penetration test as the distance in tenths of a millimetre that a standard needle weighted to 100 grammes, will penetrate vertically a sample of bitumen at 25°C in five seconds. The 'Ring and ball softening point' test (41) also accounts for the hardness of the binder as it quotes the temperature to which it is necessary to heat the bitumen to achieve a certain viscosity. The test determines, therefore, an equiviscous temperature, which means that irrespective of the grade of bitumen tested it will have the same viscosity as any other grade at the end of the test. The viscosity at the end of the softening point has been shown to be equivalent to that of a penetration of approximately 800 (42).

All bitumens display thermoplastic properties, which means they become softer when heated and harder when cooled. This phenomenon is known as 'temperature susceptibility' and is characterized by the Penetration Index (P.I) after Pfeiffer Van Doormaal (43).

The Penetration Index can be calculated from the logarithm of the penetration and the softening point temperature using the equation:

$$PI = \frac{20\,(1 - 25A)}{1 + 50A} \qquad 4.1$$

where $$A = \frac{\log(800) - \log(P\ @\ 25^{\circ}C)}{SP - 25^{\circ}C} \qquad 4.2$$

To describe the engineering properties of bitumen, Van der Poel (44) introduced the term 'stiffness' as the ratio of stress to strain for a particular loading over a certain time and temperature. His work resulted in a nomograph enabling the determination of the stiffness for any bitumen provided the penetration, softening point and penetration index are known, (figure 4.1)

4.2.2 Prediction of mix stiffness

The concept of binder stiffness has been extended to bituminous mixes, based on the work by Van der Poel. The term 'elastic stiffness' has been introduced to describe a modulus value, and this has been found to be primarily dependent on the bitumen stiffness and the void content of the mix (44,45,46) The Shell organisation extended the studies and produced a nomograph for the prediction of mix stiffness, utilising the binder stiffness and volumetric proportions of the mix (47). Based on the Shell work, Brown (48) established the following equation for the determination of mix stiffness for void contents greater than 3% and a binder stiffness in excess of 5MPa, at a given temperature and time of loading.

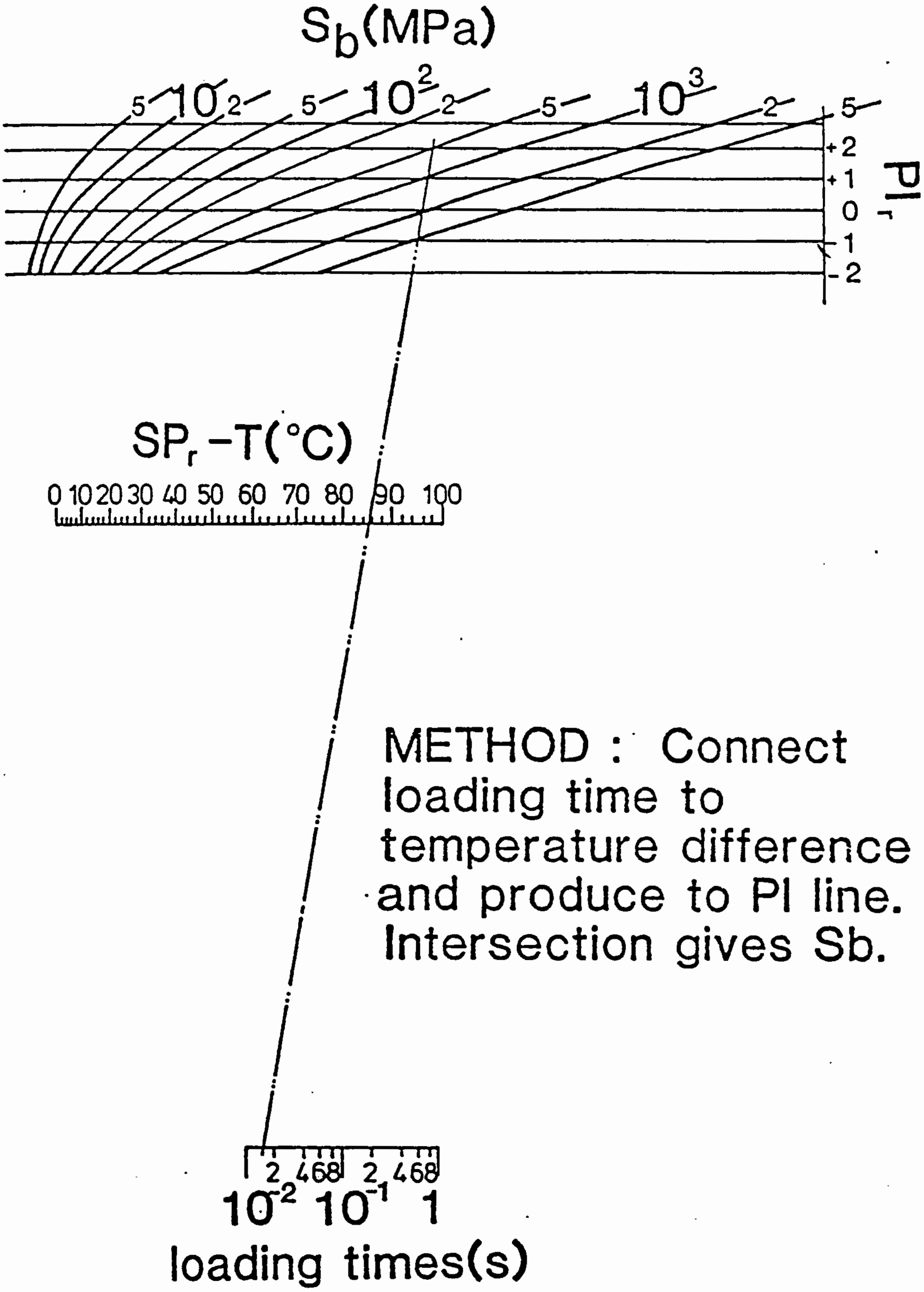
$$Sm = Sb\left[1 + \frac{257.5 - 2.5VMA}{n(VMA-3)}\right]^n \qquad 4.3$$

where Sb = binder stiffness

VMA = voids in mineral aggregate (%)

$n = 0.83 \log\left[\frac{4x10^4}{Sb}\right]$

The equation can also be presented in chart form, as shown in figure 4.2. Hence, for the typical conditions of UK pavements, the elastic stiffness values of the bituminous materials can be estimated without performing experimental tests.



$S_b = 1.157 \times 10^{-7}\ t^{-0368}\ 2.718^{-PI_r}\ (SP_r - T)^5$

Applicable over limited range

Figure 4.1 Nomograph for deriving bitumen stiffness
(Taken from the Van der Poel Nomograph ref 42.)

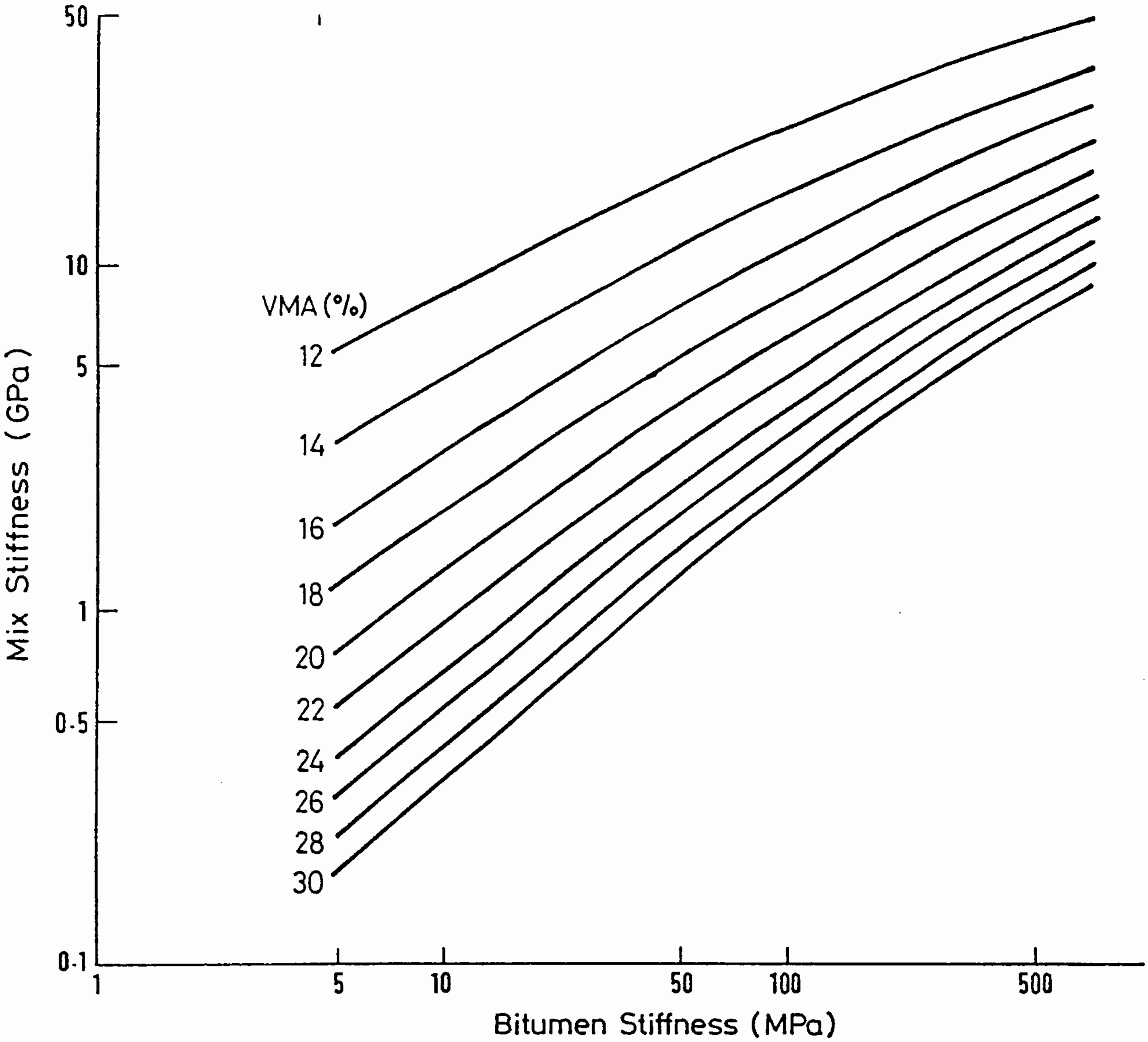


Figure 4.2 The relationship between mix stiffness, binder stiffness and VMA in the elastic region. (after Brown) (106)

From inspection of the nomograph it can be seen that for a particular bitumen stiffness, the mix stiffness increases as VMA decreases, indicating the importance of the proportions of the mix components and compaction.

4.3 RESISTANCE TO FATIGUE CRACKING

The phenomenon of fatigue cracking is the fracture of the material through a repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material.

This effect leads to one of the primary distress modes of bituminous pavements. The tensile strains generated in the bituminous layers are greatest at the underside of the bound material, and it is at this location where crack initiation can take place. For a given loading, temperature of material and layer thickness, the magnitude of the induced tensile strains are related to the elastic stiffness of the bituminous layer. A material with a high value of elastic stiffness will possess good load spreading qualities, reducing the magnitude of the tensile strains at the underside of the layer retarding the onset of cracking. However, the stiff material will generate large tensile stresses, proportional to the loading, within the asphaltic layer, which will result in rapid crack propagation through the layer after a discontinuity is established at the asphalt. The relationship between the critical stresses, and elastic stiffness is shown in Figure 4.3. Materials with low elastic stiffness values will exhibit more rapid crack initiation under the same loading conditions, but crack propogation to the surface of the bound material will be a longer process. These

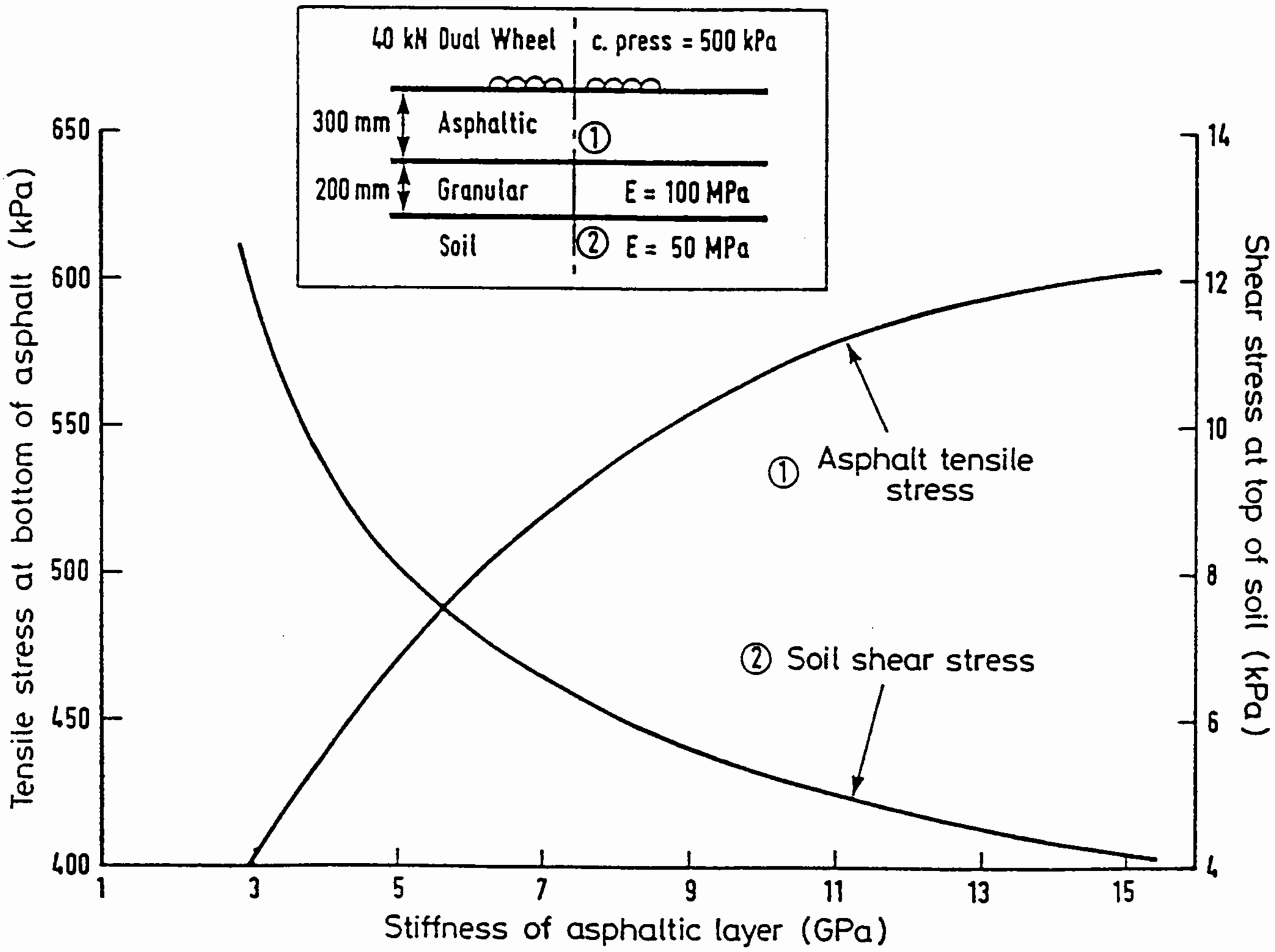


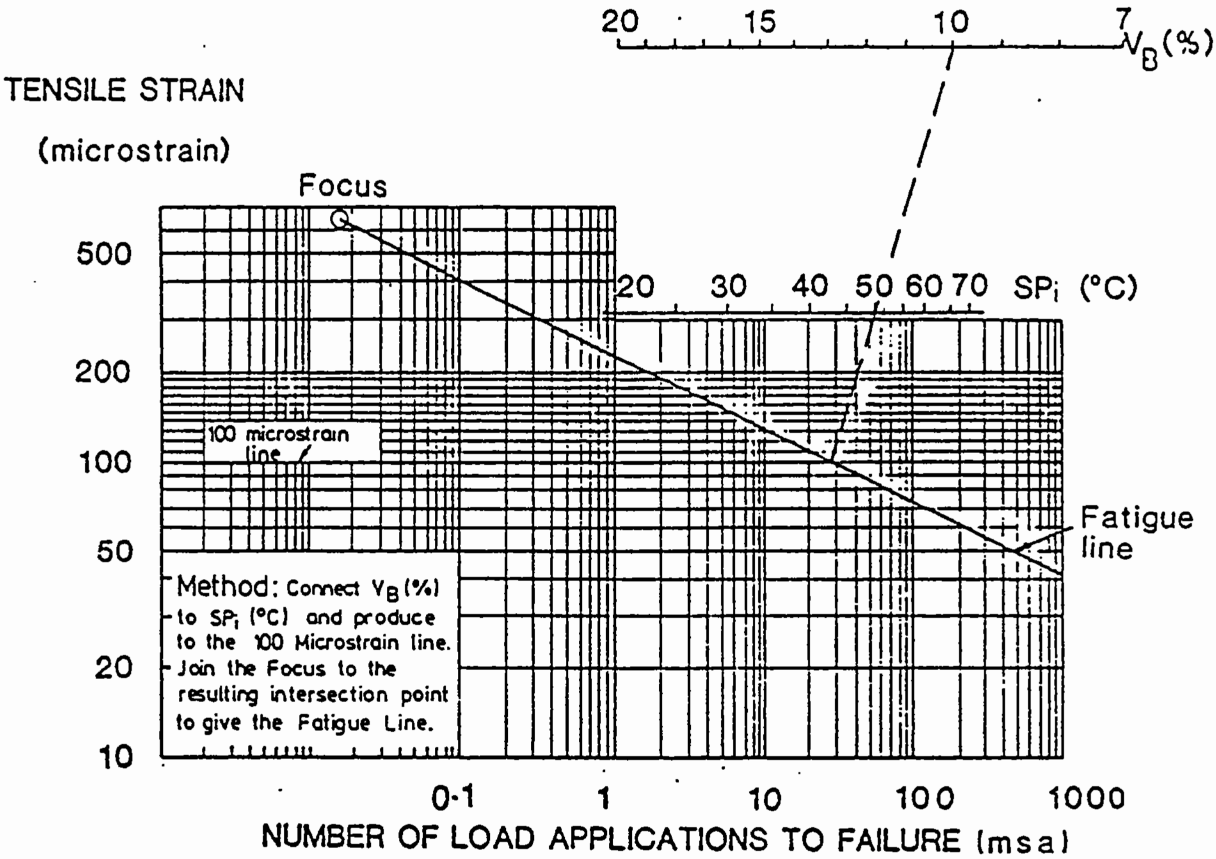
Figure 4.3 The relationship between the asphaltic layer stiffness and the critical pavement stresses. (after Brown) (106)

conditions are further modified by the thickness of the material, with thicker layers contributing to a reduction in the magnitude of tensile strains at the base.

4.3.1 Prediction of fatigue performance

Laboratory tests for fatigue performance have shown that there is a linear relationship between applied stress and the number of cycles to failure when plotted on log-log axes (49). Because of the temperature dependence of bitumen, the fatigue life of the same generic mix will change according to the temperature at which the testing is undertaken. A similar effect can be observed if tests are carried out at different frequencies, with materials exhibiting longer lives at higher frequencies. However, if the results from fatigue tests are re-plotted as the logarithm of strain against the logarithm of load applications to failure then the different results will concide on the same fatigue line, indicating that strain is the criterion of failure (50).

An extensive investigation into the relationship between tensile strain and the life of the material showed that the significant mix parameter influencing the relationship was the volumetric proportion of binder in the mix and its' Ring and Ball softening point (51). The research led to a prediction method for the determination of fatigue strength, shown in nomograph form in figure 4.4, which requires information concerning the volumetric proportions of the mix and the initial softening point of the binder.



$$\log \varepsilon_t = \frac{14.39 \log V_B + 24.2 \log SP_i - 46.06 - \log N}{5.13 \log V_B + 8.63 \log SP_i - 15.8}$$

$$\log N = 15.8 \log \varepsilon_t - 46.06 - (5.13 \log \varepsilon_t - 14.39) \log V_B - (8.63 \log \varepsilon_t - 24.2) \log SP_i$$

Figure 4.4 Nomograph for the determination of fatigue strength. (after Cooper and Pell) (51)

Applying such a prediction to estimate the service life of a pavement is likely to yield conservative results, because laboratory investigations apply continuous loading cycles to the material, whereas a pavement will experience beneficial rest periods between axle loads (52), and there is also a lateral distribution of loading in the wheel track area.

A comparative assessment of the fatigue lives of different mix formulations can be made satisfactorily, and it is in this context that the prediction method has been used in this project.

4.4 RESISTANCE TO PERMANENT DEFORMATION

The resistance to permanent deformation describes the qualities of a material to resist an accumulation of permanent strains in the regions of wheel loading which manifest themselves as wheel track ruts. The deformations may be the result of a build up of strains within any, or all the pavement layers.

The re-structuring of the aggregate matrix under the application of a load generates permanent strain in a bituminous material, which can be accentuated by the magnitude and frequency of the loading, and the temperature of the material during the load applications. For example, a sample of material subject to a constant stress regime will exhibit an immediate initial elastic response, followed by a viscous strain. On removal of the applied stress, the elastic strain is recovered, with some additional delayed recovery over a period of time, figure 4.5a. Some residual strain will remain, which is permanent strain. Figure 4.5b shows the effect as experienced by an element of material in a pavement when

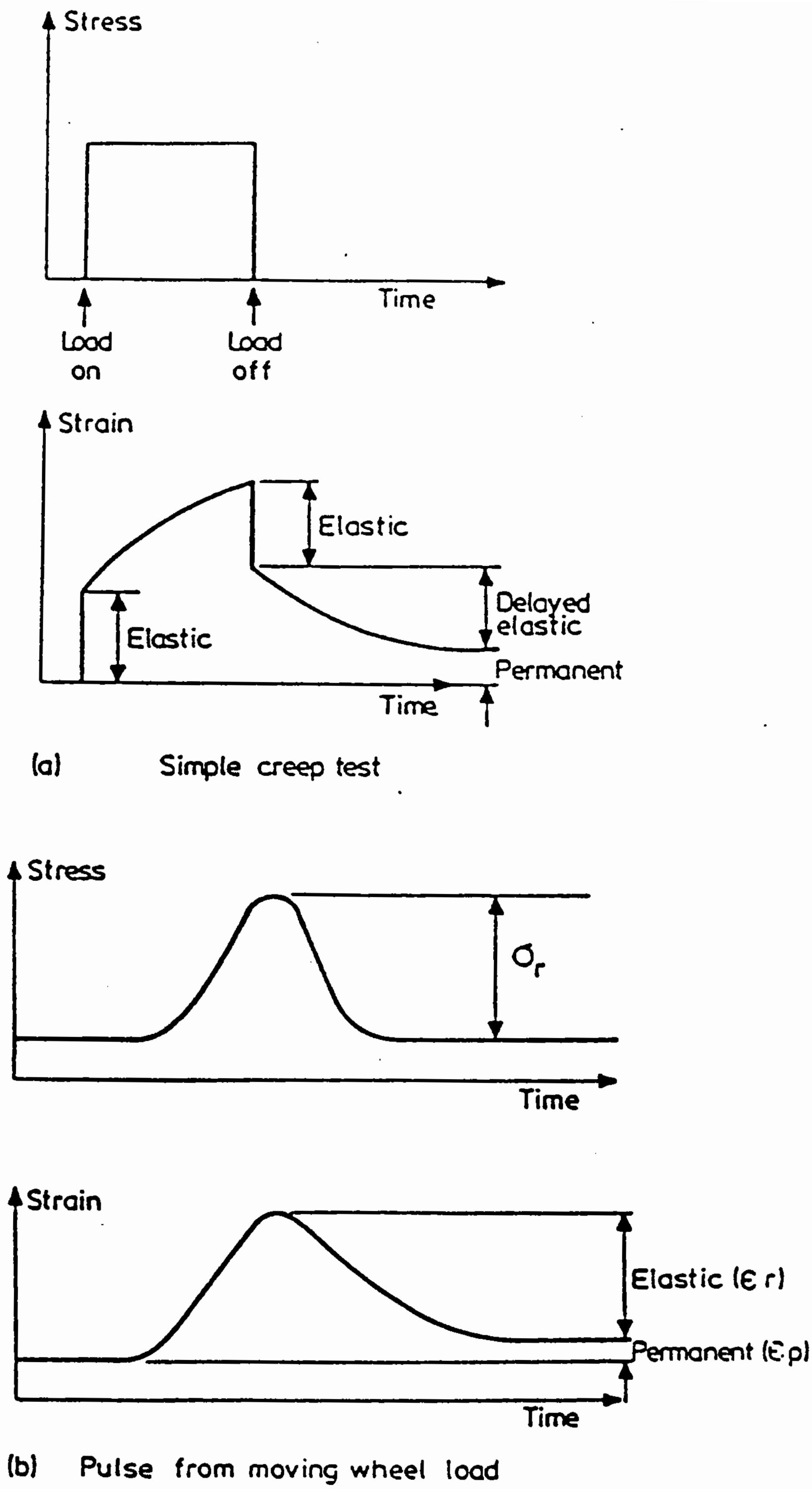


Figure 4.5 Graphical representation of the visco-elastic response of bituminous materials to applied loading. (106)

subject to wheel loading. It is the accumulation of the permanent strains resulting from each wheel load, which over millions of applications form wheel track ruts.

As permanent deformation is primarily associated with the viscous response of the material, it is at high temperatures, combined with long loading times which create conducive conditions for this mode of distress.

4.4.1 The effect of mix variables

The elastic stiffness and resistance to fatigue of bituminous mixtures has been shown to be highly influenced by the volumetric proportions of the mix and the properties of the binder. This has facilitated the development of prediction methods for these two properties utilising information concerning the aforementioned components. The deformation characteristics however, have been found to be dependent upon properties associated with the aggregate (26,53), principally the grading and shape characteristics. Roadbase mixtures of dense bituminous macadams base their structural integrity on the aggregate matrix, with the interlock between stone particles providing the inherent strength of the material.

Chapter 3 discussed the importance of aggregate grading, and investigated methods of designing a gradation to suit the properties of an individual aggregate, rather than using an envelope to specify all types. Angular aggregates provide good mechanical interlock and are generally advocated because of this characteristic but may not always be available. Surface texture is another significant factor which influences the inter-particle friction between the stone particles and the adhesive qualities with the binder, where a good bond is necessary. Concerning mix design and resistance to permanent deformation, the engineer must concentrate on the aggregate grading and the binder content. A dense grading with a low VMA generally equates with good deformation resistance provided that the mix is not "over-filled" with binder and it is satisfactorily compacted (35,54). In mechanical tests, deformation resistance has been shown to increase as binder content decreases (55), but there is clearly some limit to this.

4.4.2 Prediction of permanent deformation

As the resistance to permanent deformation of bituminous materials is strongly dependent on aggregate properties, the prediction of the amount of distress induced through this mode has not proved as easy to model as the properties of elastic stiffness and fatigue resistance. The prediction models applied to the elastic stiffness and fatigue resistance of bituminous materials have been built from statistical data obtained from laboratory creep tests. the results of which can be applied to any bituminous mixture provided information concerning the volumetric proportions and binder properties is available.

Permanent deformation characteristics are more susceptible to subtle changes between different mix formulations, which means that two mixes made with different aggregates, with similar volumetric proportions and binder contents, could have significantly different deformation resistance characteristics. A general rule for permanent deformation cannot therefore apply.

Francken (56) showed, through a series of triaxial tests on bituminous mixtures, that permanent strain depends on the following parameters:

Applied deviator stress

Loading time

Void content

Elastic stiffness

Coarse aggregate content by volume

Proportion of natural sand in the fine aggregate

The filler-binder ratio by volume

The ratio of filler and binder to the VMA

Although his data base facilitated the prediction of permanent strain, an inclusion of eight different parameters suggests that a laboratory test on a particular mix formulation would be more appropriate to quantify permanent deformation. The Shell organisation used this approach by taking results from a static creep test and obtaining a viscous modulus for a particular mix formulation. Knowledge of applied stresses from wheel loadings facilitated the prediction of resultant strains from the viscous stiffness (57,58).

The fundamental problem with the prediction of permanent deformation is that loading regimes and environmental conditions are never constant. Wheel loadings are dynamic and are allied to the type of suspension and axle arrangements of heavy goods vehicles. The engineer should be concerned with developing a mixture which exhibits the optimum characteristics for resisting the accumulation of permanent strain and quantifying this through a relevant mechanical test.

4.5 DURABILITY

The durability of a bituminous mixture describes the resistance of the material to detrimental changes to the mechanical properties from environmental effects such as moisture damage and ageing of binder films. A general summary of the term would be the qualities of the material required to retain the previously described mechanical properties.

This project is concerned with the mix design of the main structural layer of a flexible pavement, which would not be directly exposed either to traffic or air. However, these layers will experience ageing phenomena and possibly moisture damage. Although bitumen is considered a good waterproofing material, water can sometimes displace bitumen from the aggregate, particularly in the case of thin binder films, allowing moisture to penetrate the pavement. Because a pavement can fail before realising its' design life due to moisture related mechanisms, it is necessary to have some means of quantifying the durability of a mix formulation with particular reference to moisture susceptibility.

4.5.1 Air voids

The permeability of a bituminous material is linked with the volume of air voids and hence the moisture susceptibility of the mix. The influence of voids on moisture sensitivity is understood only in a general sense. The lower the void content, the lower the possibility of water penetrating the mix. Some research has suggested mixes with an air void content higher than 6% will have a high probability of sustaining moisture damage (59), although this hypothesis is more relevant to asphalt concrete wearing courses rather than base mixtures. Figure 4.6 show the theoretical relationship between air voids and the retained stiffness of bituminous mixtures following a conditioning period of immersion in water. The area of 'pessimum voids' incorporates the void contents typical of roadbase mixtures, with the optimum level of air voids coinciding with over-compacted and hence impermeable mixes and materials with very high voids, such as friction courses which are free-draining

Materials like pervious macadams which receive a high exposure to environmental effects, require thick binder films to combat ageing distress which may arise from the open void structure. Roadbase mixtures which incorporate thin binder films and a void structure which is neither impervious nor free draining, form candidate mixes for durability associated problems. The air void structure and binder film thickness which dictate the durability of the material must be evaluated in terms of the fundamental mix properties.

The design of the mix formulation may influence the durability because of the volumetric proportions resulting from the optimum mixture. Hence, a comprehensive mix design procedure should include some method of assessing the moisture sensitivity, and apply a 'durability factor' to any mix formulations which are recommended.

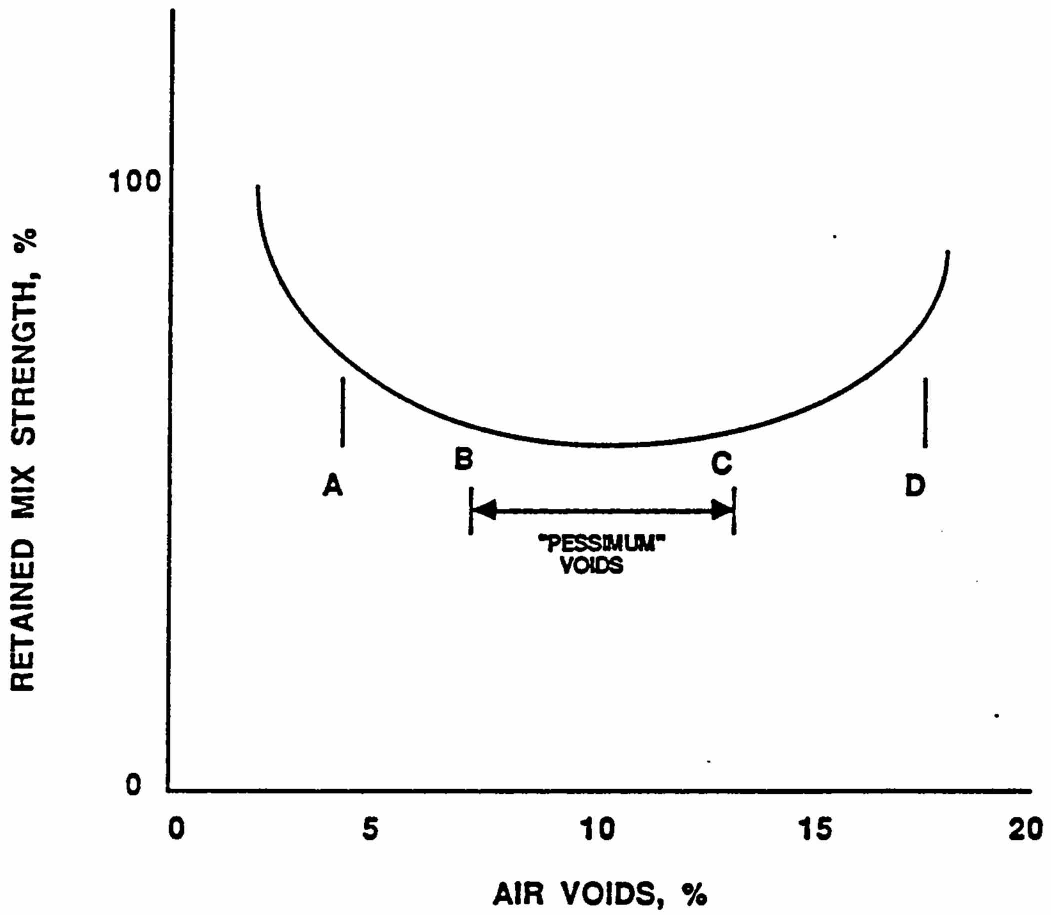


Figure 4.6 Relationship of air voids and relative strength of mixtures following water conditioning. (from SHRP-A/IR-89-003) (107)

4.6 WORKABILITY

The mechanical properties of pavement layers are critically dependent on the compacted state of the mix. The workability of a bituminous material is a general term which describes the ease of handling and compaction of that material during the paving operation. A mix with poor workability may prove difficult to compact which could in turn jeopardise the performance characteristics of that material.

The main factors influencing the workability of a bituminous mix are as follows:

Physical properties of the mineral aggregate
The content of filler in the aggregate grading
The binder content
The penetration of the binder
The mix temperature during the paving operation

The above list was proposed by Al Nageim and Fordyce (60) who performed an investigation into the workability of hot rolled asphalts. The temperature of the material during the paving operation is outside a mix design procedure and depends upon a smooth operation of transporting the material from plant to site and good workmanship but underlines the importance of compaction temperatures if material properties are to realise their design values.

The quantification of the workability of bituminous mixes has received little attention, as most mixtures can be satisfactorily compacted, but variations in mix composition coupled with workmanship elements can lead to under-compacted and over-compacted pavement layers.

Bellamy and Biczysko (61) carried out trials to investigate the workability of a 10mm dense bituminous macadam by applying concrete workability tests. The slump test (62) which is used to assess concrete, was considered because of the simplicity of the method, and the Compaction Factor Apparatus (63) was also investigated because of its applicability to low workability concrete. Both tests were intended for assessment of the 'as-delivered' material on site, as in the case of concrete. The findings of their research suggested that a slump test would be inappropriate for application to bituminous materials but the compaction factor apparatus could be used to discriminate between the workabilities of different mixes. Their most significant finding, however, was that mixes with a large size of aggregate, or a high percentage of large aggregate increased the scatter of the data.

The parameter of workability would appear to be the single most difficult one to quantify as it involves factors beyond the control of the engineer. As a generalisation, mixes typical of those used for roadbases do not present insurmountable problems concerning workability for the paving contractor. Optimisation of material properties through mix design should not significantly alter the material composition from those currently in use and, therefore, unworkable mixes should not result.

4.7 THE RELATIONSHIP BETWEEN MECHANICAL PROPERTIES AND MIX DESIGN

The aim of designing a bituminous mixture is to optimise the available materials in a composition which satisfies end-product requirements. In the United Kingdom, there is no such specification at present, with materials achieving acceptance or rejection on the basis of a recipe specification. Even the Marshall test, which is extensively employed throughout the world and applies a mechanical test to the laboratory manufactured specimens, does not quantify a mix formulation in terms of the fundamental properties which will dictate the performance characteristics of the mix.

In order to understand the behaviour of bituminous materials, knowledge of the elastic stiffness, fatigue life, resistance to permanent deformation and durability are required. Arbitrary parameters such as 'stability' and 'flow' are insufficient despite the numbers of inferences which are made from the result of such testing.

This chapter has described the individual mechanical properties associated with bituminous paving materials, and discussed the factors which influence these properties. When an engineering material only comprises two constituents, the development of an optimum mixture may not seem to pose a significant problem. However, difficulties are immediately manifested when it is realised that the ideal mix compositions to combat the principal pavement distress modes, are in direct conflict with one another. For example, a roadbase mixture with a high resistance to fatigue cracking will require a high volume of binder in the mix, ideally with a high softening point temperature, but such a mix would be susceptible to plastic flow and therefore rutting under a repeated loading regime. The elastic stiffness is determined by the volumetric proportions of the mix, which in turn influences both the fatigue life, and the deformation resistance. Durability and workability requirements both prefer high binder contents, which again conflicts with the resistance to permanent deformation requirement.

The mix design engineer must compromise therefore, between the requirements of different properties in order to try and optimise the overall performance of the mix. Aggregate properties and binder properties must also be considered, increasing the complexity of the problem. Good mix design is a very difficult task, which explains why there have not been significant advances since the inception of the Marshall method in the 1940's, but there is now sufficient understanding in asphalt technology to up-date and up-grade design methodologies. Mix formulations must be designed and quantified according to their fundamental properties and it is the task of the mix design engineer to formulate an optimum mixture without excessive compromise, (figure 4.7).

Figure 4.7 The mix design engineer.

CHAPTER FIVE
SPECIMEN MANUFACTURE

The philosophy generating the need for any mix design procedure is to check that the proposed material is capable of satisfying the demands which will be imposed on that material during its' service life. An integral part of a mix design method, therefore, is the manufacture of samples of the material, which can be subsequently tested. It is of paramount importance that the samples manufactured exhibit similar, if not identical, performance characteristics as the material to be used in practice. If the material is homogenous and behaves elastically, the fabrication of test specimens should not pose any problem, but in the case of bituminous mixtures where the behaviour is visco-elastic and the performance is influenced by aggregate type, grading, and level of compaction, then the reproduction of site material in the artificial environment of the laboratory creates a substantial problem.

Despite half a century of investigations into bituminous materials, it is only in recent times that methods of specimen manufacture have come under close scrutiny. For any test results to be meaningful, laboratory prepared specimens must be representative of in-service mixtures when proper construction procedures are used in the placement and compaction of bituminous materials.

This chapter discusses five methods of compacting laboratory specimens and considers the merits of each procedure in the light of recent investigations.

5.1 MARSHALL METHOD OF COMPACTION

The most common method of specimen manufacture is that used in the Marshall design method. It is standardised (11,64) to produce specimens 64mm in height and 102mm in diameter. Test specimens typically comprise approximately 1200g of aggregate mixed with varying quantities of binder and are fabricated in a mould consisting of baseplate, a forming mould and a collar extension. The hot mix is introduced into the mould assembly and is then tamped around the perimeter and over the interior, prior to removal of the mould collar.

The compaction mechanism is through a dead weight of 4.5kg which is dropped vertically through 457mm onto the specimen for a repeated number of times. This procedure is mechanised, allowing all Marshall specimens to be manufactured using the same apparatus, (figure 5.1). Usually either 30, 50 or 75 blows are applied to the specimen, after which it is inverted and the procedure repeated.

After the compaction process, the specimen must be allowed to cool in air prior to extraction from the mould using an extrusion jack. The resulting Marshall 'briquette' is a squat cylindrical specimen which has been compacted through the application of a dynamic axial compressive load. This creates arbitrary orientation of the aggregate, and because of the small mould size, is inappropriate for gradations with a large maximum particle size.

Fig 5.1

62-63

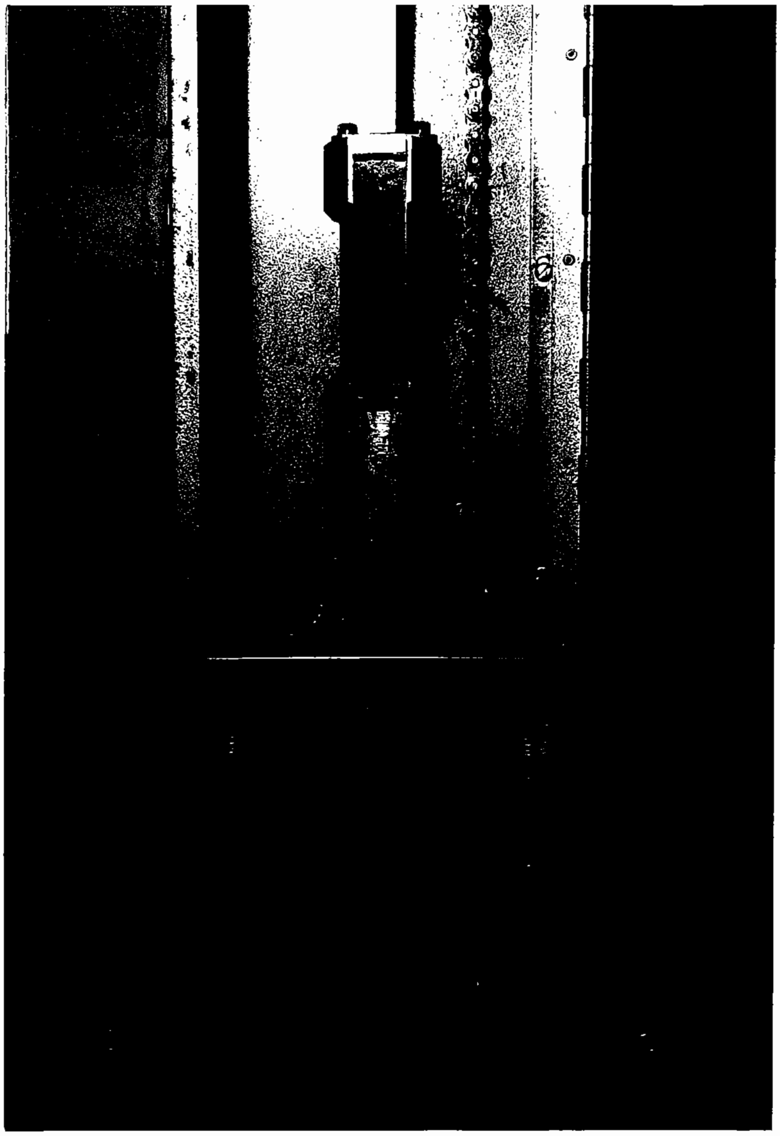


Figure 5.1 Photograph showing Marshall compaction apparatus, comprising automatic drop hammer, flat tamping face, mould and compaction pedestal.

5.2 DURIEZ COMPACTION

This compaction procedure is taken from the method of preparing bituminous samples for the LCPC Duriez Test (65). The method produces specimens which are 150mm high by either 100mm or 120mm in diameter depending on mould size. The apparatus comprises the mould, a baseplate, a split ring arrangement and a compression facility. The split ring is inserted between the mould and baseplate, and the mix introduced into the mould in three layers, each of which should receive 25 blows from a hand held tamping rod. A steel disc is then placed on top of the material, and a nominal force of 5kN is applied to the specimen. The split ring is then removed and friction generated upon application of the pre-load prevents the mould from sliding down to the baseplate. A static force is then applied via a manually operated hydraulic jack or a compression test facility. During application of the load, the mould and the compression piston are free to move relative to the baseplate, the object being to obtain a uniform density throughout the length of the specimen. The procedure is shown diagrammatically in figure 5.2.

Compactive effort can be varied by adjusting the compressive force applied to the specimen which can be between 50kN and 150kN depending upon the mix formulation and the degree of compaction desired.

Like the Marshall method, the Duriez procedure compacts mixtures through a direct overall compressive stress, although in this case the loading is applied statically. The resulting aggregate orientation is mainly dependent on the state of the mixture after manual tamping, the compaction method not facilitating preferential orientation to take place.

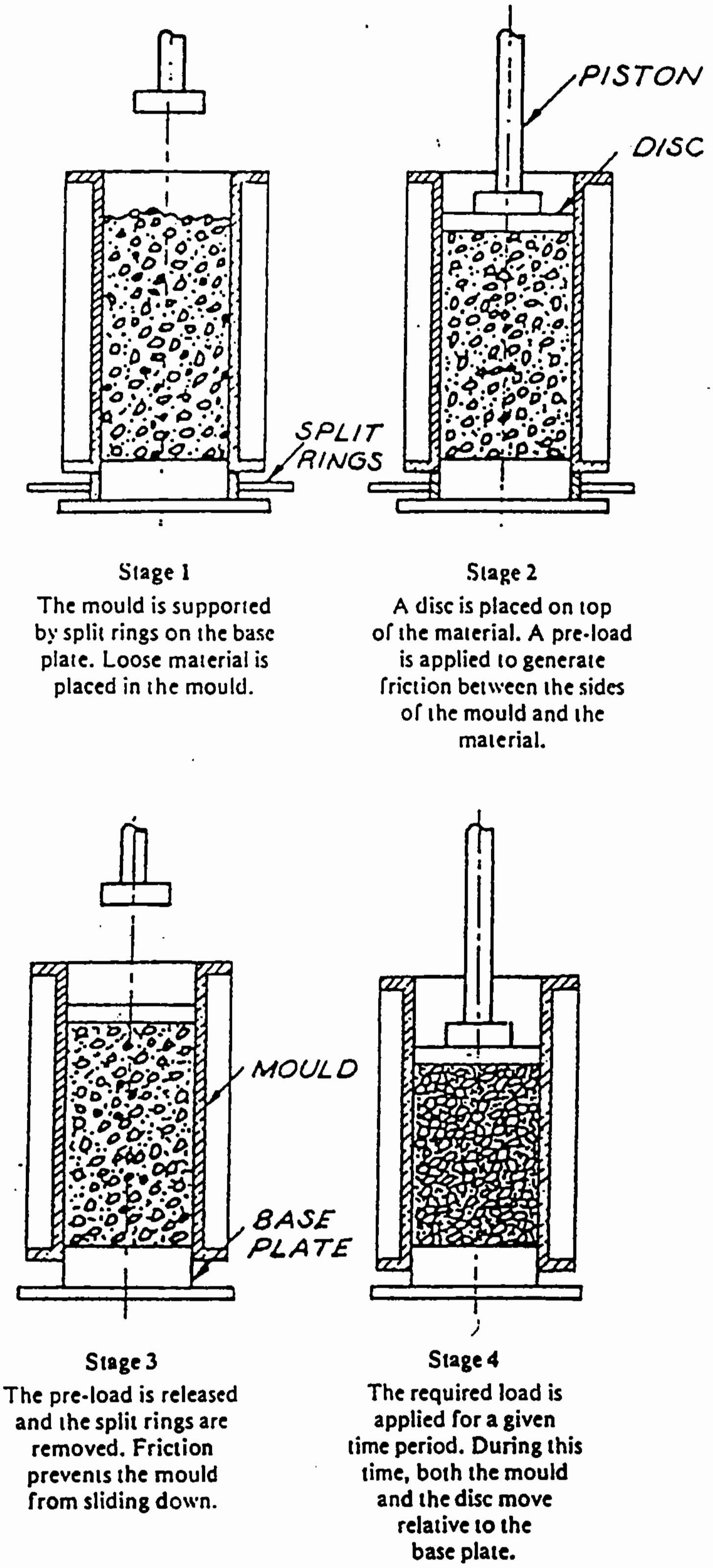


Figure 5.2 The Duriez compaction procedure. (108)

5.3 KNEADING COMPACTION

Compaction procedures on site involve the use of rollers which impart a kneading action to the bitumen stone matrix during the compaction process. In order for laboratory reproduction of site material, some form of kneading action applied to the specimen during the fabrication process is desirable. This was originally introduced as part of the Hveem method of mix design, where the California Kneading Compactor was employed.

5.3.1 The California Kneading Compactor

The apparatus required for this type of compaction consists of a mechanical hammer, a compaction foot and specimen mould (66). The mould is 102mm internal diameter by 127mm deep, into which the mix is introduced, tamped and levelled. The mould assembly is then placed in position in the mechanical compactor.

The compaction of the test specimen is accomplished by the mechanical compactor imparting a kneading action to the sample by a series of individual impressions from the compaction foot. The foot is the shape of a sector of a circle of the same diameter as the mould, (figure 5.3) which is migrated around the mould delivering the kneading action. Each load application should deliver a pressure of 3.45MPa and should be maintained for approximately 0.4 seconds.

Compactive effort can be varied by altering the number of load applications or by reducing the compaction pressure.

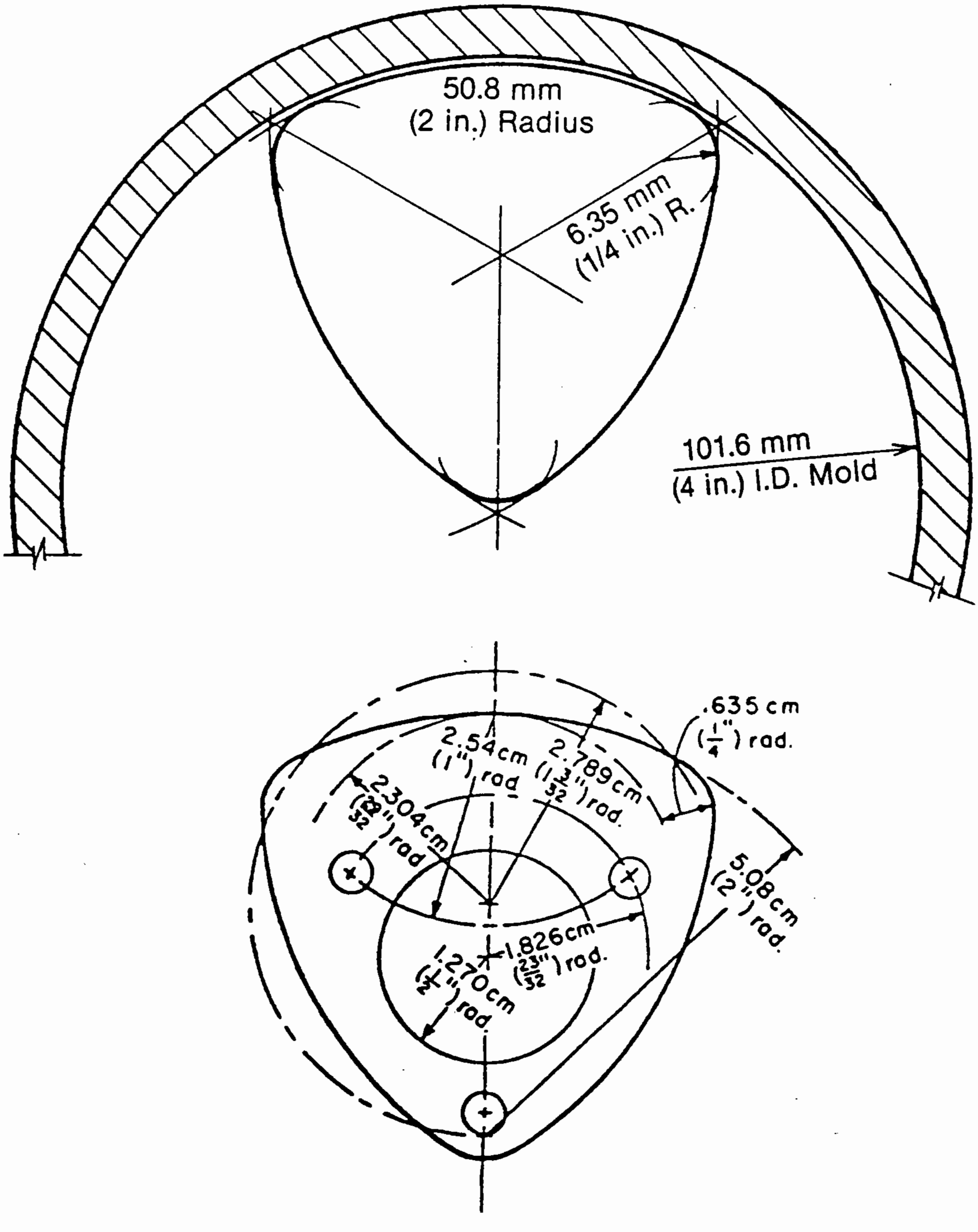


Figure 5.3 Details of compacting foot for mechanical kneading compactor. (5)

5.3.2 The PRD apparatus

The equipment used for the Percentage Refusal Density Test (67) in the United Kingdom, which was originally developed for field compaction evaluation, facilitates a method of specimen manufacture using a kneading compaction effort. The apparatus consists of a split mould of 150mm internal diameter by 170mm deep and a detachable baseplate. The equipment has two compaction tools, a small foot (102mm diameter) and a large foot (146mm diameter) which are applied via a manually operated vibrating hammer. The principal compaction tool is the small foot which the operator should migrate around the surface of the sample in a N, S, E, W, NE, SW, NW, SE, pattern, maintaining a tangential contact between the foot and the mould. The large compaction foot is used to flatten surface irregularities at the end of the compaction procedure. The detachable baseplate facilitates compaction to both ends of the specimen to try and eliminate density gradients and achieve as great a degree of homogenity as possible. Compactive effort can be varied by adjusting the duration of compaction and the material temperature. A diagram of the apparatus is shown in figure 5.4.

5.4 ROLLING COMPACTION

The most precise method of emulating the action of a road roller is to build a scale version of a roller wheel and compact specimens of bituminous materials in the same manner. Of the methods considered in this chapter, a laboratory roller compactor must be classified as specialist equipment as they are not typically found in highway laboratories within

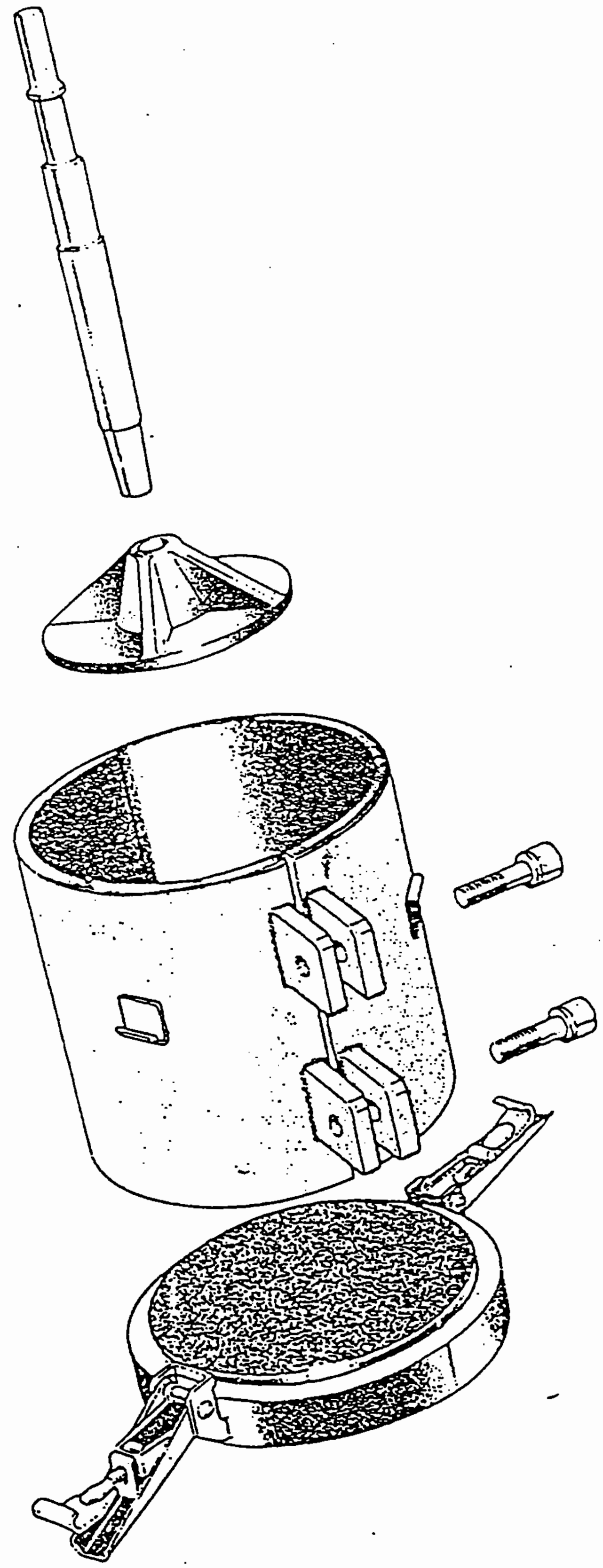


Figure 5.4 The Percentage Refusal Density test apparatus. (67)

the United Kingdom or overseas, although the technique is widely used in France.

Rolling compaction facilitates the manufacture of slabs of material, rather than the individual specimens which result from the other means of compaction. Test specimens may then be cored from the slab, allowing 100mm or 150mm diameter specimens to be obtained. The cores may also be taken either through the thickness of the slab or along the longitudinal axis of the slab. Efforts should be made to ensure that a satisfactory ratio of maximum particle size to slab thickness is maintained to facilitate preferential aggregate orientation during the compaction sequence.

The roller compactor facility at Nottingham consists of a sector of a roller wheel of diameter 900mm vertically loaded by a pneumatic actuator, and a steel mould which is mounted on a trolley powered by a horizontal actuator which provides a constant speed of travel. The mould dimensions are 400mm by 280mm by 100mm deep, and compaction is carried out to a fixed volume. A general view is shown in figure 5.5.

It is important when dealing with such a volume of material under laboratory conditions to avoid segregating the mixture upon introduction to the mould. An even distribution of the mix is vital if pavement characteristics are to be reproduced. In order to achieve this, the mixture is introduced into the mould through a steel hopper via a dividing unit, which compartmentalizes the mould into five equal divisions. The slab therefore is made up of five nominally identical batches of material. After filling each division, the dividing unit is removed from the mould prior to compaction.

Fig 5.5

66 - 67

Figure 5.5 The Nottingham University rolling compaction facility.

5.5 GYRATORY COMPACTION

The gyratory compactor provides another facility which attempts to reproduce the aggregate orientation resulting from site compaction procedures. This apparatus employs a shearing action of the mixture at a low initial pressure which allows the aggregate to be orientated in a preferential arrangement prior to high pressure compaction.

The principle of gyratory compaction is to rotate a mould on an axis eccentric to the vertical, and apply a compressive load to the material inside the mould through parallel end plates. A schematic view of the mechanism is shown in figure 5.6. In the United States this method of compaction has been standardized (68) and incorporates a circular mould 100mm in diameter and 100mm deep, and a tilt mechanism to facilitate a vertical eccentricity of 6°. Several variations on this specification have been manufactured in other countries altering the mould size and the angle of inclination of gyration. The gyratory shear compacting press (PCG test) in France (69) uses a mould of diameter 160mm and compacts material to give a specimen 150mm deep using a gyration angle of 1° and a rotational speed of 6rpm. The vertical load applies a mean compressive pressure of 0.6MPa during compaction. Gyratory compaction directly addresses the problem of aggregate orientation in laboratory prepared specimens and tries to reproduce the stone matrix which is achieved on site. As roadbase mixtures contain large aggregate particles, in order to minimise the constraining influence of the mould, it would be preferable to use moulds which have an internal diameter of at least 150mm.

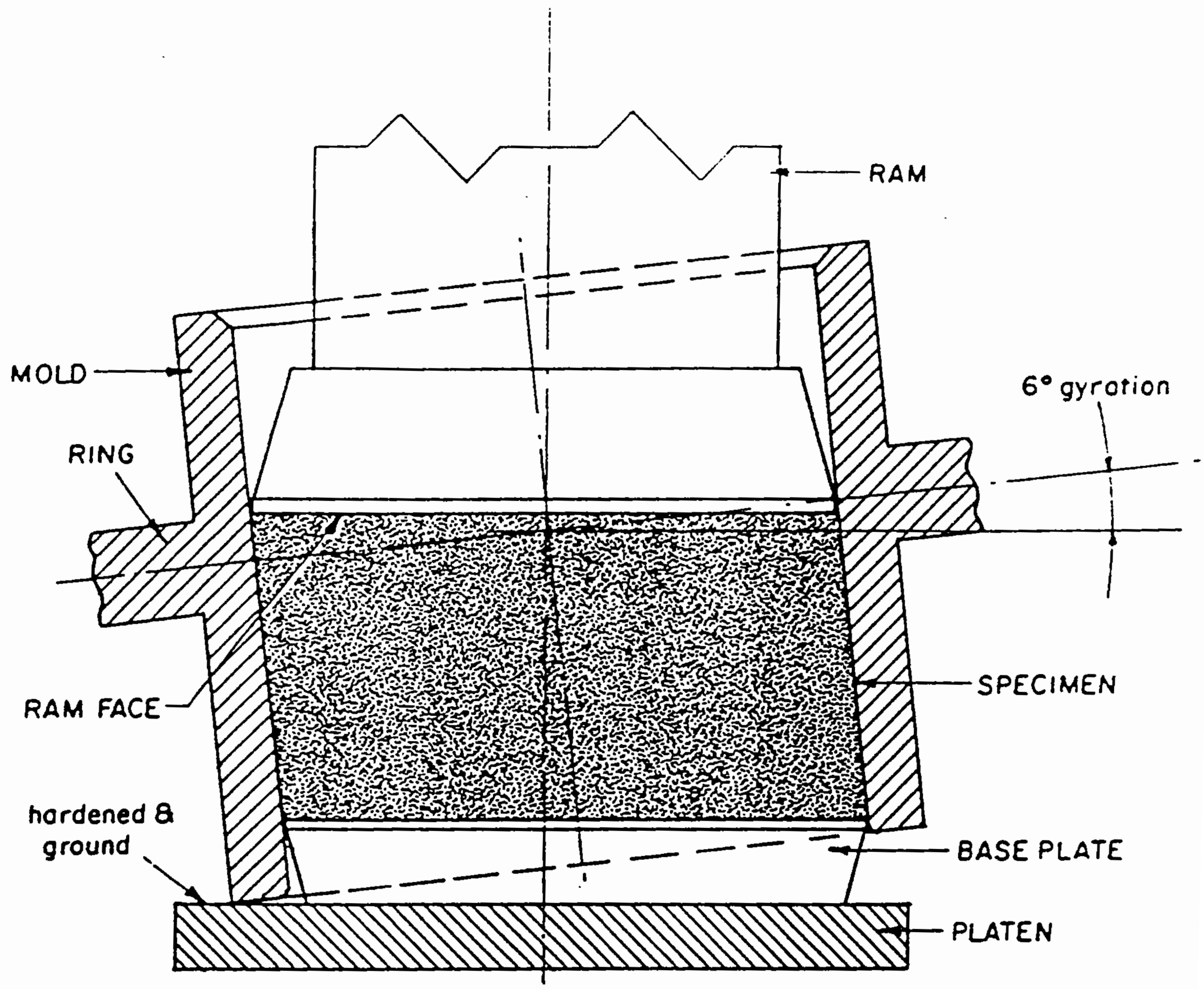


Figure 5.6 Schematic illustration of the method of gyratory compaction for bituminous materials.

5.6 DISCUSSION

The manufacture of specimens in laboratory conditions forms the cornerstone for subsequent analysis of asphalt-aggregate mixtures. Five classifications of fabrication methods have been discussed, which must be critically compared with each other and assessed with reference to their relevance in modern day asphalt technology.

Three of the methods impart some form of kneading or shearing action to the mixture, similar to the effect of site compaction, and two simply rely on direct axial loading to compress a bituminous mix to a desired dimension. Current mix design practises are dominated by the second category and the Marshall test. This fails on two points which are:

(i) Mechanism of compaction
(ii) Dimensions of mould

Some agencies in the USA have tried to improve on the test procedures by modifying the apparatus and using a 150mm diameter mould (70) but the compaction method still relies on dynamic impact loading from a vertical drop hammer.

A recent National Cooperative Highway Research Program study (NCHRP) (71) has identified the most promising laboratory compaction techniques. These were gyratory, kneading and rolling wheel compaction. These three methods came under scrutiny as part of the Strategic Highways Research Program (SHRP) investigations, where the influence of compaction method has been assessed in terms of the mechanical properties of the mixture (72). Permanent deformation and fatigue were considered to be the performance characteristics of the greatest concern, although some investigations were performed to examine the elastic stiffness values from the different fabrication methods. The major findings of the investigation can be summarized as follows:

(1) The mechanical properties of the mixtures were affected by the compaction method.

(2) Kneading compaction produced specimens which had the greatest resistance to deformation. Gyratory compaction produced specimens which had the least.

(3) Rolling wheel compaction produced specimens which exhibited the greatest resistance to fatigue under controlled stress conditions. Both kneading and gyratory compaction produced very similar results.

(4) Rolling wheel compaction produced the stiffest mixes followed in order by kneading and gyratory compaction.

On the basis of the investigation it was recommended that rolling wheel compaction should be the preferred method for the production of laboratory specimens of bituminous materials. Acknowledgement was given to the variability which could rise from kneading and gyratory compaction by use of different sized moulds, compaction feet, number of layers and the number of tamps per layer.

5.6.1 Recommendations with respect to a mix design procedure

One of the reasons why the Marshall Test has become so popular and is still used despite recognition of its limitations, is the simplicity and rapidity of the method of specimen manufacture. A mix design procedure must assess a number of different mix formulations in order to make any recommendation on the mixture composition. Hence, ease of specimen manufacture would appear an attractive asset in the introduction of a new procedure.

In UK practice, there is substantial inertia within the 'industry' which generates difficulties when trying to implement new methods and philosophies. The SHRP recommendation of using rolling compaction to manufacture all laboratory specimens would prove impractical for a UK mix design procedure as it incorporates specialized equipment which is not readily available. Furthermore, the quantity of material required to perform a full mix design using a roller compactor facility, and the time required to fabricate each slab would prove prohibitive factors concerning the procedure.

Gyratory compaction also necessitates specialized apparatus, which reduces the potential methods of manufacturing specimens to some form of kneading compaction. If attention is given to mould size, compaction feet and the thickness of specimens, then a kneading compactor should prove to be an adequate tool for mix design purposes. It is conceivable that a roller compactor could be used in the corroboration of mix design results obtained through kneading compaction methods. Taking account of the SHRP investigation, it is likely that the mechanical properties of the same mix formulation compacted to similar volumetric proportions by the two methods would yield slightly different results, with rolling compaction having the closest representation of site material. Hence, the optimum mix formulation from the mix design procedure could be reproduced using a roller compactor facility, allowing the anticipated site mechanical properties to be checked against design criteria.

CHAPTER SIX

REVIEW OF TEST EQUIPMENT

6.1 INTRODUCTION

Chapter 4 introduced the properties of bituminous materials which are of importance for pavement construction. The three fundamental mechanical properties are the elastic stiffness, the fatigue strength and the resistance to permanent deformation. Although considerable research has led to prediction methods for obtaining elastic stiffness and fatigue lives of mixes (73,51), a simple practical laboratory test would be preferable for comparing the values resulting from minor changes in mix formulation. Resistance to permanent deformation is a parameter which is strongly influenced by aggregate properties and aggregate grading and should be quantified by some form of mechanical test. Currently the only test in use which purports to quantify deformation resistance is the Marshall stability test but work has demonstrated that this is inappropriate to correctly address the problem as it is unable to rank a series of mixes in order of their deformation resistance(74).

To determine the mechanical properties of paving materials in the laboratory, tests should reproduce as clearly as possible the anticipated in-situ conditions to which the material will be subjected. This requires that temperature, loading rates, stress conditions and the volumetric proportions of the compacted specimen are all given careful attention. Reproduction of the stress conditions generated in a pavement under wheel loading creates a very complex problem, resulting in laboratory tests re-creating the stresses which are likely to be of most significance.

This chapter reviews some test methods which have been employed to measure the mechanical properties of bituminous materials, and introduces a new piece of apparatus which has been developed at Nottingham University alongside the work on this project.

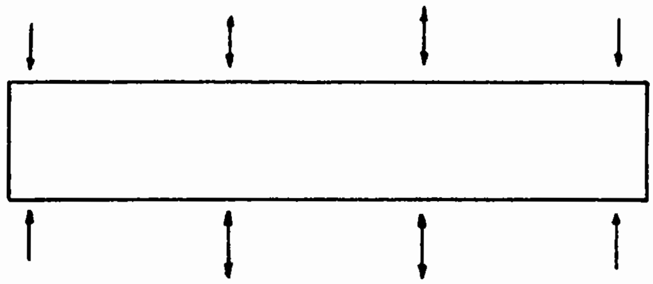
6.2 MEASUREMENT OF MECHANICAL PROPERTIES

6.2.1 Elastic Stiffness

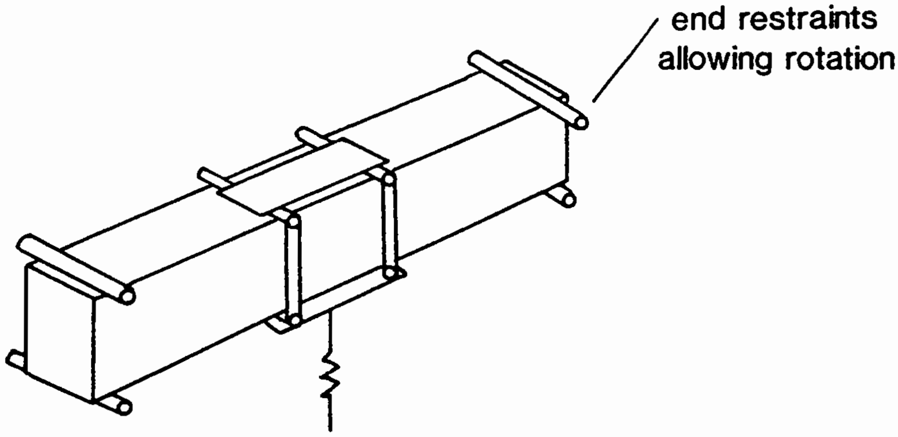
The term stiffness is used to define the ratio of stress to strain of bituminous materials. The most popular techniques for measuring this parameter are covered by the comprehensive descriptions of the simply supported beam, the uniaxial test and the indirect tensile test. (75,76,77,78)

6.2.1.1 Simply supported beam

This type of test comprises a rectangular beam of material, which can be sawn from a larger slab or prepared by compaction in a mould, simply supported at the ends and subject to a sinusoidal cyclic load applying a central deflection to the centre of the beam. The preferred test configuration is that of the four point bending test (figure 6.1) which induces a constant bending stress over the central third of the beam The test must be performed at a constant temperature. Deflections are measured by transducers, and values of stiffness can be calculated over a range of frequencies and temperatures. Dimensions of the test specimens are in the order of 370mm long, 100mm wide and 60mm deep (75).



a) 4 point bending arrangement



b) Diagrammatic view of beam test

Figure 6.1 Diagrammatic representation of the four point bending test for evaluation of elastic stiffness.

6.2.1.2 The Uniaxial test

This test arrangement consists of an unconfined cylindrical specimen which is subject to an axial sinusoidal cyclic load. Deformations are measured over central gauge lengths on either side of the specimen usually by Linear Variable Differential Transformers (LVDT's). An ASTM standard (11) specifies the use of cylindrical specimens either 100mm or 150mm in diameter and 250mm or 300mm in length, with a stress regime which does not induce high permanent strains in the range between 0-240kPa.

Similar tests have been conducted at Nottingham University on cylinders 100mm in diameter by 200mm in length (76), where the load is cycled with a mean value of zero so that both tensile and compressive stresses are applied. Testing conditions were standardized at 5°C and 21°C over a frequency range of 0-40Hz with an axial stress amplitude of 200 kPa.

6.2.1.3 The Indirect Tensile Test

The repeated load indirect tensile test obtains the elastic response of a briquette-sized bituminous cylinder through the application of a load pulse across a diameter, and measurement of the resulting deformation across a perpendicular diameter. The test has gained significant status in the USA, since it was introduced in 1971 (77), and a standard test procedure has been incorporated for the test (78). The significant difference between this test method and the two described previously, is that the indirect tensile test uses specimens of 100mm diameter by 64mm long. This facilitates the use of specimens manufactured by procedures as discussed in Chapter 5, and cores taken from the road pavement whereby specimens are often difficult to obtain.

As the load is applied to a section of the circumference of the specimen, loading strips with concave surfaces having a radius of curvature equal to the normal radius of the test specimen must be used. A series of conditioning pulses should be applied to the test specimen in order to satisfactorily seat the loading strips onto the surface of the material, prior to applying the test pulses. The horizontal deformation, corresponding to a pulsed load on the vertical diameter is measured by a pair of LVDT's mounted on a light-weight frame as shown in figure 6.2.

For this arrangement the stiffness is calculated from the equation:

$$S = \frac{P\,(\upsilon + 0.27)}{t\Delta H} \qquad 6.1$$

where S = Elastic stiffness (MPa)

P = Applied load pulse (N)

υ` = Poisson's ratio

t = Specimen thickness (mm)

ΔH = Instantaneous recoverable horizontal deformation (mm)

The value of Poisson's ratio for bituminous mixtures must be estimated because of the inherent properties of the material. The value is generally considered to lie between 0.3 and 0.4, although at extremes of temperature, this assumption will not hold true. For testing at 20°C, a Poisson's ratio of

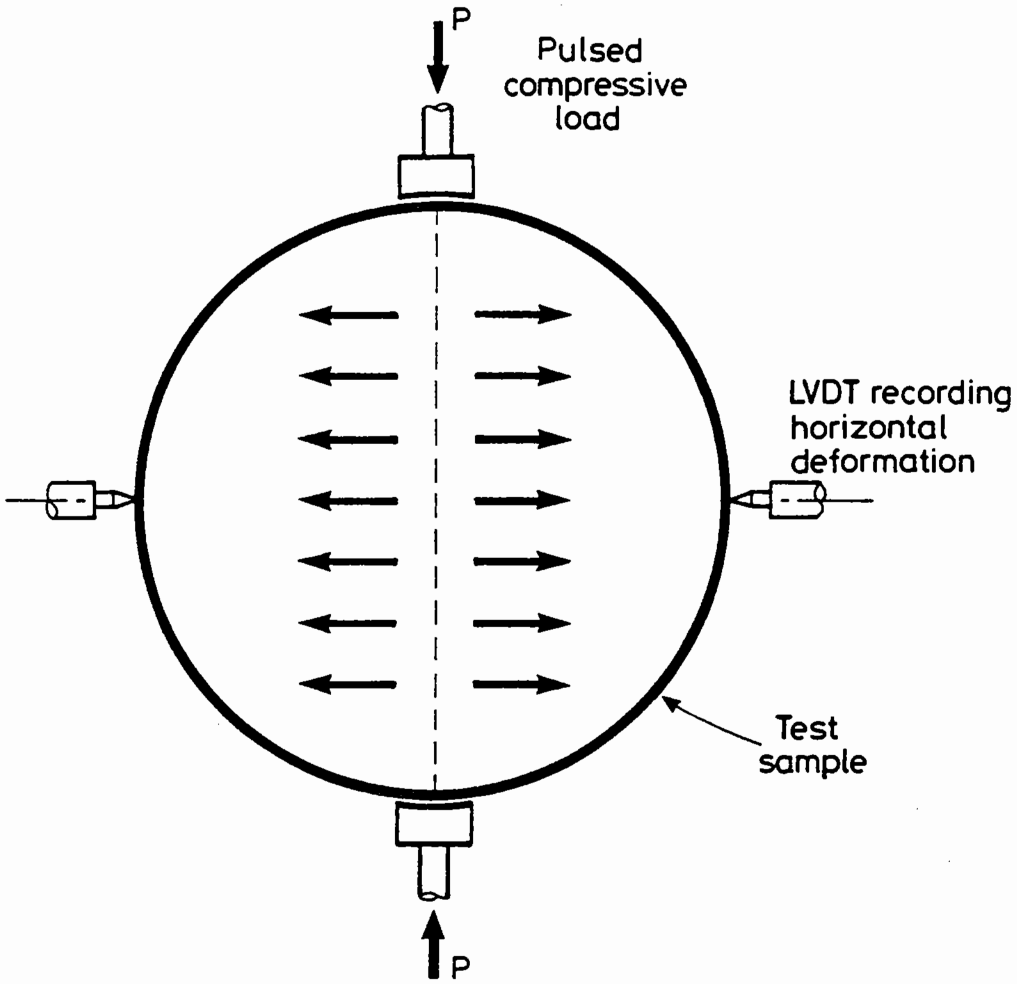


Figure 6.2 Schematic of test configuration for the Repeated Load Indirect Tensile test.

0.35 has been found to be a reasonable estimate (78) although discrepancies in the true value may effect calculated stiffness values by up to 7%.

6.2.2 Fatigue Strength

Several of the test arrangements for the measurement of elastic stiffness can also be used for fatigue testing. The 4-point bending test described in 6.2.1.1 has been used to investigate the fatigue resistance of mixtures (79) by relating the induced tensile stress to the number of stress cycles to failure. Similarly, the push-pull test on cylindrical specimens has been used by the direct application of loading creating a sinusoidal tension-compression stress regime in the specimen (80).

The rotating-bending test has been extensively used at Nottingham University (51) where a moulded cylindrical sample of reducing diameter to a minimum value at the centre of the specimen is mounted as a vertical cantilever on a rotating shaft. A point load is applied through a bearing at the top of the specimen, resulting in a sinusoidal variation of bending stress throughout the specimen with a maximum value occurring at the neck. Load cycles are repeated until failure occurs. Variations on this test arrangement, such as trapezoidal cantilevers where a maximum bending stress occurs at a distance from the fixed end, have also been used to generate data concerning the fatigue resistance of bituminous materials.

The main problem with fatigue tests is the number of samples which must be tested in order to make a proper evaluation of fatigue strength due to the inherent scatter of results.(80) To date a rapid test method has not been developed to address this problem making this kind of test inappropriate for routine use for mix design work.

6.2.3 Resistance to permanent deformation

6.2.3.1 Axial tests

Following basic research performed by Hills (81), the uniaxial creep test has found widespread acceptance as a means of assessing resistance to permanent deformation. It is a simple test and can be carried out on laboratory prepared samples or cores taken from a road pavement. The test conditions have been standardised as a stress of 100kPa acting on the specimen for 1 hour at a temperature of 40°C (82). Various configurations of possible test arrangements exist, with the object of the test being to measure the vertical deformation exhibited by the bituminous sample under the specified conditions.

A more complex test arrangement is the repeated load triaxial apparatus, where a confining stress is applied to the specimen as well as an axial compressive stress. This type of test can make a fundamental assessment of the material under different loading conditions, but requires more complex equipment.

6.2.3.2 The Wheel Tracking test

The wheel tracking test is intended to simulate the action of wheel loading on the road pavement. The Transport and Road Research Laboratory carried out a large amount of work on the deformation resistance of wearing course materials using a reciprocating wheel tracking apparatus (83), which has led to their test method becoming virtually standard.

The test assembly consists of a square slab of material which is rigidly restrained on all four sides, subjected to a reciprocating motion under a fixed loaded wheel. A solid rubber tyre, 204mm diameter by 47mm wide applies a force of 525N to the slab, giving a mean normal pressure of 530kPa, and indents a straight track in the specimen. The wheel is loaded by a weighted lever arm, to which contact is made by an LVDT facilitating deformation measurement. The apparatus is constructed so as to allow two slabs to be tested simultaneously, as shown in figure 6.3. The test is able to demonstrate the rate at which materials deform, and quantify mix formulations by a relative assessment of deformation resistance.

6.3 THE NEED FOR SIMPLE TEST METHODS

Due to the complex nature of the relationship between the mechanical properties of bituminous materials and the temperature and speed of loading, several of the test methods previously described involve lengthy procedures and require relatively sophisticated equipment. Simple test methods are necessary for mix design and end-product specification of material, as well as failure investigation and quality control purposes.

From the above descriptions, elastic stiffness and deformation resistance can be quantified by a form of simple test but such a method has so far proved elusive for the fatigue strength of the mixture. The number of samples which require testing in order to evaluate this property exclude fatigue testing from routine mix design work.

Fig 6.3

78-79

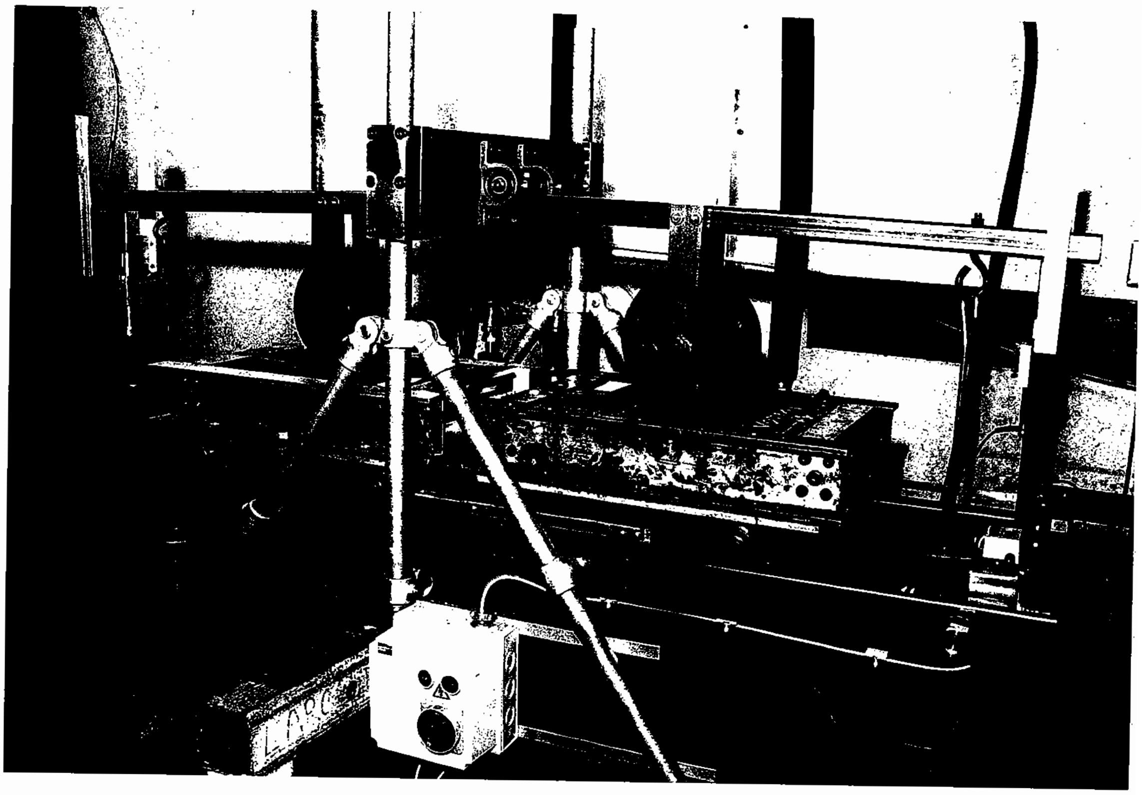


Figure 6.3 The Nottingham University reciprocating wheel tracking apparatus.

Current methods of quality control using compositional analysis to check compliance with recipe specifications are indirect and unsatisfactory, yet form the basis for the acceptance or rejection of highway materials.

The development of new test equipment adopted the philosophy that the apparatus should be inexpensive, easy to operate and require the minimum effort in specimen preparation. The uniaxial creep test and the Repeated Load Indirect Tensile (RLIT) test are both suitable for inclusion in such an apparatus as they meet the practical aspect of the above criteria. The RLIT test is non-destructive allowing the the elastic stiffness of a specimen to be measured before using the same sample in a permanent deformation test.

6.4 THE NOTTINGHAM ASPHALT MIX TESTER

6.4.1 Background

The Nottingham Asphalt Mix Tester (NAT) is the result of development work performed at Nottingham University on the measurement of mechanical properties of bituminous mixes (84). It is capable of assessing the elastic stiffness and deformation characteristics of different mixtures. The apparatus (Figure 6.4) operates from a compressed air system allowing portability of the equipment, as pressurized air may be obtained directly from a compressor or a compressed air cylinder. The compressed air drives an actuator which applies the loading to the specimen. During testing the pneumatic system is controlled by computer generated signals which are transmitted via an interface system. The hardware of the

Fig 6.4

79 & 80.

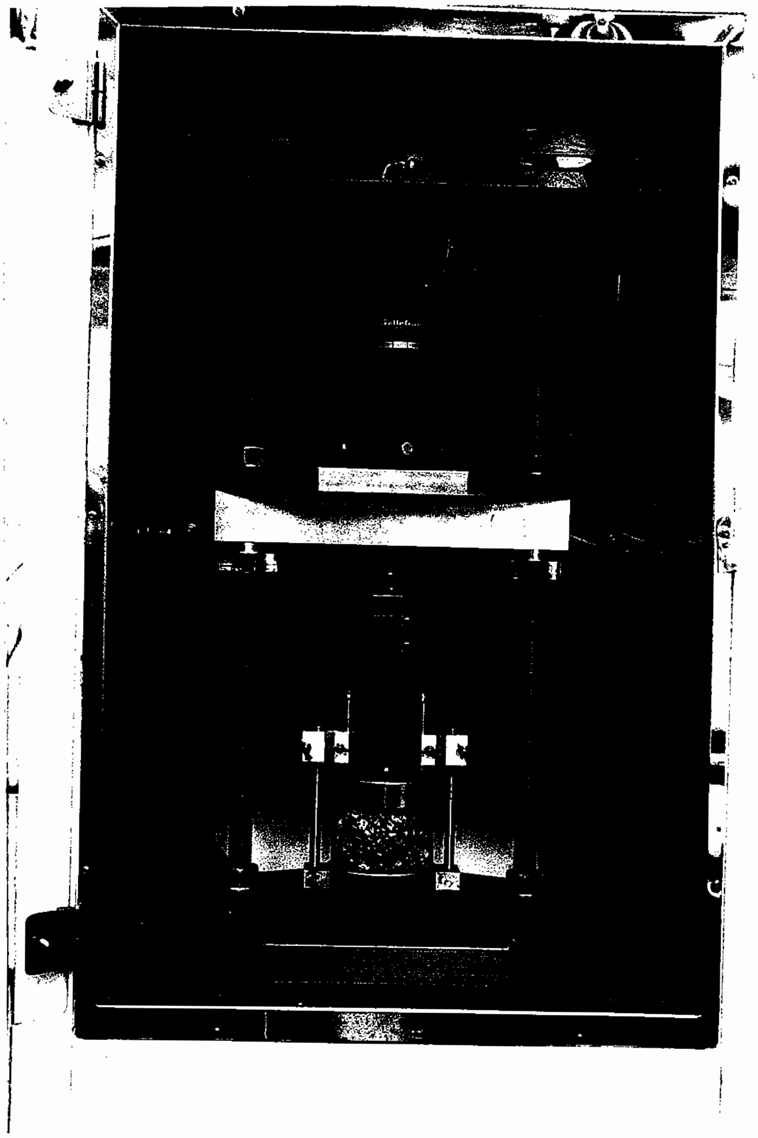


Figure 6.4 The Nottingham Asphalt Mix Tester. Illustration of loading frame, adjustable height crosshead and central actuator in the deformation test arrangement.

apparatus consists of a loading frame with an adjustable crosshead, and a central actuator and loading arrangement, and the digital interface system. The user must add a microcomputer, a compressed air supply, and a facility to house the loading frame in a temperature controlled environment.

6.4.2 Mechanical Tests

During development of the apparatus it was found that slight modifications to the configuration used for the uniaxial creep test would also facilitate RLIT tests making the NAT a comprehensive test unit. The layouts for the apparatus are shown in Figures 6.5.

The uniaxial creep test can accommodate 100mm diameter and 150mm diameter specimens, and applies a static pressure of 100kPa to the sample for a duration of 1 hour. A conditioning load of 10kPa may be applied to the specimen for 10 minutes prior to full load application, and a relaxation period of fifteen minutes may be observed after the test to monitor elastic recovery. Axial deformation is monitored via one or two LVDT's which can be seated either on the loading platen or on a flange connected to the loading rod.

Although results from the creep test have exhibited reasonable correlation with data from sophisticated repeated load triaxial tests (85), the test is not adequate for distinguishing between mixes with difference aggregates and gradings. The static creep test measures deformations caused by a viscous flow of the binder film, and once aggregate to aggregate contact has been established, the creep of the specimen will be arrested. If an unbound

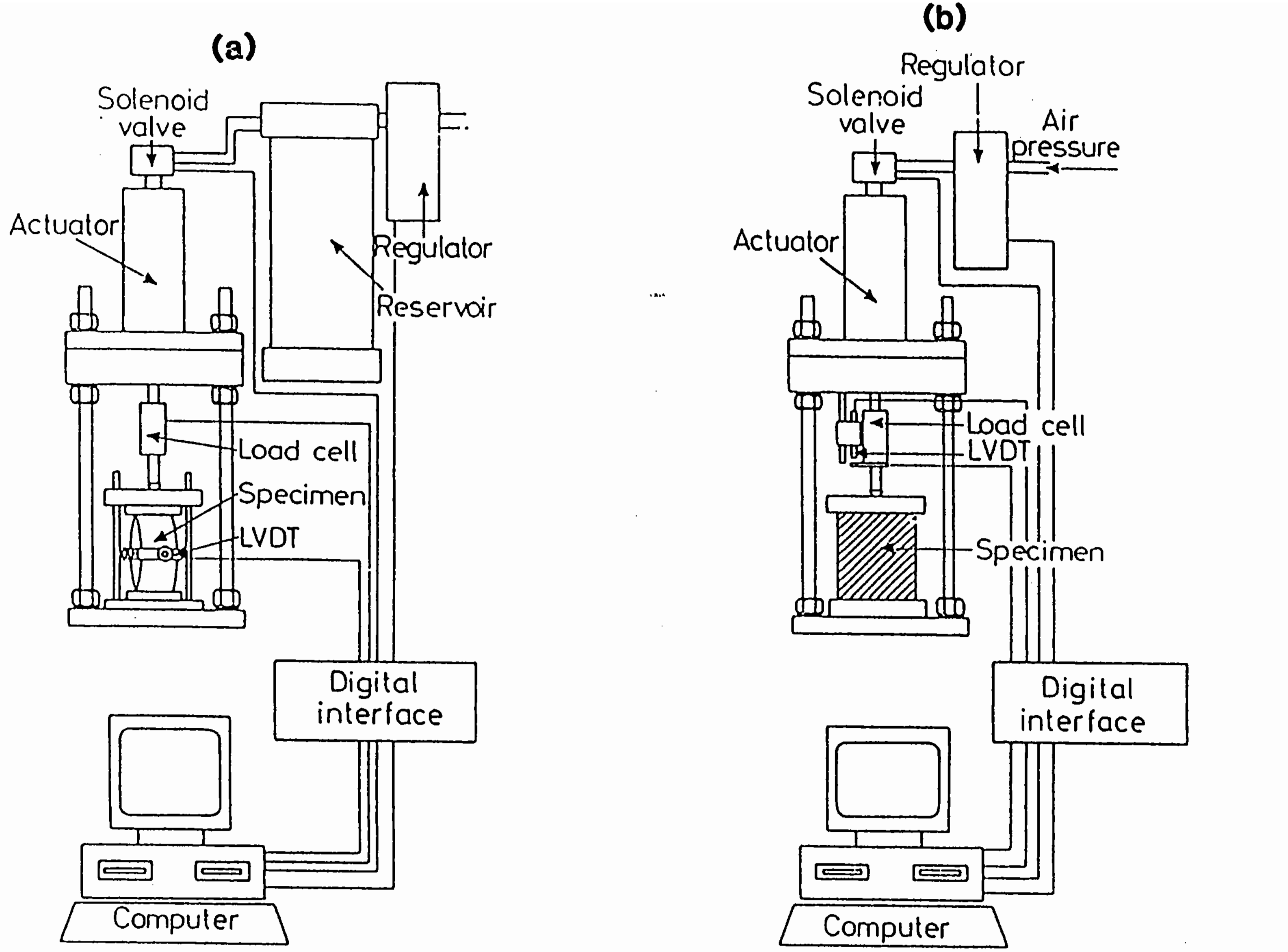


Figure 6.5 (a) NAT loading arrangement for Repeated Load Indirect Tensile Test.

(b) NAT loading arrangement for uniaxial compression test.

compacted aggregate is subject to a pulsed load, permanent strain will be generated upon each load application. However, if the aggregate receives a static pressure, an initial strain will take place upon application of the pressure but subsequent deformation through creep phenomena will not occur. Therefore, in order to make an assessment of the role of the aggregate in the deformation resistance of a bituminous mixture, it is necessary to apply some form of repeated load to the specimen, figure 6.7. Consequently, the test apparatus should provide a facility for repeated load testing of materials. This has been achieved using the NAT by maintaining the conditions of the uniaxial creep test but modifying the loading regime. The variation of load with time approximates to a square wave with the load applied for one second, followed by a second rest period. The duration of the test becomes two hours, corresponding to 3600 load applications. In order to standardise the test, the first ten load applications are taken as a conditioning period allowing intimate contact to be established between the loading platens and the specimen. The deformation at the tenth load application therefore becomes zero and acts as the datum for the remainder of the test. The configuration of the test apparatus remains the same as that for the uniaxial creep test.

The RLIT test is used to determine the elastic stiffness of the test specimens. This requires a different frame and loading platens to be located around the specimen and located in the NAT as shown in figure 6.5a. The adjustable crosshead facilitates 100mm or 150mm diameter specimens to be tested but sample thickness is limited to about 75mm for this type of test. For mix design purposes, the test is typically performed at 20°C, and a value of Poisson's ratio for bituminous materials is assumed as 0.35.

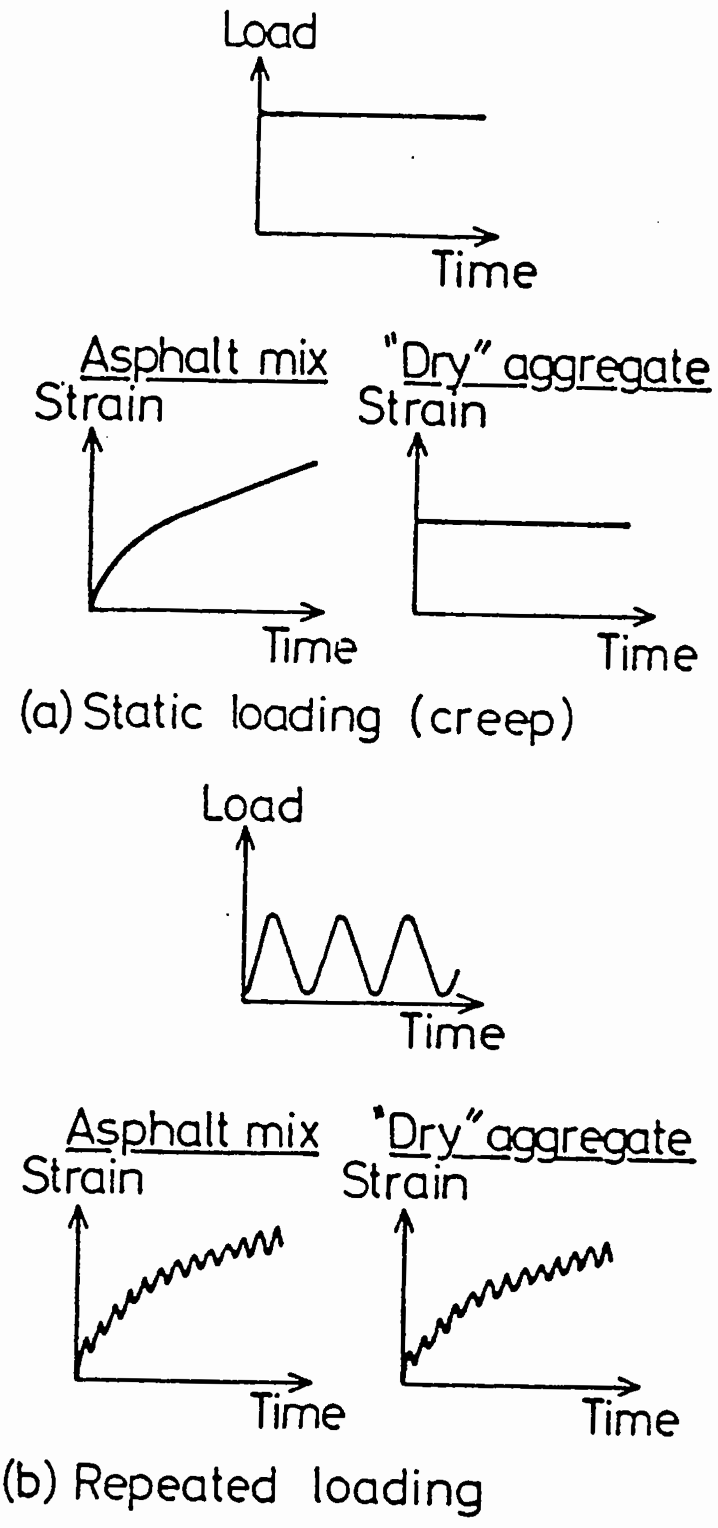


Figure 6.6 Theoretical comparison between induced deformations through static and repeated loading.

6.4.3 Discussion

The development of simple test procedures for quantifying the mechanical properties of bituminous materials forms an integral part of asphalt technology. The introduction of the NAT facilitates rapid and easy testing on laboratory prepared specimens and cores cut from the road pavement. It must also become a key tool with reference to mix design work as the ability to quantify the mechanical properties of specimens with different mix formulations is of paramount importance.

CHAPTER SEVEN

APPLICATION OF MIX DESIGN PRINCIPLES

Figure 4.7 illustrated the complexity of the design of a bituminous paving mixture. By assessing materials in terms of their mechanical properties, the task of the engineer becomes clearly defined, as the end product properties of the material constitute the parameters which will accept or reject the design. A fundamental understanding of the effect of mixture variables on the mechanical properties is required if mix designs are allowed to make a significant contribution to pavement performance. As a compromise must be met between the optimisation of the different mechanical properties, it is of utmost importance that the correct balance is achieved. The traditional philosophy adopted by the Marshall method has been to 'maximise the binder content without compromising mix stability'. This means incorporating a high binder content, without resulting in a deformation susceptible mix, which would theoretically give a very durable pavement with good resistance to the two principal distress modes. However, this forms a dangerous practice when considering the state of the art of the Marshall test. Deformation resistance is arbitrarily quantified by the stability test on a specimen in which the mode of loading is not representative of field conditions. A more rational approach would be to concentrate on whichever distress mode caused the greatest problems in the area under consideration and adjust the mechanical properties according to those requirements.

7.1 PAVEMENT TRIALS IN A HOT CLIMATE

In the United Kingdom, the majority of problems which occur on major highways are caused through the related mechanisms associated with the action of repeated loading from heavy traffic. Material composition and level of compaction of the bituminous layers will have the greatest influence on the response to applied loading. Environmental factors have a limited contribution to material performance, as average ambient temperatures are around 9°C, and pavement temperatures vary between 1.5 and 2.0 times the air temperature (86). Generally, winter temperatures do not maintain extended spells of very cold weather, and conversely, lengthy spells of very warm weather seldom occur during summer months. Hence, thermal influences on material distress have limited application in the UK, although with threatening climatic change, this may not always be the case. Other countries do not have such a co-operative climate, for example in the U.S.A thermal cracking is a common form of pavement distress in the Northern States. It develops through differential cooling of the bituminous material from the surface to the interior, causing variable rates of contraction throughout the material depth. The interior material prevents full surface contraction leading to development of cracks. Research has been able to estimate the effect of thermal cracking on bituminous mixtures (87) and would have to be incorporated into a mix design when considering material for such an environment.

Hot climates pose another problem to the highway engineer as daily high temperatures will result in bituminous paving materials exhibiting a predominantly viscous response to applied loading. Incorrect mix design will very quickly manifest itself through a rutted pavement. One of the Municipalities of the United Arab Emirates was experiencing such problems with their highways, and Nottingham University was invited to assist in an investigation of the mix design which had been used to specify the materials used in the Municipality. This investigation involved a co-ordinated effort from the members of the Nottingham University Pavement Research Group, including a contribution from the author.

The asphaltic concrete had been designed according to the Marshall method, and was proving inadequate for the environmental conditions and axle loadings to which the pavements were subjected. The designs provided an example of maximising the binder content of the mixtures, which was proving to be an ill-founded philosophy under the Arabian conditions. The approach to mix design taken as part of this project, and performed in the Arab pavement trials, was to minimise the binder content without compromising crack resistance and durability.

7.1.1 The Arabian Marshall design

The aggregate gradation used in the Arab state was typical of asphaltic concrete as used in the U.S.A comprising a continuous grading with a high percentage of fine aggregate. The grading was fine of the maximum density Fuller curve as expressed by the equation:

$$P = \left(\frac{d}{D}\right)^{0.45}$$

where P = percentage of material passing a sieve of size dmm

D = maximum particle size (mm)

If the exponent were changed to 0.4, the equation would become more representative of the gradation used. This produces a continuously graded dense material with a low percentage of coarse aggregate. Marshall mix design gave an optimum binder content of 4.3% for this gradation.

The binder was a 60/70 penetration grade which would help achieve high levels of elastic stiffness.

7.1.2 The Nottingham design

Samples of the aggregate and bitumen were sent to Nottingham to allow mix design trials to take place. Aggregates were in the quarry bin sizes which were 28mm - 22mm, 22mm - 12mm, 12mm - 5mm, passing 5mm and passing 0.075mm. In practice overall control of the gradation was limited to the four bin sizes, but for the laboratory investigation the material was sieved into individual size fractions. Specific gravity and absorption determinations were made according to BS 812 (88), the results of which are displayed in Table 7.1.

Table 7.1 Aggregate Specific gravity determinations

Particle size (mm)	28-22	22-12	12-5	5 down
Apparent SG	2.959	2.973	2.977	3.006
Bulk SG	2.927	2.939	2.943	2.971
Bulk SG (SSD)	2.938	2.951	2.954	2.983
Effective SG	2.943	2.956	2.960	2.989
Absorption (%)	0.38	0.39	0.39	0.40

SSD denotes saturated surface dry

Three gradations were investigated. These comprised the grading which was used on site, the maximum density 'Fuller' curve, and a grading corresponding to a curve lying on the coarse side of the maximum density line. This would facilitate examining the effect of 'opening up' the gradation and introduce a more dominant coarse aggregate skeleton. It was anticipated that this modification should produce a mixture with greater resistance to permanent deformation if mixed with an optimum quantity of binder. The gradings were classified as follows:

Grading A: Job specification grading as used in the Arab state

Grading B: Maximum density grading from the equation $P = \left(\frac{d}{D}\right)^{0.45}$

Grading C: Coarse gradation (corresponding to a curve lying under the maximum density line)

Gradation C required a means by which it could be specified. In order to achieve a coarser grading than the Fuller curve then it is necessary to increase the value of the exponent in the maximum density equation. Cooper et al (35) investigated the possibility of assessing aggregate gradings by adjusting the exponent, but found that this approach could lead to curves with unrealistic filler contents, figure 7.1. This is because the equation only controls the upper boundary of the range of values, and allows the lower boundary, in this case the filler content, to be dictated by the exponent. In order to rationalize the grading equation it became necessary to fix the lower bound value of the range. By adjusting Fuller's equation and considering the factors affecting the curve, Cooper et al proposed the following equation:

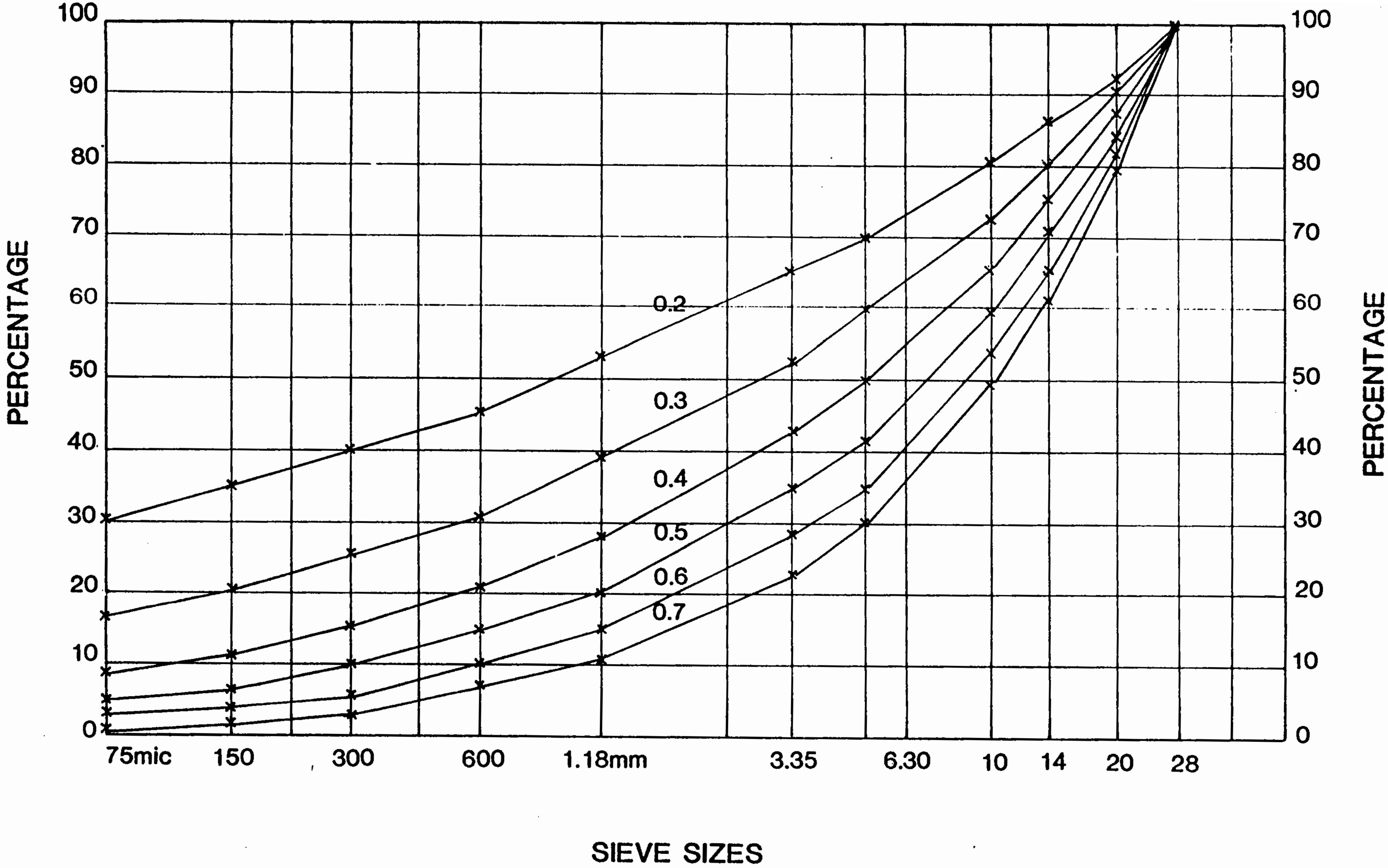


Figure 7.1 Gradation curves resulting from the equation $P=(d/D)^n$

(exponent values 'n' ranging from 0.2 to 0.7)

$$P = \frac{(100 - F)(d^n - 0.075^n)}{(D^n - 0.075^n)} + F \qquad 7.1$$

where P = percentage passing a sieve of size d(mm)

D = maximum particle size in mm

F = filler content (material passing the 0.075mm sieve)

n = an exponent between 0 and 1

Therefore, the filler content must be established prior to specifying a gradation by use of this equation. This can allow the engineer flexibility in the determination of the aggregate grading, which facilitates consideration of the quantity of dust inherent with some aggregates. The Arabian aggregate was relatively dust-free, and it was decided to use a filler content of 10% and an exponent of n = 0.65 for grading C. Gradings A, B and C are shown in figure 7.2.

The high filler content was chosen to compensate for the 'open' grading, which would give a high void content if manufactured using a conventional filler content of around 5%. If the aggregate matrix provides a good skeletal interlock, a higher void content need not effect the deformation resistance of the mix. However, in such an environment as the Middle East, conditions can lead to durability problems because of the ageing effects on the binder film. In chapter 4 the significance of air voids, with respect to durability, was discussed, primarily in terms of moisture damage, but gaseous infiltration of the mix can also lead to problems. Increasing the filler content of the mix provides a dual insurance against pavement deterioration through durability associated mechanisms:

(i) The addition of more fine material to the mixture will have the effect of reducing the void content. This may be a desirable result when considering particularly open gradings.

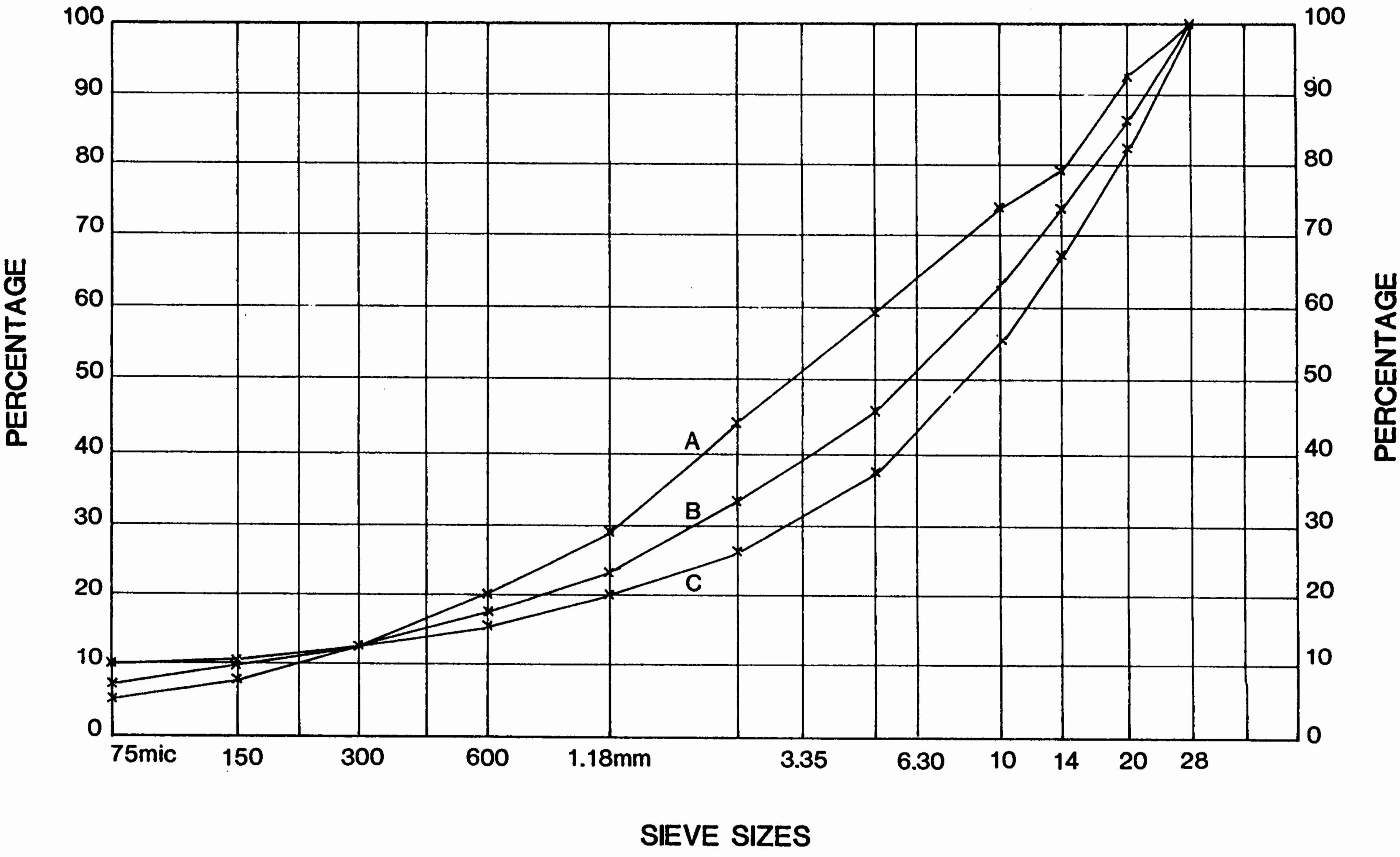


Figure 7.2 The gradations investigated in the mix design trials.

(ii) The quantity of binder required to coat all the material must be increased. Therefore a thicker binder film thickness can be achieved.

The second point illustrates the difficulty of balancing the mix constituents in order that one or more of the mechanical properties will be sacrificed. The distress mode manifesting such problems in the Arab state was rutting, which can be accentuated by higher binder contents. The approach of the mix design procedure was to manufacture specimens of material at each grading over a range of binder contents. Gradations A and B were mixed with binder contents of 3.8%, 4.3% and 4.8%, with the intermediate value corresponding to that of the Marshall design. Gradation C was mixed at two binder contents of 3.4% and 4.0% as it was considered that these mixes saturate at a higher level of binder content. A final set of 3 specimens was manufactured corresponding to grading C with a reduced filler content of 8%.

7.1.2.1 Specimen manufacture

The importance of the method of specimen manufacture has been discussed in chapter 5, and it was concluded that some form of kneading compaction must be applied to a mix if laboratory specimens are to be representative of site material. The effect of variations in compactive effort also requires addressing for a full analysis to be made of the performance of an individual mix formulation.

The material used in the Middle East had been designed by a 75 blow Marshall test which resulted in a binder content of 4.3%. The Marshall test had been performed on specimens containing 28mm aggregate, which had been compacted by vertical drop hammer. This example highlights how the Marshall procedure is an unrealistic method by which to design asphaltic concrete. The net result in this case history was that greater densities of the material were achieved in the field under the action of kneading traffic, than had been predicted in the Marshall test. This was leading to flushing of the binder, severe viscous deformation and hence rutting.

An alternative mix design procedure would need to address all these aspects. The chosen apparatus for the manufacture of specimens was the equipment which is normally employed for the Percentage Refusal Density (P.R.D) test. The apparatus is described in section 5.3.2 and illustrated in figure 5.4. The intermediate size of mould allows the incorporation of 28mm sized particles without causing disruption to the aggregate matrix through confining effects of the mould. Compaction is applied via a vibrating hammer with a small tamping foot allowing a kneading action to be transmitted to the material. The effect of compactive effort was addressed by varying the length of time of compaction, and the temperature of the mix during compaction. Three levels were chosen, which can be summarized as follows:

Level 1	Compaction to refusal (100 PRD) 2 minutes compaction on each face of the specimen using the small compaction foot. Mix temperature 160°C.
Level 2	Intermediate level (approx 97 PRD) 60 seconds compaction on each face of the specimen using the small compaction foot. Mix temperature 100°C.

Level 3 Low level of compaction (approx 94 PRD) 30 seconds compaction on each face of the specimen using the small compaction foot. Mix temperature 90°C.
All temperatures were monitored by a probe.

Level 1 adheres to the conditions of the Percentage Refusal Density test which facilitates an examination of the volumetric proportions of a particular mix formulation at a very high state of compaction, and also allows the relative states of compaction of levels 2 and 3 to be calculated. Compaction level 2 should be representative of well compacted site material with a PRD of 96 or 97. Level 3 represents a slightly under-compacted mix of around 94 PRD.

The conditions advocated for levels 2 and 3 were achieved by the manufacture of a series of trial specimens. The target values of PRD form approximate predictions which will change according to the aggregate type used in the mix design. In this respect the PRD method of specimen manufacture can also be used to gauge the workability of a mix, for example if level 2 produces specimens at 98 PRD, then the mixture will be very workable, and conversely a mix could be classified as relatively unworkable if the same conditions yielded a PRD of 94. Information such as this should be given consideration, and if necessary passed on to the engineer responsible for the paving operation.

7.1.2.2 Mixing apparatus

In order to manufacture specimens of bituminous materials, a facility is required for mixing the stone and binder, ensuring a consistent film thickness on all particles. Marshall mixes are typically mixed in commercial bread dough mixers with wire stirrers, but more robust equipment may be advisable when dealing with larger quantities of material. Each batch for a PRD specimen constituted about 4.3kg of aggregate plus the appropriate quantity of binder.

The mixing facilities used for this project consisted of sun and planet mixers, which were fitted with heated oil jackets to prevent any heat loss and hence changes in binder viscosity during the mixing process. Figures 7.3 and 7.4 show general details of the facilities and the paddle arrangement of the mixers. The mixers can easily cope with up to 6kg of mix per batch, and a mixing duration of about four minutes is generally adopted to ensure full coating of the aggregate.

After mixing, the coated material is stored on trays in heated ovens prior to being introduced into the PRD moulds. The PRD apparatus of moulds and compaction feet are also heated in ovens to mixing temperature prior to compaction.

7.1.2.3 Preparation of test specimens

The compaction procedures result in finished specimens which are 150mm in diameter and approximately 100mm deep. After cooling, the baseplates are removed and the split moulds opened to release the specimens. To obtain test specimens, a central core of 97mm diameter was taken from each sample and then trimmed using a masonry saw to a length of about 70mm, leaving test specimens of similar size to Marshall briquettes.

Fig 7.3. & 7.4

93 & 94

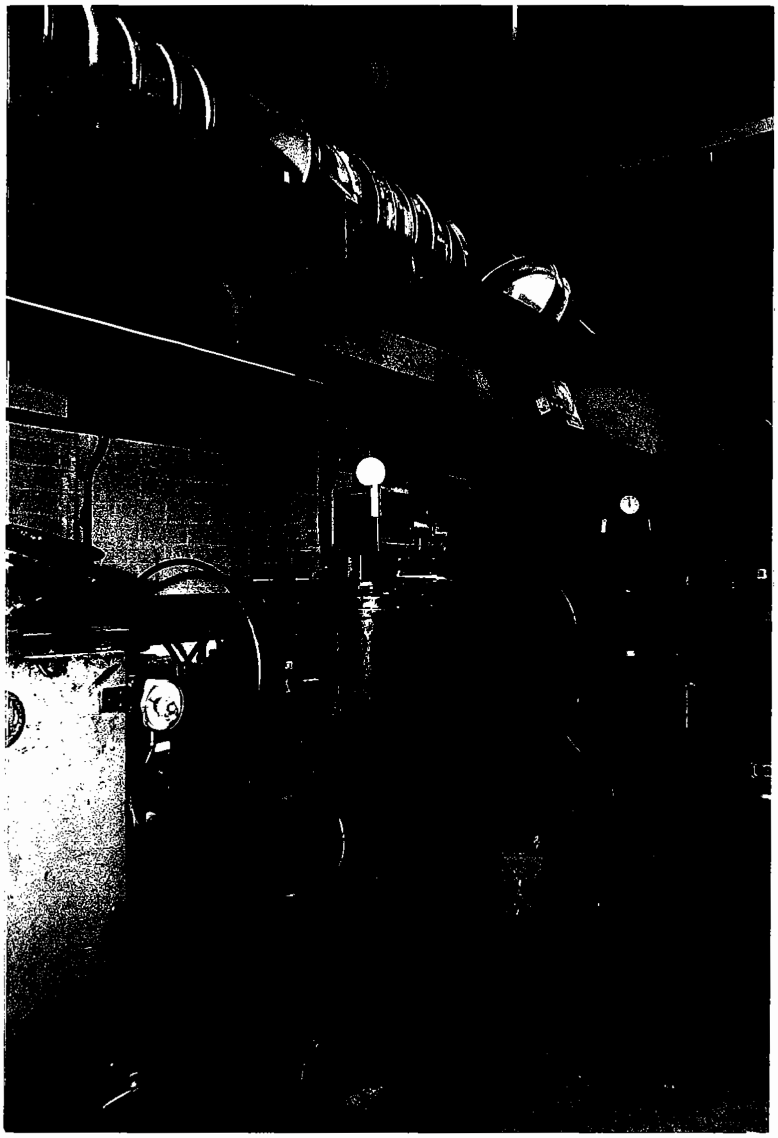


Figure 7.3 General view of the Nottingham University mixing facility.

Figure 7.4 Paddle details of a single mixer.

7.1.3 Testing

The tests performed on the specimens can be categorized into volumetric and mechanical property determinations. The volumetric proportions which must be obtained are the air voids (V_V), the binder volume (V_B), and the voids in the mineral aggregate, (VMA). Following density measurements these parameters can be calculated using knowledge of the specific gravities of aggregate and binder.

The mechanical properties measured were the elastic stiffness and deformation resistance of the specimens.

7.1.3.1 Volumetric proportions

Densities were obtained through gravimetric means. Specimens were not sealed prior to immersion in water. The volumetric proportions of the thirty specimens manufactured are displayed in Table 7.2, with the letter in the specimen nomenclature referring to the gradation type. Six specimens were manufactured corresponding to the Arabian mix formulation, (A1-A6) as part of establishing the compaction procedure. In general, the level of compaction varied between 94 and 100PRD for each formulation as anticipated by the different conditions of compaction.

Table 7.2 Volumetric proportions of mix formulations

SPEC	M_B (%)	V_v (%)	V_B (%)	VMA (%)	PRD (%)
A1	4.3	1.9	11.3	13.2	100.0
A2	4.3	3.8	11.2	15.0	98.0
A3	4.3	4.7	11.0	15.7	97.1
A4	4.3	5.7	10.9	16.6	96.1
A5	4.3	7.1	10.7	17.8	94.7
A6	4.3	9.2	10.5	19.7	92.6
A7	3.8	4.1	9.9	14.0	100.0
A8	3.8	6.9	9.6	16.5	97.1
A9	3.8	9.4	9.3	18.7	94.4
A10	4.8	1.3	12.6	13.9	100.0
A11	4.8	3.6	12.3	15.9	97.7
A12	4.8	6.6	12.0	18.6	94.6
B1	4.3	0.3	11.6	11.9	100.0
B2	4.3	2.1	11.3	13.4	98.2
B3	4.3	5.1	10.9	16.0	95.3
B4	3.8	1.4	10.2	11.6	100.0
B5	3.8	3.8	9.9	13.7	97.6
B6	3.8	7.1	9.5	16.6	94.3
B7	4.8	0.1	12.8	12.9	100.0
B8	4.8	1.5	12.6	14.1	98.6
B9	4.8	3.4	12.4	15.8	96.7
C1	4.0	0.0	10.8	10.8	100.0
C2	4.0	2.0	10.6	12.6	98.0
C3	4.0	4.1	10.3	14.4	95.9
C4	3.5	0.7	9.5	10.2	100.0
C5	3.5	4.5	9.1	13.6	96.2
C6	3.5	6.3	8.9	15.2	94.4
C7	4.0	0.9	10.7	11.6	100.0
C8	4.0	4.0	10.4	14.4	96.8
C9	4.0	7.2	10.0	17.2	93.6

7.1.3.2 Mechanical tests

The elastic stiffness of the specimens was measured using the repeated load indirect tensile test at a temperature of 20°C. Two tests were performed on each specimen on different axes, and the results averaged to give a mean value of stiffness. Deformation resistance was assessed using the uniaxial creep test. This mode of test was chosen in preference to a repeated load axial test so that results could be compared directly with creep tests which had been performed in the Arab state. Test conditions involved the application of 100kPa stress acting on the specimen for 1 hour at 40°C.

7.1.4 Results

An inspection of the volumetric proportions of the specimens shows that the manufacturing method was successful in fabricating samples with varied levels of compaction. The intermediate compaction level typically gave relative compactions of between 97 and 98 PRD which is slightly higher than what may be expected on site. The low level of compaction exhibited PRD's of between 94 and 95, allowing an assessment to be made of the overall effect of compaction on the different mix formulations. Specimens compacted to 100PRD exhibited very low void contents particularly with gradations B and C. This is not an unexpected result concerning the maximum density gradation, (grading B) but appears anomalous with the coarse gradation, (grading C). This grading has a dominant coarse aggregate content and should provide an aggregate matrix with a higher VMA than the maximum density grading. However, the choice of a high filler content for grading C must be the factor which has influenced the volumetric proportions, reducing the interparticle voids, hence densifying the matrix.

The RLIT test results of elastic stiffness values are shown in descending order of magnitude in Table 7.3.

Table 7.3 Ranked values of Elastic stiffness of test specimens

SPEC	PRD (%)	ELASTIC STIFF. (MPa)
C4	100.0	6200
B4	100.0	5900
C1	100.0	5500
A7	100.0	5300
A1	100.0	5200
B5	97.6	5200
C6	94.4	4600
C5	96.2	4600
B1	100.0	4400
C7	100.0	4400
A3	97.1	4300
C8	96.8	4200
A8	97.1	4000
C2	98.0	4000
B6	94.3	4000
A2	98.0	3800
A11	97.7	3600
C3	95.9	3500
B2	98.2	3500
B3	95.3	3400
B7	100.0	3400
B8	98.6	3100
A4	96.1	2800
A5	94.7	2700
A10	100.0	2700
B9	96.7	2700
C9	93.6	2600
A12	94.6	2300
A9	94.4	1900
A6	92.6	1700

The highest values of stiffness were measured on five of the specimens compacted to 100PRD, although gradings A and B at binder contents of 4.8% exhibited inferior stiffness at compaction level 1. Of the specimens manufactured at compaction levels 2 and 3, B5, C5 and C6, which all had binder contents of 3.8% exhibited the highest values of stiffness of between 4600 MPa and 5200 MPa. All the specimens with stiffness values greater than 4600 MPa had low VMAs relative to the other mixes. This corroborates stiffness prediction methods where VMA and S_b are the variable parameters which dictate the mix stiffness, with a reducing VMA resulting in an increased mix stiffness. The relationship between measured mix stiffness and VMA is shown in figure 7.5 which is in accordance with the general rule of decreasing VMA corresponding with an increasing stiffness. However, from the scatter of results and examination of Table 7.3 it can be seen that other factors must influence the value of elastic stiffness, underlining the need to measure this property through a form of simple test.

The uniaxial creep test results are shown in descending order of deformation resistance in Table 7.4, with specimen A8 exhibiting the least strain at the end of the test.

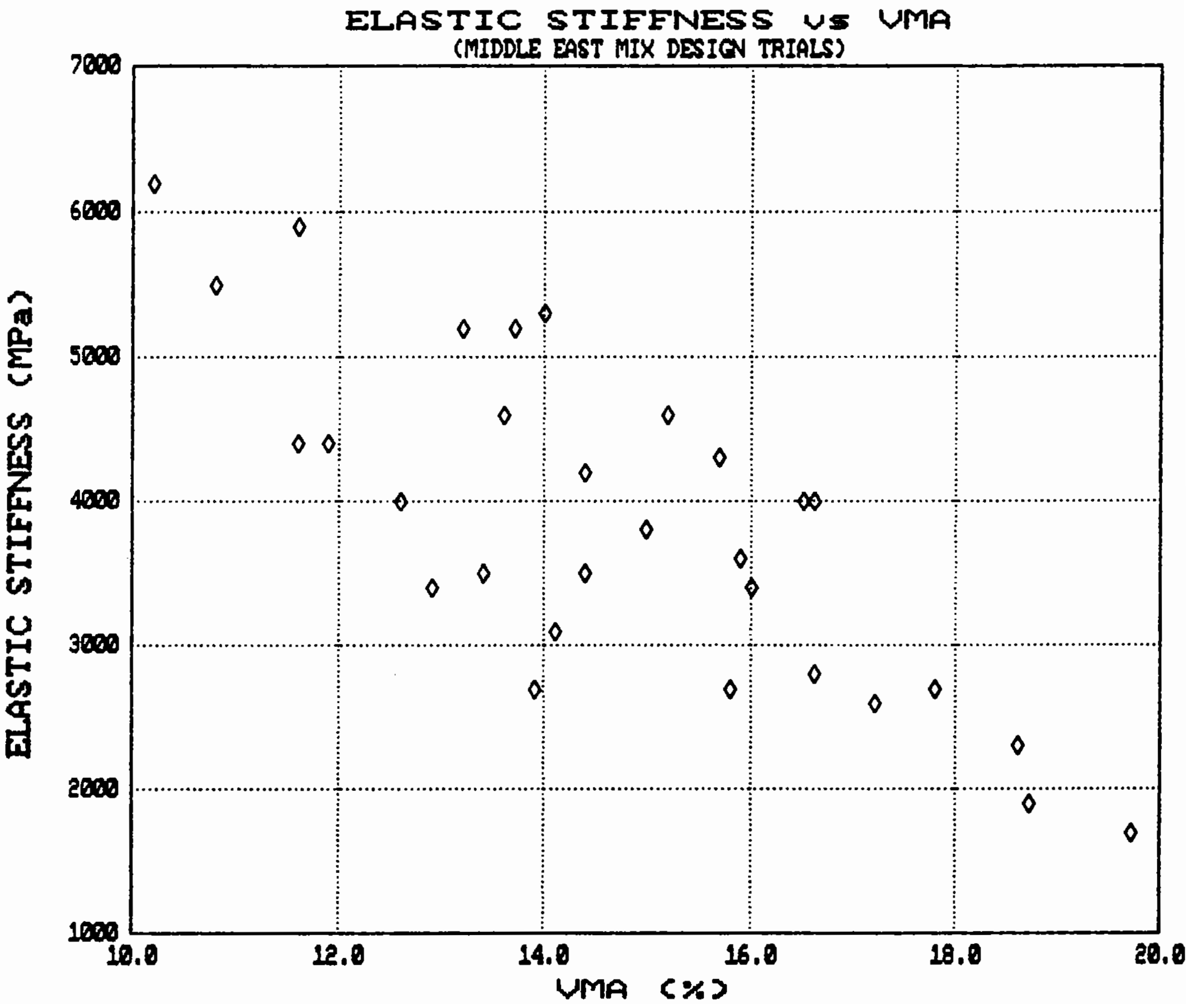


Figure 7.5 The relationship between VMA and measured values of elastic stiffness (Middle East mix design trials).

Table 7.4 Ranked values of permanent axial strain of test specimens

SPEC	PRD (%)	AXIAL STRAIN(%)
A8	97.1	0.079
C3	95.9	0.102
C2	98.0	0.109
C5	96.2	0.114
A7	100.0	0.118
A1	100.0	0.126
A2	98.0	0.134
B7	100.0	0.139
B4	100.0	0.141
B5	97.6	0.145
C4	100.0	0.149
A3	97.1	0.158
C9	93.6	0.160
A4	96.1	0.163
A9	94.4	0.167
B6	94.3	0.180
A11	97.7	0.186
B3	95.3	0.192
C6	94.4	0.195
A6	92.6	0.195
B1	100.0	0.197
A12	94.6	0.203
A5	94.6	0.213
C1	100.0	0.234
A10	100.0	0.259
C8	96.8	0.263
B2	98.2	0.316
B8	98.6	0.321
C7	100.0	0.324
B9	96.7	0.343

This mix formulation used the Arabian gradation with a binder content of 3.8% and was compacted to 97.1% of its refusal density. The effect of compaction on the resistance to permanent deformation of the different mix formulations is illustrated in figures 7.6 to 7.8. Considering gradation A, there appears to be an optimum level of compaction for each binder content, with both under and over-compaction adversely affecting results. The reduced binder content of 3.8% quite clearly exhibits preferential deformation resistance characteristics. The only similar relationship with gradation B occurs with the leanest binder content, with apparently inverse relationships exhibited as the binder content increases. It is unlikely that the results corresponding to the binder contents of 4.3% and 4.8% are genuinely representative of the rheological properties of the mix, particularly at the 100 PRD level. As this particular grading creates a low VMA, high levels of binder content will flood the mix, effectively over-lubricating the aggregate matrix. The application of a very high compactive effort will result in the binder, and some fines, migrating to the surface of the specimen leaving an internal matrix which is artificially lean. Therefore, after the sample is cored and trimmed, the test specimen is not representative of the original mix formulation. This is the mechanism which has occurred with specimens B1, B7 and may possible have effected specimens A1 and A10, although to a less significant degree.

Good deformation resistance has been achieved from specimens C2, C3 and C5, although these formulations appear particularly sensitive to change in compactive effort.

Making a combined assessment of the mechanical properties, specimens B5 and C5 exhibit good deformation resistance characteristics and high values of elastic stiffness. A1 and A7 also compare favourably, but the deformation properties of A1 must be questioned because of the bleeding effect at 100PRD.

7.1.5 Conclusions from mix design trials

The investigation into the nine mix formulations demonstrated how grading, binder content and compactive effort can effect the mechanical properties of bituminous mixes. The distress mode in the Arab state which was causing disruption to their highway infrastructure was that of rutting. The mix used in their pavement construction had a binder content of 4.3%, and an air void structure compliant with their design specifications of between 3% and 5%. However, material was densifying to a zero void content in the wheel tracks, causing bleeding of the binder and severe rutting. The same mechanism occurred in the laboratory when compacting specimens, which had a high binder content, to 100PRD. Quantification of the change in deformation resistance could not be made because of the effective alteration of the composition of the test specimens at this level, but the significance of over-compaction, and the volumetric proportions of such mixes becomes clear.

Figure 7.6 shows that a reduced binder content from the Job Mix Specification, exhibits less deformation over the range of compaction levels at which specimens were manufactured. Optimum deformation resistance was coincident with a PRD of 97.1 and a binder content of 3.8%, with axial strain increasing as more compaction was applied. The significant observation which can be made is that the void content of specimen A7, (M_B=3.8%,PRD=100) is 4.1%, whereas the void content of

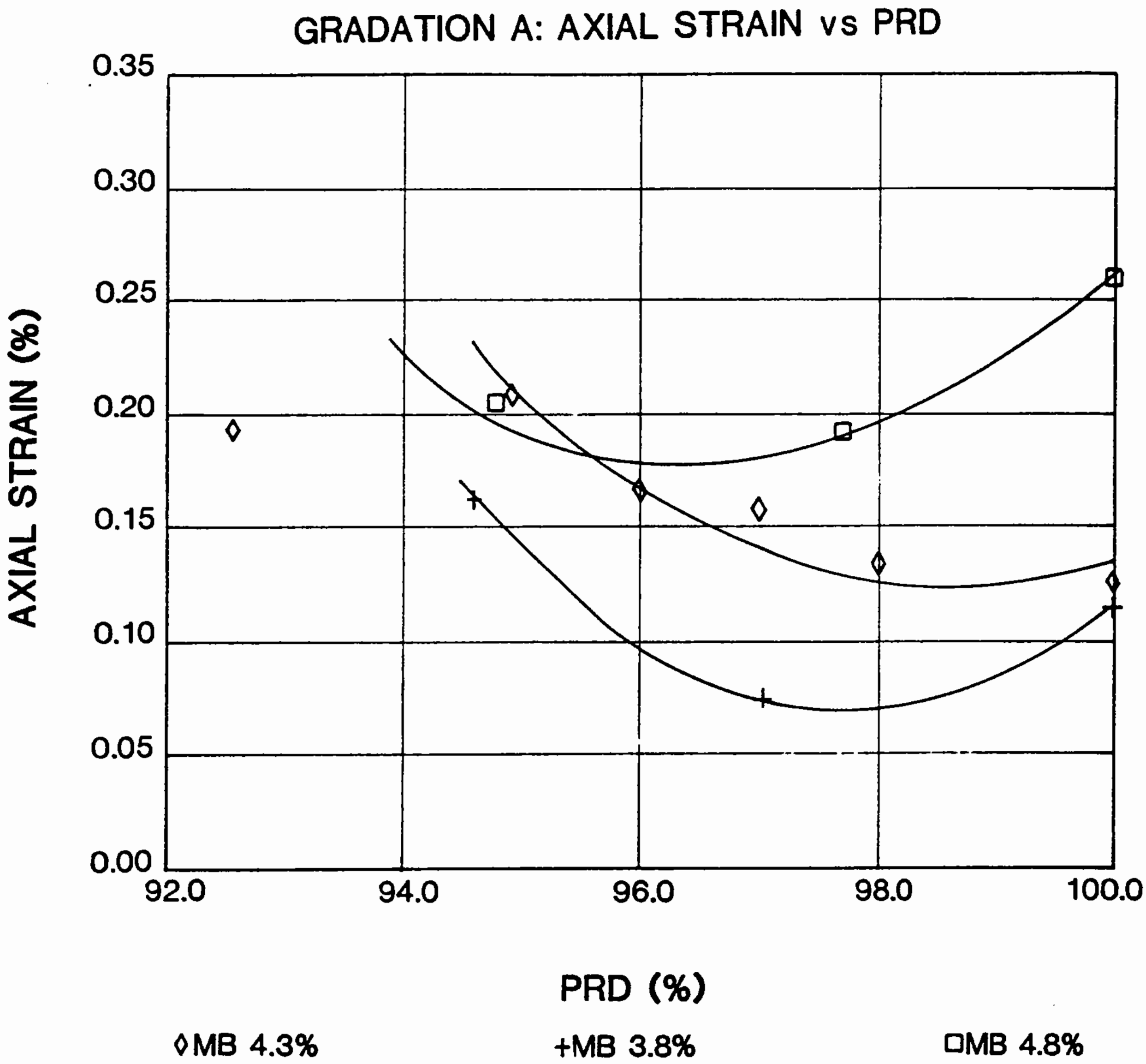


Figure 7.6 Influence of PRD on resistance to permanent deformation (Gradation A, Middle East mix design trials)

specimen A1 representing the site material at 100PRD, was calculated as 1.9%. Hence, using the same grading with a reduced binder content should not only exhibit improved deformation resistance characteristic, but also be capable of withstanding subsequent compaction in the wheel tracks without the mix flushing and rutting. The same conclusions can be drawn from figures 7.7 and 7.8 representing the other two gradations.

In general, it would appear that a leaner, more open mix would be a desirable formulation for the asphaltic concrete to be used in the Arab state. This can be achieved by changes to the aggregate grading and the binder content. The Job Mix Specification had avoided the maximum density gradation by adhering to a grading on the fine side of the Fuller curve, which had the result of increasing the quantity of fine aggregate in the mix at the expense of the attenuation of the coarse aggregate component. This increases the volume of voids within the aggregate matrix but does not accentuate the coarse aggregate skeleton which aids resistance to permanent deformation. The dominant fine aggregate matrix requires a higher binder content for mix lubrication and durability than a coarser grading, which is to the detriment of deformation resistance.

A tentative recommendation would be to use an aggregate grading slightly coarser than the maximum density curve, (corresponding to an exponent 'n' of 0.5 or 0.55 in equation 7.1) and assess the effect of variations in binder content and compaction level on the mechanical properties of the mix. From the results obtained in the Arab mix design trials, it would appear that the philosophy of minimising the binder content without compromising fatigue resistance certainly enhances the resistance to

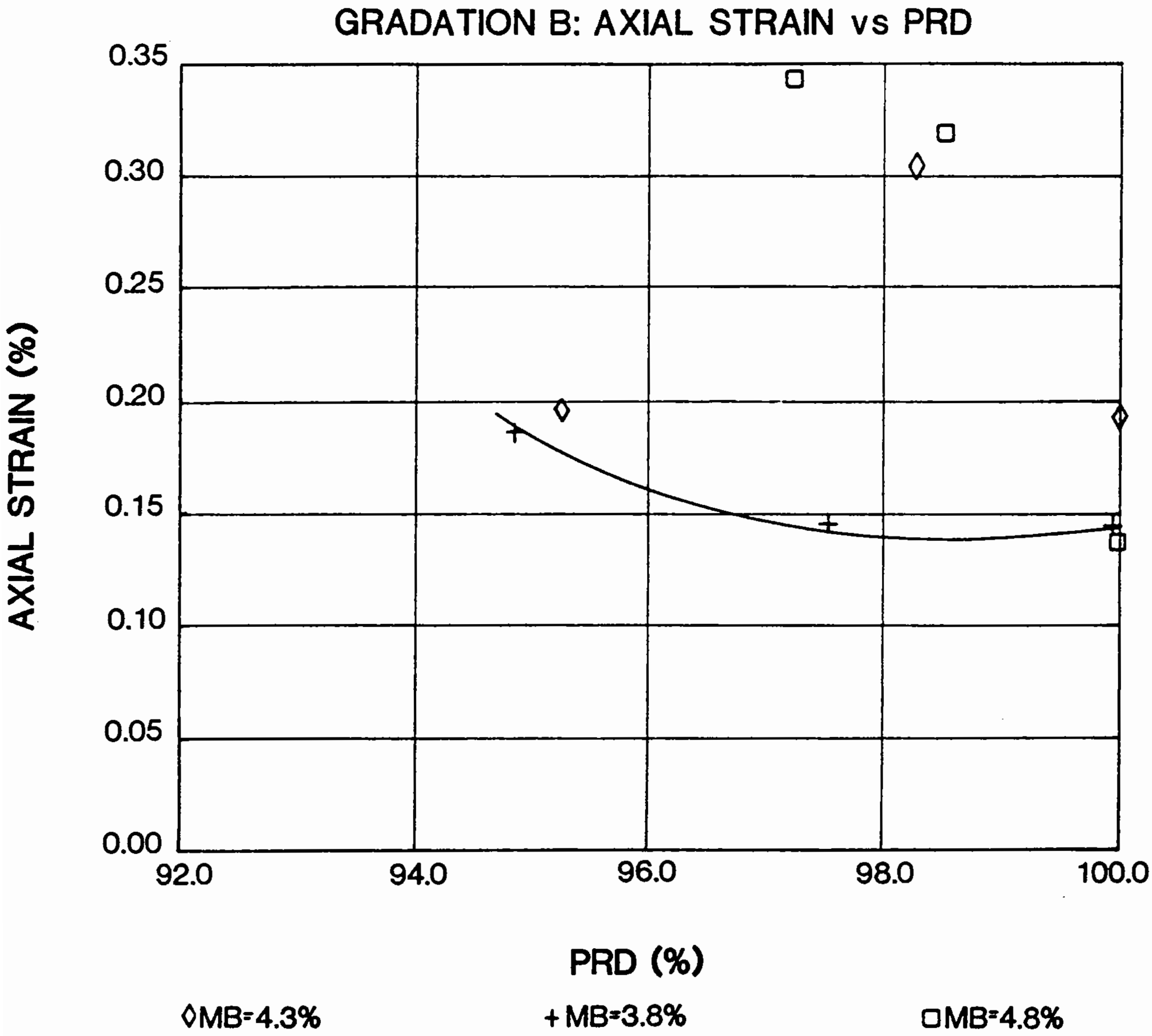


Figure 7.7 Influence of PRD on resistance to permanent deformation (Gradation B, Middle East mix design trials)

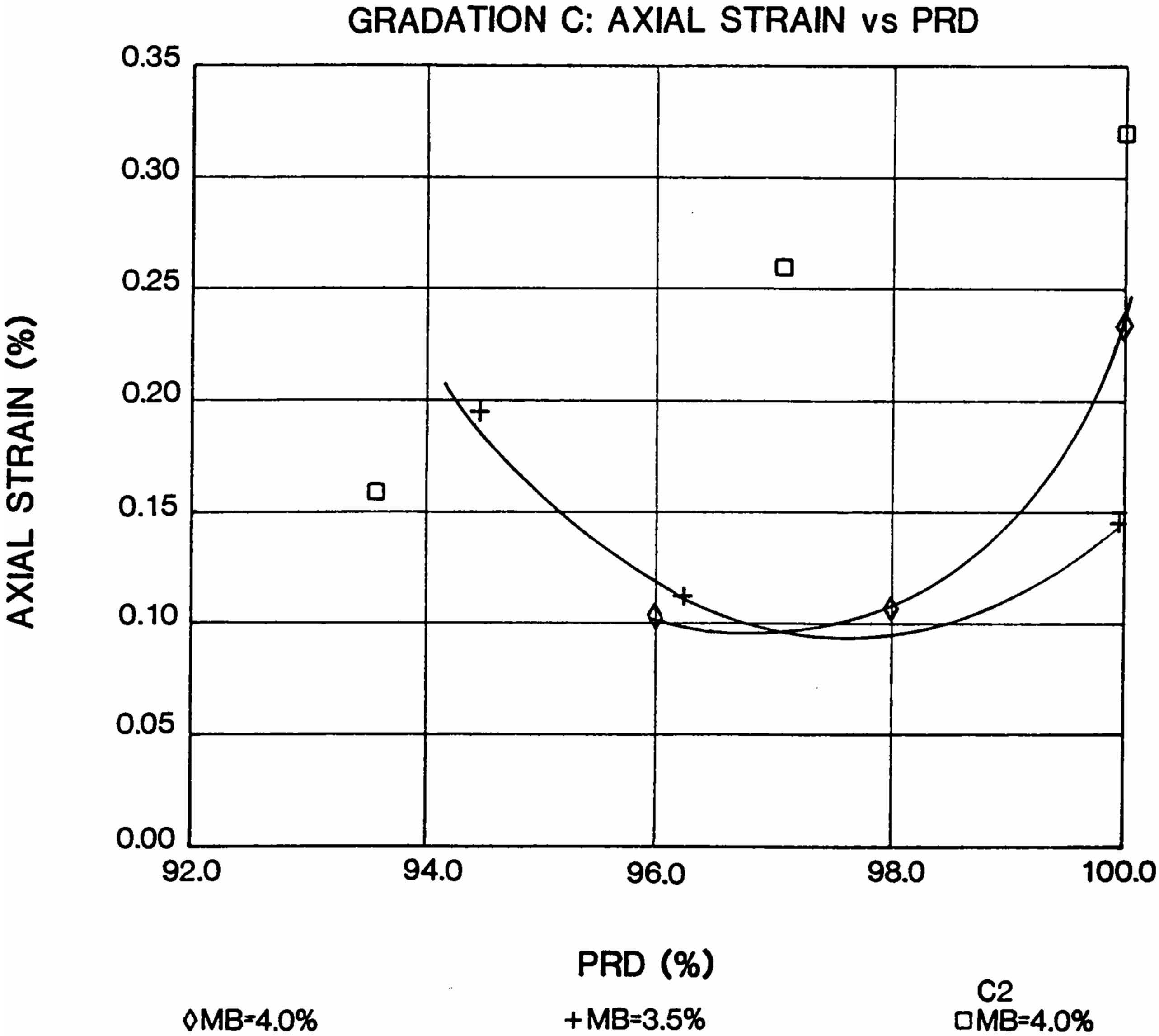


Figure 7.8 Influence of PRD on resistance to permanent deformation (Gradation C, Middle East mix design trials)

permanent deformation. Attention must be directed to the volumetric proportions of mixes at the 100PRD level to ensure that excessive compaction, which can take place in the wheel tracks after construction, does not lead to the condition where the voids filled with binder reach a critical state causing bleeding and subsequent deformation of the pavement. Careful consideration must also be given to the test specimens at 100PRD, to assess whether any binder has migrated from the aggregate matrix, as in the case of specimens B1 and B7, leaving an unrepresentative lean mix formulation.

If this is achieved, subsequent checks must be made on fatigue strength and durability characteristics of the optimum mix formulation to try and avoid serious pavement distress through cracking.

CHAPTER EIGHT
TOWARDS A NEW METHOD OF MIX DESIGN

This chapter considers the developments which were made to the embryonic design procedure introduced through the Arabian mix design trials. It documents the thinking behind the rationale of the approach to the problem and introduces several new ideas, which, although simple in concept have considerable implications for mix design, and to highway engineering in general. Emphasis has been directed at maintaining a simple format, whilst still including the necessary salient features. This would hopefully lead to a procedure which satisfies the aims of mix design, and provides a practical step by step process, the implementation of which should be relatively straight forward.

8.1 MATERIALS

Experimentation and development work was performed utilising aggregates from two sources, both of which supply paving materials for highway contracts. Two rock types were selected, in order to evaluate the effect of different aggregate characteristics on the mix design procedure. All bitumens used were straight run penetration grades.

8.1.1 Granite Aggregate

Granite aggregate was obtained from Bardon Hill Quarry in Leicestershire. Material was taken from six bin sizes, (passing 28mm, 20mm, 14mm, 10mm, 6.3mm and 3.35mm), and samples of filler were also taken. Grading analyses were carried out on each bin size, the results of which are presented in Table 8.1. Particle index evaluations to BS 812 part 1 (89) or ASTM D 3398 (90) were not performed, only a visual inspection of the angularity of the aggregate was made. Compaction tests were carried out on each bin size, (section 3.3.3.1) which indicated void contents of approximately 42% for the compacted aggregate in a dry state, giving an angularity number of 9 (89), corroborating the visual analysis.

Table 8.1 Gradings within quarry bin sizes for granite aggregate.

Sieve size (mm)	Nominal size of aggregate (mm)					
	% Passing 28	% Passing 20	% Passing 14	% Passing 10	% Passing 6.3	% Passing 3.35
28	90	100	100	100	100	100
20	18	95	100	100	100	100
14	0	5	95	100	100	100
10	0	1	20	95	100	100
6.3	0	0	4	9	95	100
3.35	0	0	0	2	6	98
2.36	0	0	0	0	2	88
1.18	0	0	0	0	0	62
0.6	0	0	0	0	0	40
0.3	0	0	0	0	0	24
0.15	0	0	0	0	0	23
0.075	0	0	0	0	0	8

8.1.2 Limestone aggregate

Samples of limestone aggregate were obtained from Tarmac Roadstone's Dene Quarry in Derbyshire. Material was taken from seven bin sizes in decreasing size from 28mm. Table 8.2 shows the gradings within each size fraction. The limestone aggregate incorporates one extra bin size covering material passing the 3.35mm sieve which facilitates close control of the aggregate when blending the bin sizes to a target gradation.

Figure 8.1 illustrates a visual comparison between the two aggregate types which have been used in this project.

Table 8.2 Gradings within quarry bin sizes for limestone aggregate

Sieve size (mm)	Nominal size of aggregate (mm)						
	% Passing 28	% Passing 20	% Passing 14	% Passing 10	% Passing 6.3	% Passing 3.35	% Passing 2.36
28	95	100	100	100	100	100	100
20	10	93	100	100	100	100	100
14	1	6	94	100	100	100	100
10	0	1	17	98	100	100	100
6.3	0	0	2	12	95	100	100
3.35	0	0	0	2	35	93	100
2.36	0	0	0	0	19	70	89
1.18	0	0	0	0	10	41	63
0.6	0	0	0	0	8	28	46
0.3	0	0	0	0	7	21	34
0.15	0	0	0	0	6	17	24
0.075	0	0	0	0	4	14	10

8.1.3 Bitumen

Air blown bitumen (20 pen) and straight run 200 pen binder were supplied by Mobil Oil Co. All bitumen used within the project corresponded to conventional penetration grades (40), blended from the two types supplied.

Fig 8.1

105 – 106.

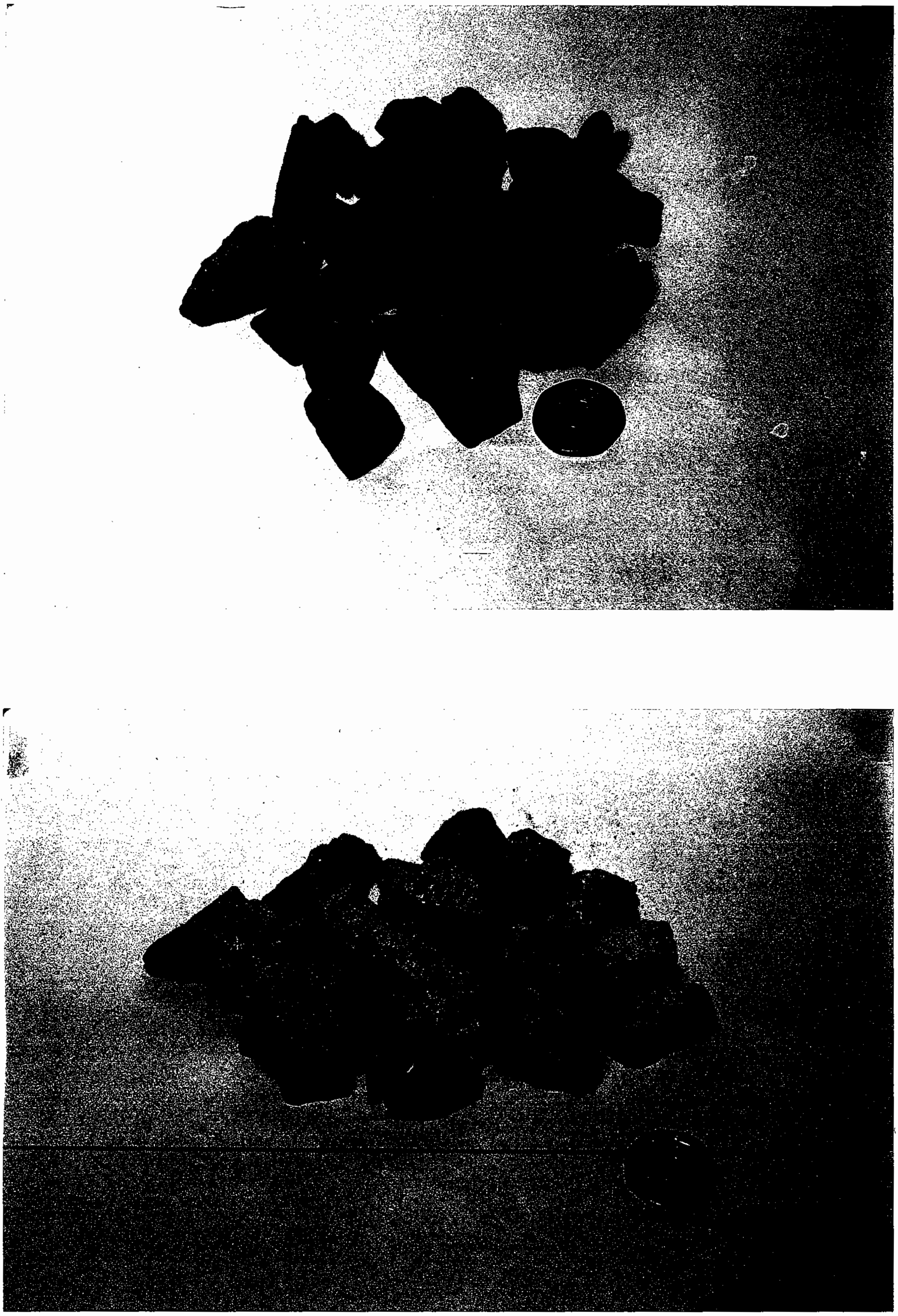


Figure 8.1 Visual comparison of granite aggregate (top) and limestone aggregate (bottom).

Mixing and compaction temperatures should be specified according to binder viscosities. In a paper on 'characterising asphaltic bitumens', Heukelom (91) described both the penetration and viscosity of binders as functions of temperature using the equation:

$$\frac{-5.42 \log\left(\frac{\eta}{13000}\right)}{8.5 + \log\left(\frac{\eta}{13000}\right)} = A\,(T - T_{800}) \qquad 8.1$$

$$\text{where } A = \frac{\log\,(\text{pen at } T) - \log 800}{T - SP}$$

and η = viscosity (poises)

T = Temperature (°C)

SP = Ring and Ball softening point temperature (°C)

A refers to temperature susceptibility.

Rearranging equation 8.1 allows the temperature function to be isolated.

$$\text{from} \quad \frac{-5.42 \log\left(\frac{\eta}{13000}\right)}{8.5 + \log\left(\frac{\eta}{13000}\right)} = \left(\frac{\log P - 2.9}{25 - SP}\right)(T - SP)$$

$$\text{re-arranging } T = \frac{-5.42 \log\left(\frac{\eta}{13000}\right)}{8.5 + \log\left(\frac{\eta}{13000}\right)} \cdot \left(\frac{25 - SP}{\log P - 2.9}\right) + SP \qquad 8.2$$

Mixing asphalt and aggregate should take place at a binder viscosity of 1.5 poises, and site compaction requires a maximum bitumen viscosity of 2.5 poises. Substituting these values into equation 2 results in the following:

$$\text{Mixing temperature } T_m = \frac{4.7(25\text{-}SP)}{\log P\text{-}2.9} + SP \qquad 8.3a$$

$$\text{Compaction temperature } T_c = \frac{4.2(25\text{-}SP)}{\log P\text{-}2.9} + SP \qquad 8.3b$$

8.2 DEVELOPMENT PROCEDURE

The approach taken for the mix design trials described in Chapter 7 laid out the basic principles which would formulate a new procedure for the design of dense bituminous concrete mixtures. The format addressed the three fundamental parameters of aggregate gradation, binder content and level of compaction, which dictate the overall performance of the bituminous layers. Analysis of the mix formulations incorporated a study of the volumetric proportions, and mechanical tests to evaluate elastic stiffness and deformation resistance properties. Therefore, mixes could be quantified in fundamental terms concerning the end product characteristics of the material, and the effect of variations in binder content or compaction level for a particular mix formulation could be assessed. The philosophy of minimising the binder content without compromising fatigue strength and durability has been maintained throughout the project and represents a major departure from traditional thinking.

8.2.1. Aggregate grading

The properties of aggregate particles from different sources were discussed in Chapter 3, where an investigation was carried out into the potential of designing an aggregate grading according to the inherent properties of the material from that source. Theoretically, this approach should provide the ideal method of selecting a gradation, but a practical solution to the problem was not found. Currently, gradations are either specified by an envelope which has been empirically developed, or one which is based on the maximum density curve. The envelopes usually incorporate wide tolerances at each sieve size which facilitate easy compliance for the supplier but can also result in considerable variations in the mechanical properties of, what is nominally, the same formulation. It was decided to identify aggregate gradings by individual curves which would give target gradations to which aggregates must be blended to the best possible fit. This would not change current practice, as mixing plants follow a blending procedure to meet envelope requirements but it would result in tighter control over the gradings.

A continuous aggregate grading must be quantified in some manner in order that deviations from the target gradation, which will occur in practice, may be analysed. Gradation C in the Arab mix design trials was specified by equation 7.1.

$$P = \frac{(100 - F)\,(d^n - 0.075^n)}{(D^n - 0.075^n)} + F$$

where P = percentage of material passing a sieve of size d mm

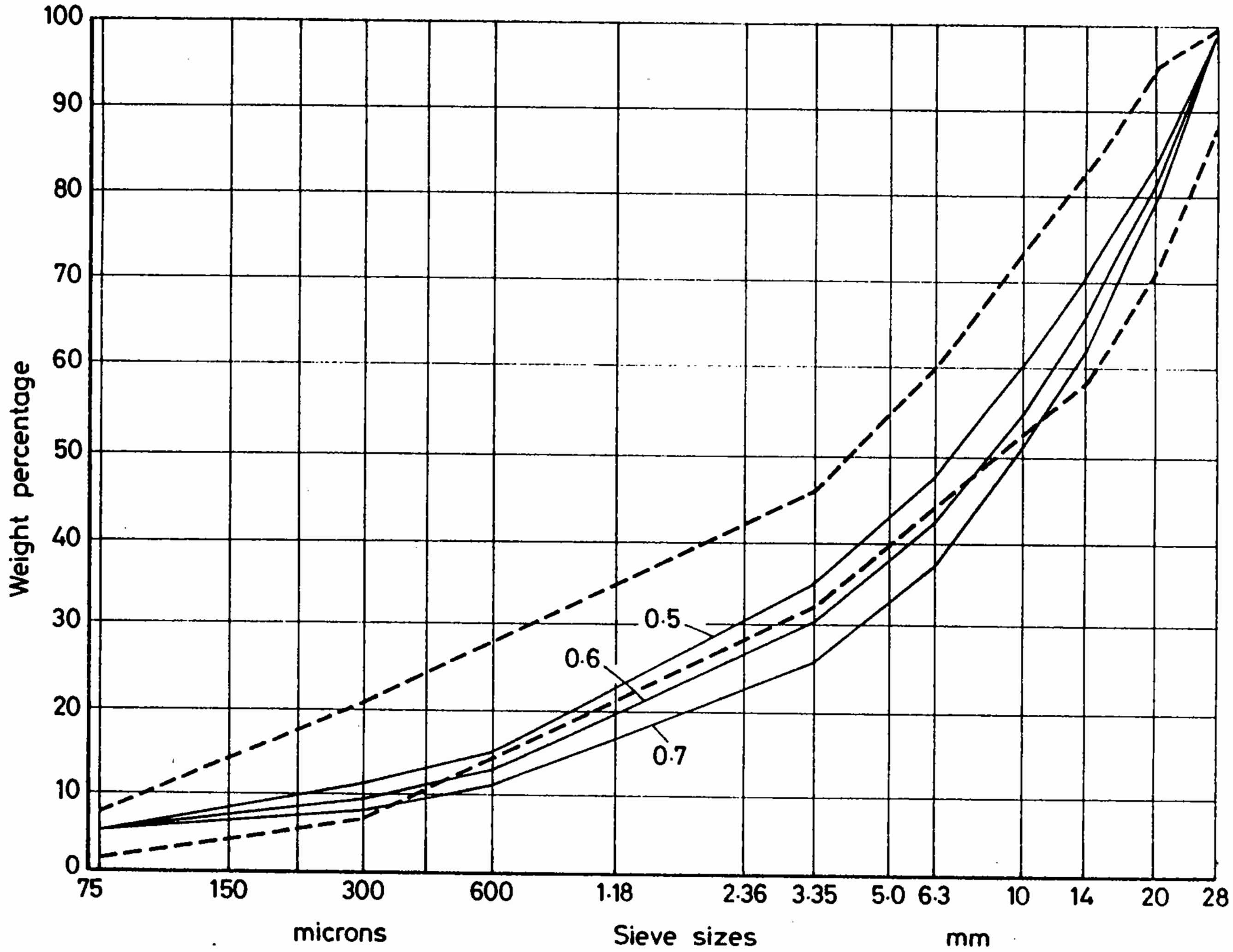
D = maximum particle size, mm

F = Filler content, %

n = an exponent between 0 and 1

This equation allows the construction of a continuous grading between an upper bound value, (the maximum particle size) and a lower bound value, (the filler content). The use of the exponent dictates the general format of the curve, with n=0.45 corresponding to the maximum density gradation. It was considered that the 'Fuller' curve should be avoided as a particularly dense grading may be subject to flushing, if a state of over-compaction is reached in the wheel tracks. The grading may be 'opened up' by an adjustment to the fine side of the maximum density curve (decreasing exponent) or an adjustment to the coarse side (increasing exponent). A gradation finer than the 'Fuller' curve has the effect of increasing the quantity of fine aggregate in the mix, and decreasing the coarse aggregate component. This in turn requires an increase in the binder content to coat the aggregate and maintain fatigue strength, which can potentially have the result of producing a deformable mix.

It is advocated therefore that a series of aggregate gradations on the coarse side of the maximum density curve should be considered for mix design purposes. This will have the effect of maximising the role of the coarse aggregate within the particle matrix, and will help conform to the philosophy of minimising the binder content. It is suggested that three gradations corresponding to exponent values of n = 0.5,0.6 and 0.7 be assessed as part of the mix design procedure. These grading curves are shown in Figure 8.2, corresponding to a maximum particle size of 28mm and a filler content of 6%. The BS 4987 envelope for a 28mm dense roadbase macadam has also been included on the Figure for comparison. It can be seen that over the central part of the gradings, between the 6.3mm and 0.6mm sieves, the 0.5 and 0.6 curves lie close to the coarse or lower limit of the empirical envelope, with the 0.7 curve representing a notably



Grading curves corresponding to n = 0·5 to n = 0·7
BS 4987 envelope

Figure 8.2 The grading curves resulting from the use of exponents n=0.5, 0.6 and 0.7 in the equation $P = \frac{(100-F)(d^n - 0.075^n)}{(D^n - 0.075^n)} + F$ and the envelope for 28mm Roadbase Macadam from BS4987.

coarser gradation than currently accepted. At the limits of the gradings, all curves are within current specifications.

8.2.1.1 Filler content

The identification of individual aggregate grading curves requires the engineer to fix a value of the filler content. The grading envelope in Figure 8.2 denoting existing specification, has an acceptance range of between 2% and 8% for the quantity of material passing the 75 micron sieve. Throughout the history of UK pavement engineering, very little attention has been paid to the influence of the role of filler material in a bituminous mixture, with specifications resulting from empirically based grading envelopes.

The concept that filler material may influence the behaviour of bituminous mixtures was considered as early as 1947 (92), where the role of different mineral fillers was assessed in terms of tar/filler ratios for surfacing materials. This early research caused investigators outside of the UK to question whether the behaviour of filler was associated with the aggregate, or whether it contributed to the properties of the binder, effectively becoming independent of the aggregate gradation. There can be little doubt that at least a proportion of the mineral filler combines with the binder, making an 'asphalt mortar'(93) influencing the binder properties, and the degree of influence is affected by the physical and chemical composition of the type of filler (93,94). Most of these investigations have concentrated on the effect of different filler types in sandsheet mixtures, which are representative of wearing coarse materials and not roadbase layers. It is likely that small variations in the quantity of filler material around a medium value of approximately 6%, will not significantly alter the properties of roadbase and basecourse layers. Recent studies have shown that the effect of varying the filler content between 5% and 10%, in a continuously graded material, is considerably less than the effect of making small adjustments to the binder content (95).

Considering current practice within the United Kingdom and abroad and the findings of the aforementioned research, it is recommended that aggregate gradation with a filler content of 6% are selected for assessment as part of the mix design procedure. As this parameter is adjusTable, it does facilitate change which may be necessary for particularly dusty aggregates, where it may become advisable to reduce filler content.

8.2.2 Adherence to target gradations

The introduction of a series of individual grading curves to specify the aggregate content of a bituminous mix may appear an impractical recommendation when absolute adherence to the suggested curves will not occur in practise. Good control over the aggregate grading is determined by the gradation of material within the quarry bin sizes, and the number of bins required to constitute the full grading. Table 8.1 showed that Bardon Hill Quarry used six bins to make up a 28mm top size continuous grading, with five bins containing single size material, and one bin providing a graded material from the 3.35mm sieve down to material passing the 75 micron sieve. For the same overall gradation, Dene Quarry utilised seven bins, four of which contained single size material, and three containing graded material. The individual bin sizes must then be blended together to a grading as close to the target as possible.

Existing procedures in both drum and batch mix plants use computerized systems to control the blending of bins to meet material requirements, so the input of target gradations corresponding to n=0.5, 0.6 and 0.7 would not make a significant deviation from current practice. The accuracy to which a target gradation may be approximated is dictated by the inherent gradation within the available bins.

The use of single sized materials for each bin size greater than 3.35mm ensures that a target gradation may easily be reproduced over the coarse section of the curve, but problems may be encountered concerning the fine material if there is only one bin size available to meet the target specification. For the recommended gradings of n=0.5, 0.6 and 0.7, the quantity of material passing the 3.35mm sieve ranges from 35%, to 30%, to 26% respectively. The Bardon Hill material has therefore one bin size to cater for nearly one third of the aggregate within the gradation, with only filler material available as an extra supplement. The approximation to the n=0.5 curve, below the 3.35mm sieve for the two aggregate types is shown in Figure 8.3, where it can be seen that a reasonable adherence is achieved for both aggregates at the optimum blend. It must be stressed that the inherent gradings within bin sizes can vary from week to week because of the random nature of the crushing process at the quarry, which may significantly change the potential of the gradation to adhere to the intended target. In view of this consideration, it is desirable that more than one 'dust' product is available, as at the Dene Quarry, to compensate for such variations. Current British practice only includes two checks on the aggregate grading for the material finer than 3.35mm. These occur at the 300 and 75 micron sieves, which is inadequate for close control of the fine aggregate. If material fails to meet compliance through insufficient

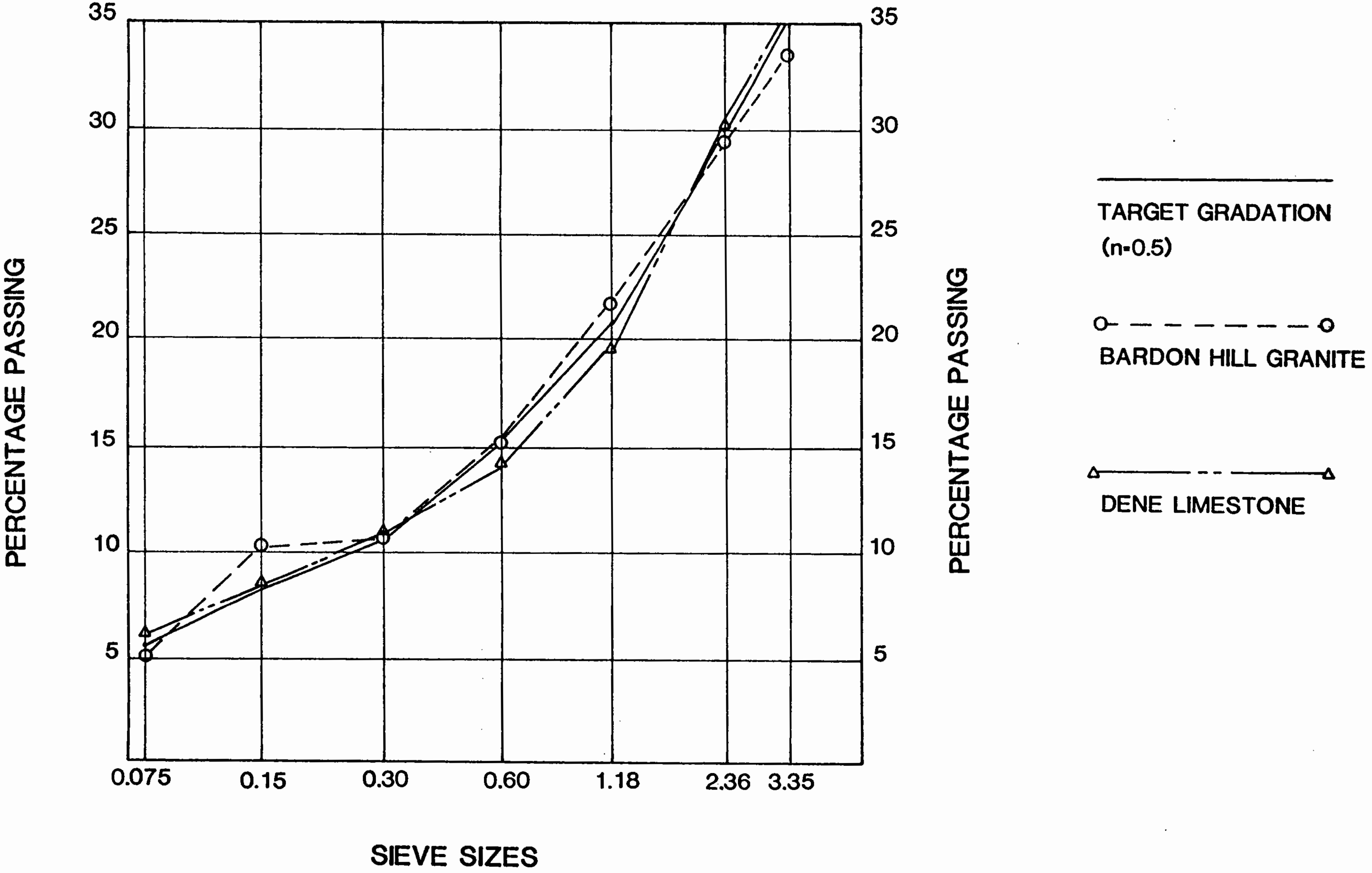


Figure 8.3 The approximations to the n = 0.5 curve for the two aggregate types using the available bin sizes from the quarries.

fine material, then more filler is added to the grading until compliance with the envelope is achieved.

Although this pragmatic solution can be used to satisfy a recipe specification, it is not performed with the aim of enhancing the mechanical properties of the mix. Figure 8.3 demonstrates that for the materials used in this investigation, a quite satisfactory approximation to an individual grading curve can be achieved, and although variations occur, the n=0.6 grading will invariably be coarser than the n=0.5 grading, and n=0.7 will provide the coarsest curve. Hence, any deviations which occur from the target gradations will be relative to the other curves, allowing representative comparisons to be made on the different gradings.

8.2.3 Binder content

A method of identifying the ideal or target binder content for a continuous aggregate grading has never been satisfactorily found without the empirical procedure of assessing a range of binder contents and applying non-fundamental mechanical tests to predict performance. Other methods have been proposed, such as the calculation of binder film thickness from assumptions of aggregate surface area (96) and retained quantities of oil when drained through a gradation (9), but all have failed to identify an exact value of binder content appropriate to the gradings under consideration. In the Arab mix design trials, binder content was assessed in a manner not dissimilar to the Marshall test, where a range of binder contents was investigated with each gradation. In the absence of another method, this would appear to be the most rational approach. For mix design purposes it is necessary to cover a range of binder contents which will facilitate the manufacture of lean and rich specimens, whilst incorporating the critical binder contents which will have the greatest influence on the performance of the mixes.

For a 28mm Dense Roadbase Macadam, current practice specifies a binder content of 4.0%± 0.5% with a crushed rock aggregate. Although exact precision cannot be expected, the tolerance applied to the target value of 4.0% allows a 12.5% variation to this value on the positive and negative side. Such a scope for variation in the binder content has the potential of making significant differences to the volumetric proportions and mechanical properties of mix formulation. Any mix design procedure must be able to evaluate the effect of variations in binder content in terms of mix performance, and binder content should be selected on this basis.

The advocated binder contents for use in the mix design procedure are 3.5%, 4.1% and by 4.7% by total mass of mix. These have been selected using current specification as a datum, but the overall range has been extended to increase the range of volumetric proportions which can result from these binder contents.

8.2.4 Specimen manufacture

The introduction of the PRD apparatus as a facility for manufacturing specimens of material proved to be a promising technique during the Arabian pavement trials. Aggregate of size 28mm was satisfactorily compacted to three different levels, using a realistic, kneading type of compaction. This apparatus succeeds in fulfilling a dual role concerning mix design, combining a fabrication method with a compaction to refusal facility. The distress mode which was evident in the Middle East indicated the importance of assessing the volumetric proportions of a mix formulation at a very high state of compaction. The potential of a mix becoming compacted to a state of zero voids and leading to flushing the binder, must be analysed for any given mix formulation and parameters of gradation and binder content should be adjusted to avoid this condition.

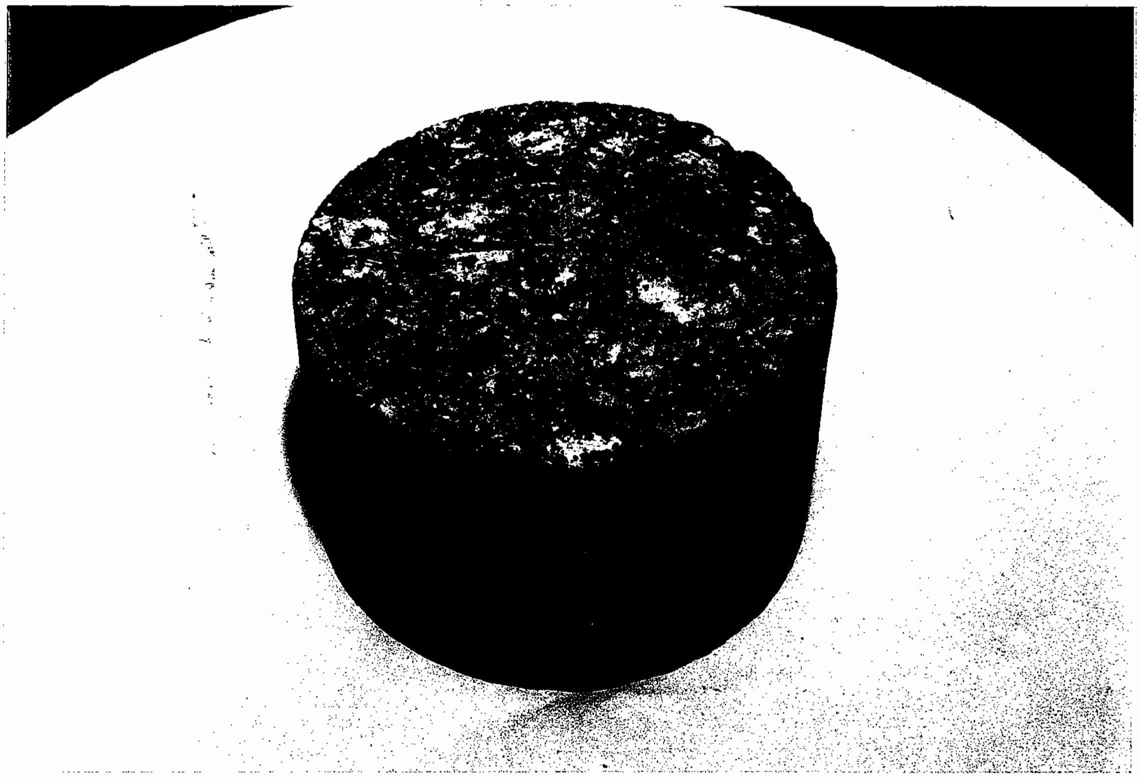
The Percentage Refusal Density test was introduced to UK practice by the Department of Transport in 1985 (67), as a method of assessing the compaction of dense roadbase macadam. The basis of the test is a comparison of the measured bulk density of a core of roadbase material, to the density of the same core after heating, and recompaction with a vibrating hammer, to a state of refusal. The relative density of the site core is then expressed as a percentage of its' refusal density. It must be stressed that the term 'refusal' is a relative expression, and does not indicate the ultimate state of compaction which can be achieved. 100PRD represents a very high state of compaction which will not be achieved through conventional rolling on site, but may be approached through subsequent compaction by traffic in the wheel tracks.

Use of this apparatus requires manual operation which means the reproducibility of the test may be subject to large variance. However, it was found that during preliminary investigations, different operators would achieve consistent densities for the same mix formulation when compacted to 100PRD. This is probably due to the fact that the conditions, under which refusal density is achieved, are so severe that it is unlikely that independent operators will significantly alter the final value. It is conceivable that different operators will achieve different densities at

Fig 8.4

116 - 117

(a)



(b)

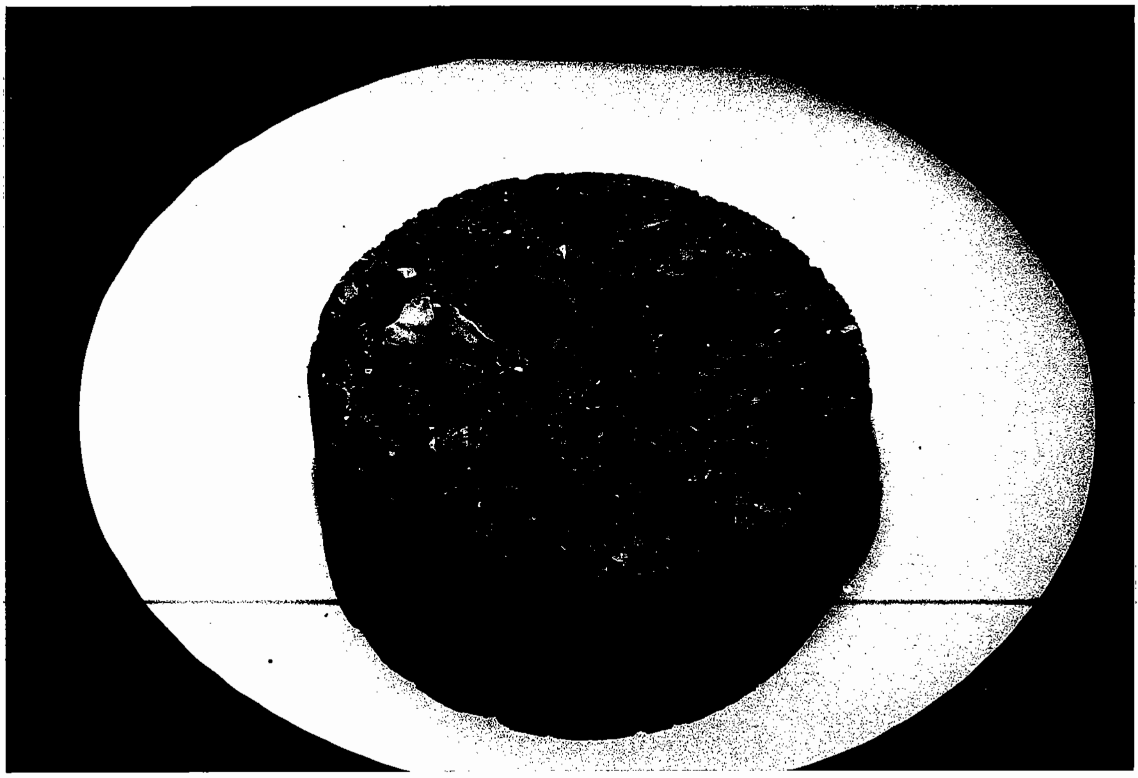


Figure 8.4 (a) Cored and trimmed specimen (100mm diameter)

(b) Trimmed specimen (150mm diameter)

compaction levels two and three, but this is of less importance as these specimens are intended to represent site materials of varying densities.

Cumbria County Council have made an effort to standardize the compactive effort applied during the PRD test by weighting the hammer. The vertical force transmitted to the specimen is therefore constant, and the operator's duty is to migrate the hammer and tamping foot over the surface of the mixture without contributing to the downward force. Such an approach could be adopted in an effort to produce a standard format for use of the PRD apparatus but insufficient evidence is available to confirm whether this would be necessary.

8.2.4.1 Preparation of test specimens

The PRD equipment fabricates specimens which are squat cylinders, 150mm in diameter by approximately 100mm deep. The nature of the compaction produces results in specimens which have an uneven upper surface making an unsuitable interface for mechanical testing. A modification to the original de-moulded specimen is required therefore to facilitate testing. The ends must be trimmed using a masonry saw to give a specimen approximately 70mm deep with clean, flat, parallel surfaces. Early investigations in development of the design procedure extracted a 100mm diameter core from the centre of the PRD specimen prior to trimming, resulting in a Marshall briquette type specimen. Examples of the two types of test specimens are illustrated in Figure 8.4a and 8.4b.

The requirement of modifying the PRD specimens prior to performing any tests adds an unfortunate complication to the procedure. One of the appealing elements of the Marshall test is that the method produces specimens which do not require further attention subsequent to their manufacture. It would appear, however, that the fabrication of material representative of that taken from site requires a manufacturing effort which does not always reproduce the same uniform specimen. In order to apply relevant mechanical tests to bituminous materials, certain conditions concerning the surfaces of the materials must be satisfied, which requires the trimming and possible coring of specimens in the case of the PRD method.

After preliminary investigations it was decided to omit the coring process and use specimens as shown in Figure 8.4b, in order to simplify the preparation procedure.

8.3 THE FORMAT OF TEST VARIABLES

The assessment of the optimum mix composition for a given aggregate type, and binder type, has been condensed into analysing the three mixture variables of aggregate gradation, binder content and level of compaction, and varying each of these in three increments. This tri-variance on each parameter results in an overall format of 27 mix specimens, as illustrated in Figure 8.5, to be investigated through the design procedure. This allows nine different mix formulations to be assessed at three different levels of compaction, encompassing the range of variability which could be expected for this material on site.

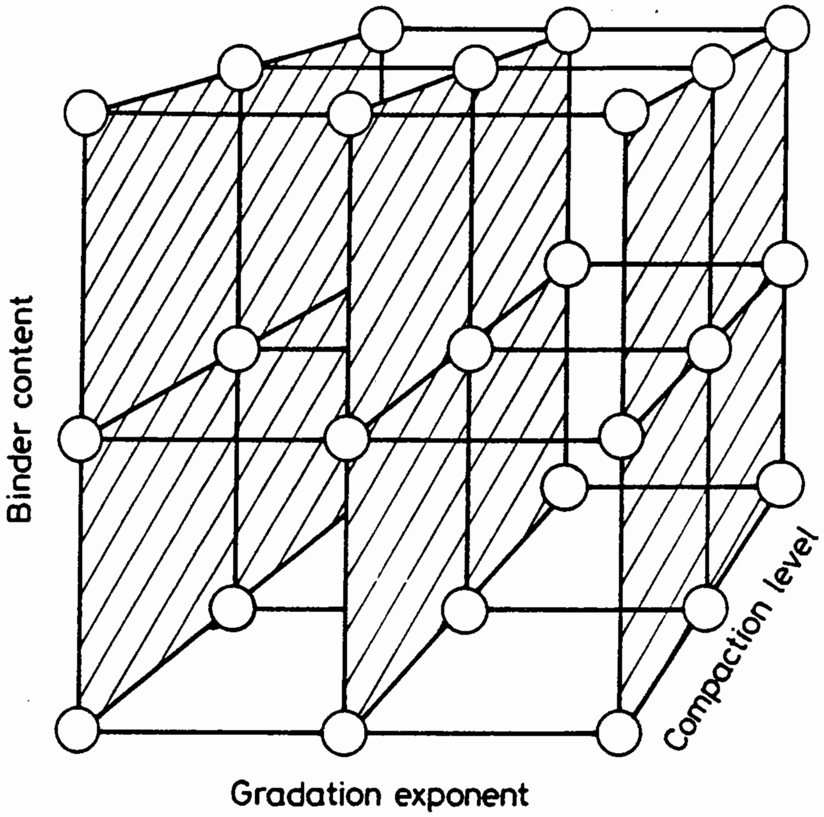


Figure 8.5 The structure of the 27 mixes to be evaluated in the mix design procedure.

8.4 VOLUMETRIC ANALYSIS

The first step in the analysis of the test specimens, is an assessment of the volumetric proportions resulting from the fabrication procedure. The volumetric composition of bituminous materials has a significant influence on the mechanical properties, allowing the volumetric proportions of the twenty seven specimens to be used as a preliminary assessment on the acceptability of the different mix formulations. By applying limiting criteria to the void content and VMA, thus controlling all the volumetric parameters of the mix, a target range concerning the volumetric compositions associated with optimal mechanical performance has been introduced.

The Asphalt Institute in their recommendations for the mix design of asphalt concrete apply rigid criteria to the void content of the Marshall specimens, stating that the percentage of air voids of the mix design specimens should lie between 3% and 5%. A minimum value is applied to the VMA according to the maximum stone size of the gradation, Figure 8.6, which in the case of 28mm aggregate would be 13%.

These criteria result in a very tight specification which although it may be applicable to Marshall specimens, provides unrealistic values for material in practice. Indeed, it has been stated that "some asphalt concrete mixtures are placed with air void contents over 8% and sometimes even over 10%" (98) after the material has been designed according to the Asphalt Institute criteria.

Therefore, the need to have some knowledge of the bituminous mixture's volumetric composition is apparent but careful attention must be paid to applying criteria to the volumetric parameters in the context of a mix design procedure. This is particularly important when considering that a comprehensive design method must address the variable characteristics of different rock types, as discussed in Chapter 3, where packing characteristics of different aggregates will influence the void contents and VMAs of bituminous mixtures.

8.4.1 Measurement of volumetric proportions

The traditional method of obtaining the relative density of a specimen of bituminous material is through gravimetric means, whereby the mass of the dry specimen in air is obtained and compared with the mass of the same specimen when submerged in a water bath. A knowledge of the aggregate and binder specific gravities allows the theoretical maximum specific gravity for any mix formulation to be calculated, facilitating the subsequent calculation of voids, binder volume and VMA of the specimen. It is usual practice to seal the surface of the specimen with an impervious membrane prior to immersing the sample in a water bath to prevent any ingress of water into the specimen, which could falsify the density calculation. Molten wax is typically used for this purpose, but can be difficult to remove from the specimen after the weighing operation. This aspect must be given consideration as clean specimen surfaces are required in order to perform mechanical tests. On cut, or sawn surfaces, an application of talcum powder prior to sealing the specimen, usually ensures that complete removal of the wax can be achieved but on the surface of moulded samples the irregularities may prevent satisfactory cleaning to take place.

With dense materials, typical of those under consideration in this project, it is questionable whether it is necessary to provide a sealed surface, as the internal void structure of the mix is relatively small. A visible examination during the weighing operation can establish whether any air escapes from the specimen during submersion and could be used as a guideline as to whether the samples are sealed or not. It is probable that a particular void structure will commence to influence the measured mass of the specimen in water, as voids integrate forming a connected system allowing the water to permeate through the specimen. Figure 8.7 shows the relationship between the calculated void contents of a series of specimens obtained without a sealant, and the measured void contents of the same specimens with sealed surfaces. The sealant used in this example was a self adhesive film.

Allowing for experimental scatter in the results there appears to be reasonable correlation between the sets of data up to a void content of around 6.5%, beyond which, ingress of water in the unsealed specimens appears to artificially depress the calculated values. A well-graded, well-compacted roadbase material should have a void content less than 8%, from which it may be concluded that DBM type mixes would not be subject to significant change in measured void contents through sealing the material surfaces. From closer scrutiny of the data, a more prudent value of where the two sets of results cease to correlate would be 5%, indicating that some DBM mixes may exhibit false volumetric proportions if densities are obtained from specimens with untreated surfaces.

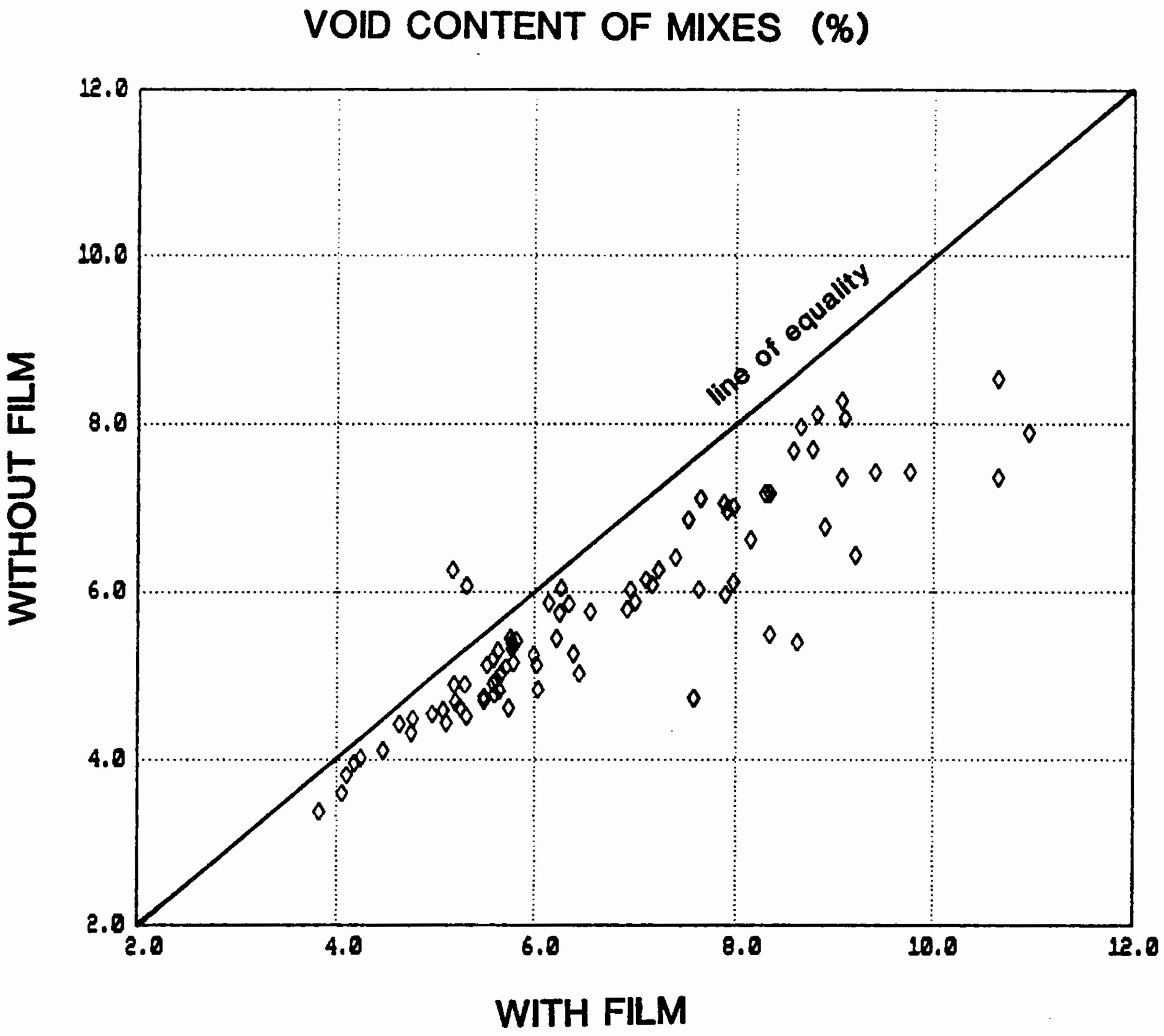


Figure 8.7 The effect of using a film sealant on the calculated void contents of test specimens.

8.4.1.1 PRD specimens

The specimens resulting from the PRD method of manufacture at compaction levels 2 and 3 exhibit considerable surface irregularities because of the confining effect at the interface between mould and material. This introduces a further complication when determining the volumetric composition of the specimen, as the circumferential region of the sample incorporates a void structure which is different from the internal voids. Hence, if the specimen is sealed then the calculated void content may be artificially inflated because of the surface voids. This phenomenon would be particularly apparent if a film or adhesive type sealant were used as this would have the effect of being tangential to surface voids, in other words the sealant would not mould itself to accommodate the surface irregularities. Therefore the elevated void content could be significantly different from the representative internal void structure of the specimen.

Weighing an unsealed specimen overcomes this effect as the water fills all the surface voids but the problem of ingression of water into the internal void structure re-manifests itself. If test specimens are cored and trimmed, as in Figure 8.4a, then a sealing agent for the measurement of density is recommended. However, it has been advocated that the coring procedure be omitted from the preparation of test specimens, requiring density measurements to be made on samples with surface irregularities and non-representative circumferential void structures. Given this situation, it is suggested that the material surface is not treated with any impermeable agent, and that measurements of the submerged mass of the specimen are taken with the sample in an unsealed state.

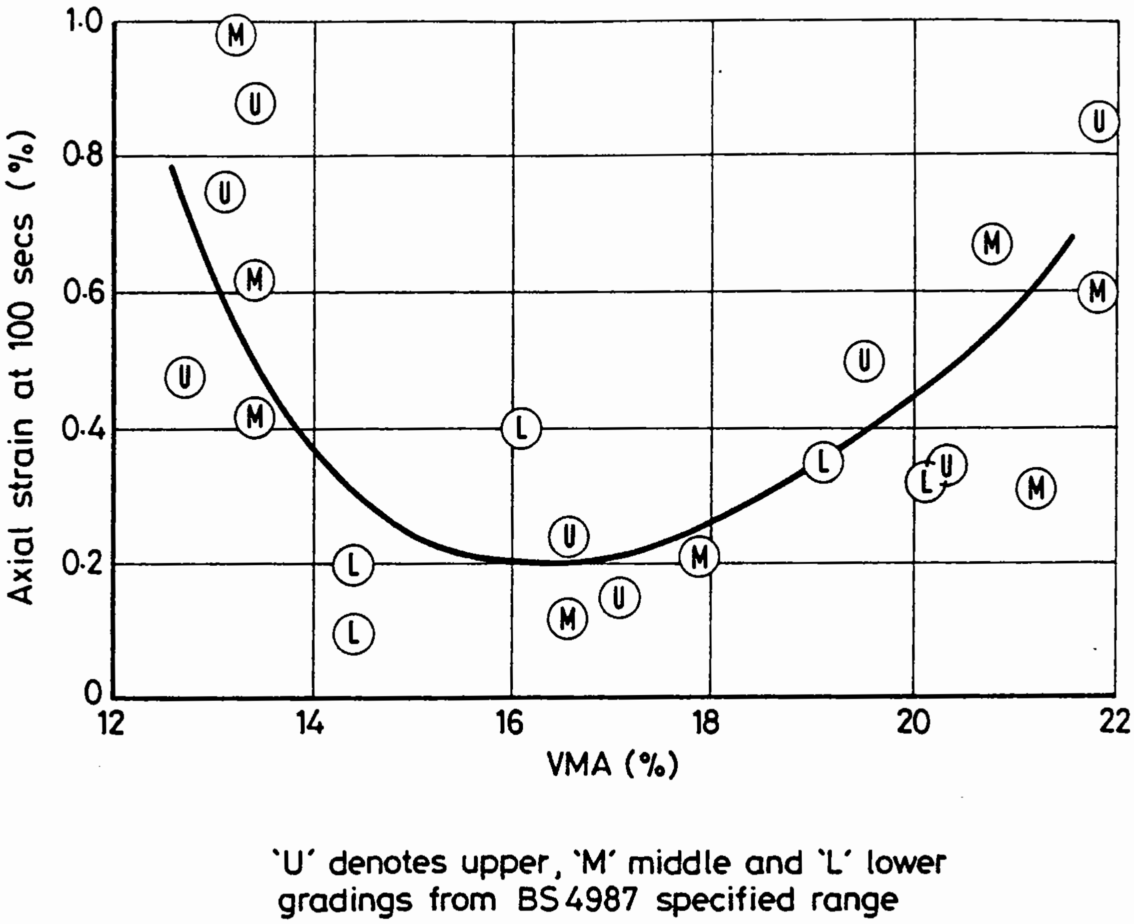


Figure 8.8 The influence of VMA on the resistance to permanent deformation for 20mm dense macadams. (After Cooper et al ref 35)

8.4.1.2 Maximum specific gravity

The volumetric compositions of the bituminous specimens are calculated from knowledge of the measured density of the samples and the maximum specific gravity achievable for such a mix formulation. The value of maximum density is obtained theoretically from the aggregate and binder specific gravities by using equation 8.3:

$$\text{Maximum density} = \frac{100}{\left(\frac{M_A}{G_a} + \frac{M_B}{G_b}\right)} \qquad 8.4$$

The accuracy of this equation in producing a true value of the maximum density, is completely dependent upon the accuracies of the values allocated to the aggregate and binder specific gravities. An average value of the effective specific gravity of the different aggregate size fractions is used for calculation purposes, which represents a good approximation to the overall relative density of all the aggregate used in the mix, but does not necessarily meet the exact value of the overall gradation. Therefore compound errors may occur through the weighing operation and reference to an incorrect value of maximum density. Such errors can be minimised by measuring the absolute specific gravity of the paving mixture itself instead of relying on the values obtained for the aggregate and binder. The 'Rice Method' (99) outlines a procedure whereby a specimen of a given mix formulation is re-heated and broken down by use of a laboratory spatula. The resulting particles are submersed in a sealable vessel such as a gas jar, and a partial vacuum is applied to eliminate any entrapped air. The mass of the particles, water and jar is then obtained, and compared with the mass of the dry particles. Allowance for the weight of the jar and water facilitates the calculation of the specific gravity of the broken down specimen. This value represents the maximum specific gravity of the mix formulation, and should be used for the subsequent calculations of the volumetric composition of the mix. This method is commonplace in the United States, but is not part of UK practice, where maximum specific gravities are calculated from typical values of aggregate densities.

Addressing the aspect of practicalities, introducing the Rice method would require duplicate specimens to be manufactured at each mix formulation, increasing the total number of specimens fabricated to thirty-six. It would be possible to compromise this value by only manufacturing an extra three specimens corresponding to the intermediate grading of n=0.6 at the three binder contents. This approach would make the assumption that a small change in the aggregate gradation would not effect the maximum specific gravity of a paving mixture if the binder content were to remain constant. The duplicate specimens should be fabricated at the intermediate level of compaction.

8.4.2 Assessment of volumetric proportions

The volumetric composition of bituminous materials has a fundamental influence on the mechanical performance, and can be used therefore as a preliminary appraisal of the potential of a series of different mix formulations. The importance of volumetric proportions has long been recognized as fundamental information of paving materials, with extreme values of voids being synonymous with inferior mixtures. Figure 8.6 shows the criteria applied to the VMA, and the constraints on percentage of air voids stated by the Asphalt Institute in their recommendations for Marshall mix design. These criteria do not consider variations in aggregate characteristics and constitute a very high specification which some roadbase and basecourse materials may be unable to meet. A general set of criteria must be associated with mixes which are likely to exhibit satisfactory mechanical properties in order that an analysis of the volumetric proportions can be used to 'screen' and therefore eliminate unsatisfactory mix formulations at an early stage of the design procedure.

8.4.2.1 Voids in the mix

The voids requirement of the roadbase mixture should be linked with compaction requirements. A satisfactory state of compaction is 95PRD, which corresponds to a void content of 5% greater than that at refusal. This value must associate with a minimum void content to ensure against over-compaction in the field, which should be taken at approximately 3%. All mixtures should exhibit some voids at 100PRD in order to maintain structural integrity, which results in a void content regime of 2%-3% minimum voids, to a value corresponding to the void content at 95PRD. Adopting 3% as the minimum acceptable void content, the upper limit becomes 3% + 5% = 8%.

8.4.2.2. Volume of binder

The binder volume is one of the principal parameters which influences the fatigue strength of the mix. An increasing volume of binder in the aggregate matrix contributes to the fatigue strength, enhancing the resistance of the material to cracking. Therefore a minimum value of V_B

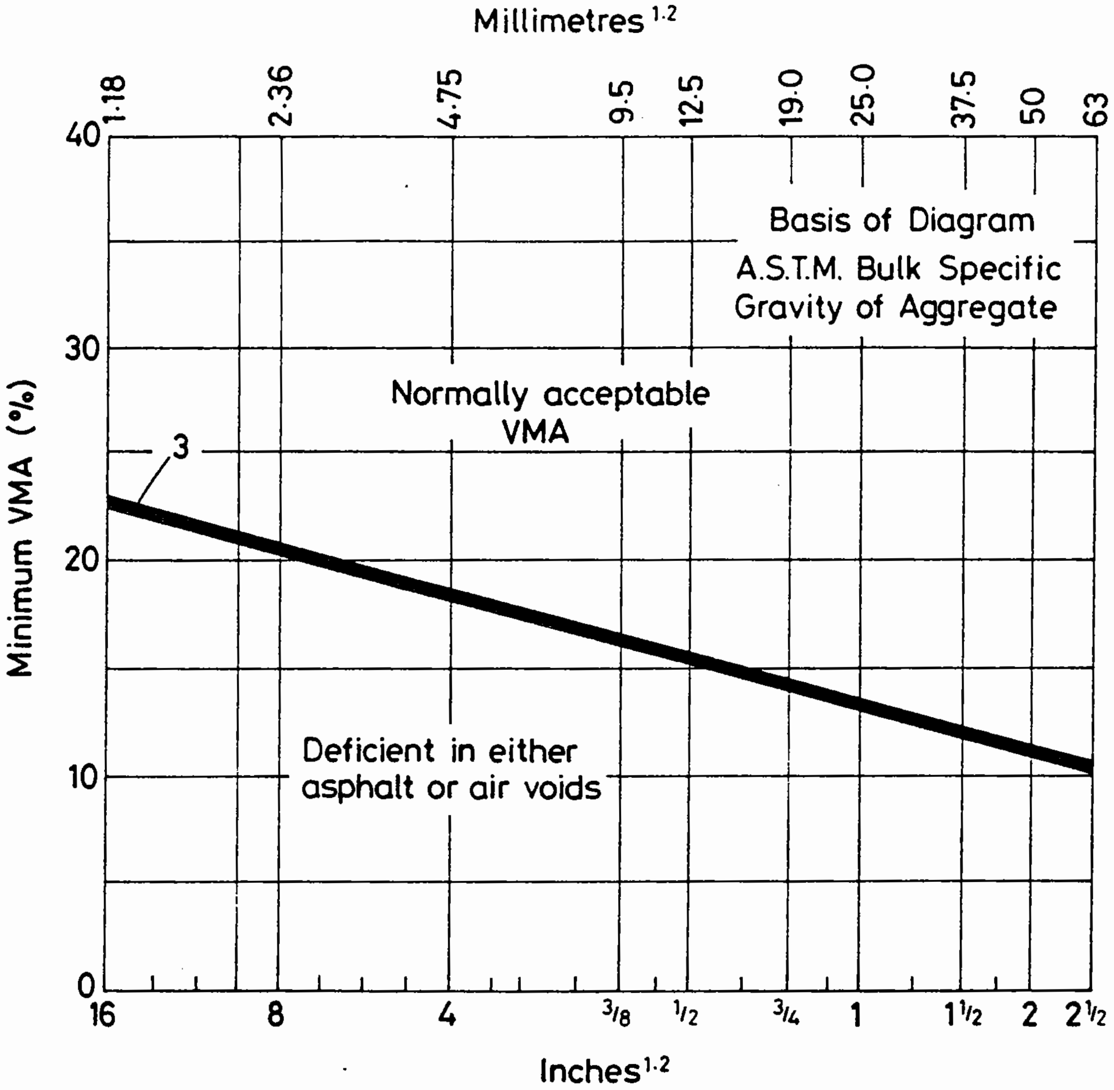


Figure 8.6 The criterion for minimum voids in mineral aggregate (VMA) (From the Asphalt Institute MS-2 ref 5)

must be recommended, in order to avoid producing a mix with inadequate cracking resistance. A value of 8% is suggested, which will tolerate binder contents of 3.5% by mass but will eliminate any leaner formulations.

8.4.2.3 Voids in Mineral Aggregate

Voids in the mixed aggregate (VMA) represent the state of the dry aggregate matrix, which must have sufficient voids to allow partial filling by the binder without the mix becoming deficient in air voids. Upper and lower bound criteria are required for this parameter, values of which should be related to the mechanical properties of the bituminous mixtures. From work performed on the correlation between VMA and mechanical properties (35), the lower bound criterion should be 13%, which coincides with the Asphalt Institute recommendation and the upper bound value 18%.

Although the range is somewhat empirical, experimental data on the effect of VMA on the deformation resistance of dense bituminous macadams, Figure 8.8 shows that preferential performance occurs over the values suggested. This is particularly emphasized for the tests carried out on the coarse gradation, which is the most typical of the gradings considered in the design procedure.

By controlling three of the volumetric proportions, the other parameter of voids filled with bitumen (VFB) is also controlled. This value represents the percentage of the VMA occupied by bitumen, and should provide an indication as to whether the mix formulation may be susceptible to durability problems. For the range of binder contents incorporated within the design procedure and the criteria applied to V_V and VMA, the VFB parameter will vary between 50% and 80%. It is likely that both extreme values would result in mixes which are either too lean and would suffer durability or fatigue problems, or too rich which may be susceptible to deformation. Although such mixes may not be eliminated from an assessment of the volumetric proportions, their inferior mechanical properties would be identified at the test stage, where rejection of such mix formulations would take place.

8.4.3 The effect of different aggregate types

The criteria applied to the volumetric proportions of the mix formulations can be summarized as follows:

Vv : 3% to 8%
VMA : 13% to 18%
VB : 8% minimum

These values form a range of acceptance criteria which are considered to encompass the spectrum of volumetric proportions, associated with mix formulations possessing satisfactory mechanical properties. The volumetric composition of a bituminous mixture is not solely dependent upon the aggregate grading, binder content and compactive effort applied to the mix, but also the packing characteristics of the aggregate particles. This idea can be described by considering two mixes comprising the same gradation and binder content, with one using a rounded aggregate, and the other a flaky aggregate. Given that both mixes receive the same compactive effort, it is quite conceivable that the resulting mixtures have different volumetric compositions. It is important that these differences are quantified so relative assessments of mix formulations containing different aggregates can be made. The concept of aggregate packing characteristics and their influence in bituminous mixtures, was introduced in Chapter 3 which documents an investigation into the design of gradations using a series of dry aggregate compaction tests. Although this work did not yield favourable results, the ideology of the approach may be transposed to address the problem of selecting appropriate volumetric criteria for bituminous mixtures of different aggregate types.

A datum must be established to act as a reference point for different aggregate types. This has been chosen as the void content of a mix compacted to refusal with a gradation corresponding to the n=0.5 curve and a binder content of 3.5%. This datum has been selected because n = 0.5 represents the densest grading under consideration, and 3.5% is the leanest binder content. Therefore the matrix of material at refusal density is representative of the aggregate type and characteristics. Any increase in binder content will start to fill the voids, and a change in grading exponent will open the matrix increasing the VMA.

Hence, different aggregate types will have different volumetric proportions when compacted to the datum level. The variations in void content may range from values approaching 0% to those in excess of 4% depending on the packing characteristics of the aggregate. This could lead to the situation where Mix type A has a void content of 6% at 95PRD and Mix type B has 9% for the same relative state of compaction, with both mixes having potentially similar mechanical properties. This emphasizes the importance of taking account of the aggregate properties, as well as assessing the performance of the final mixture.

The parameters described previously, from the Asphalt Institute method prove inadequate to deal with the variations which can occur in acceptable mixes. In order to accommodate these variations and facilitate an analysis of rock types from different sources, a series of sub-divisions must be introduced to the overall criteria.

The sub-divisions are governed by the void contents corresponding to the $n = 0.5$, $M_B = 3.5\%$, 100PRD mixes and are shown in Figures 8.9 to 8.11. Each Figure illustrates the overall criteria, described by dotted lines on each chart, and the sub-divisions for each category of voids are described by a bold outline.

Figure 8.9 considers the range of acceptable volumetric proportions for mixes which compact to 1% voids at the datum level. The criteria depicting the range of acceptability reduce the voids requirement to between 3% and 6%, and the VMA requirement to between 13% and 16%. The upper bound value on voids coincides with the 95PRD level of compaction for the datum mix formulation which ensures satisfactory compaction of acceptable specimens, while the lower bound value prevents over compacted specimens meeting the criteria. The reduction in the upper bound criterion of VMA from 18% to 16% has been made to form a compact target range for mix types corresponding to this category. Figures 8.10 and 8.11 show the expansions of the sub-divisions for 2% and

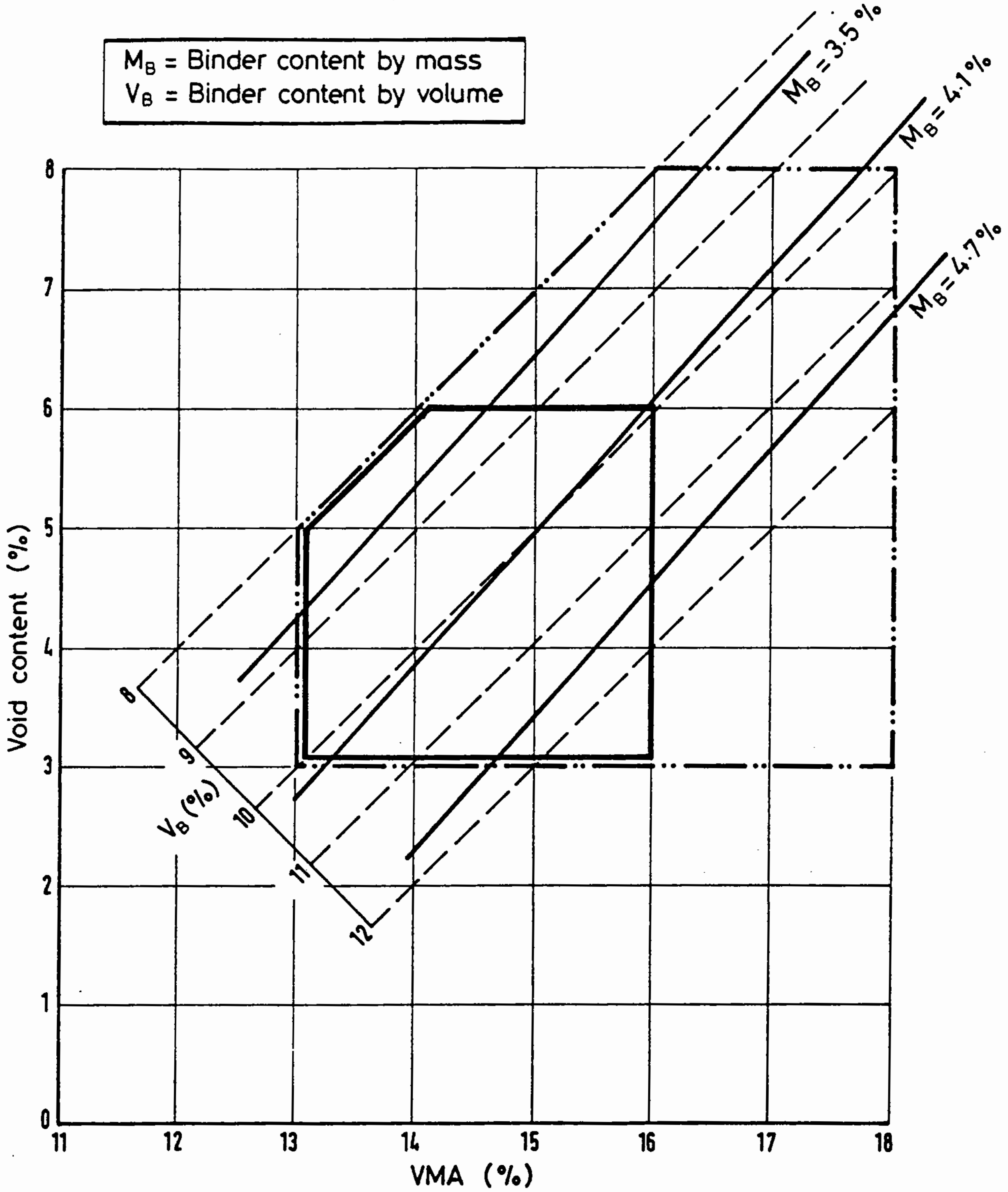


Figure 8.9 The recommended range of acceptable volumetric proportions for n=0.5, M_B=3.5% which compact to 1% voids at 100PRD.

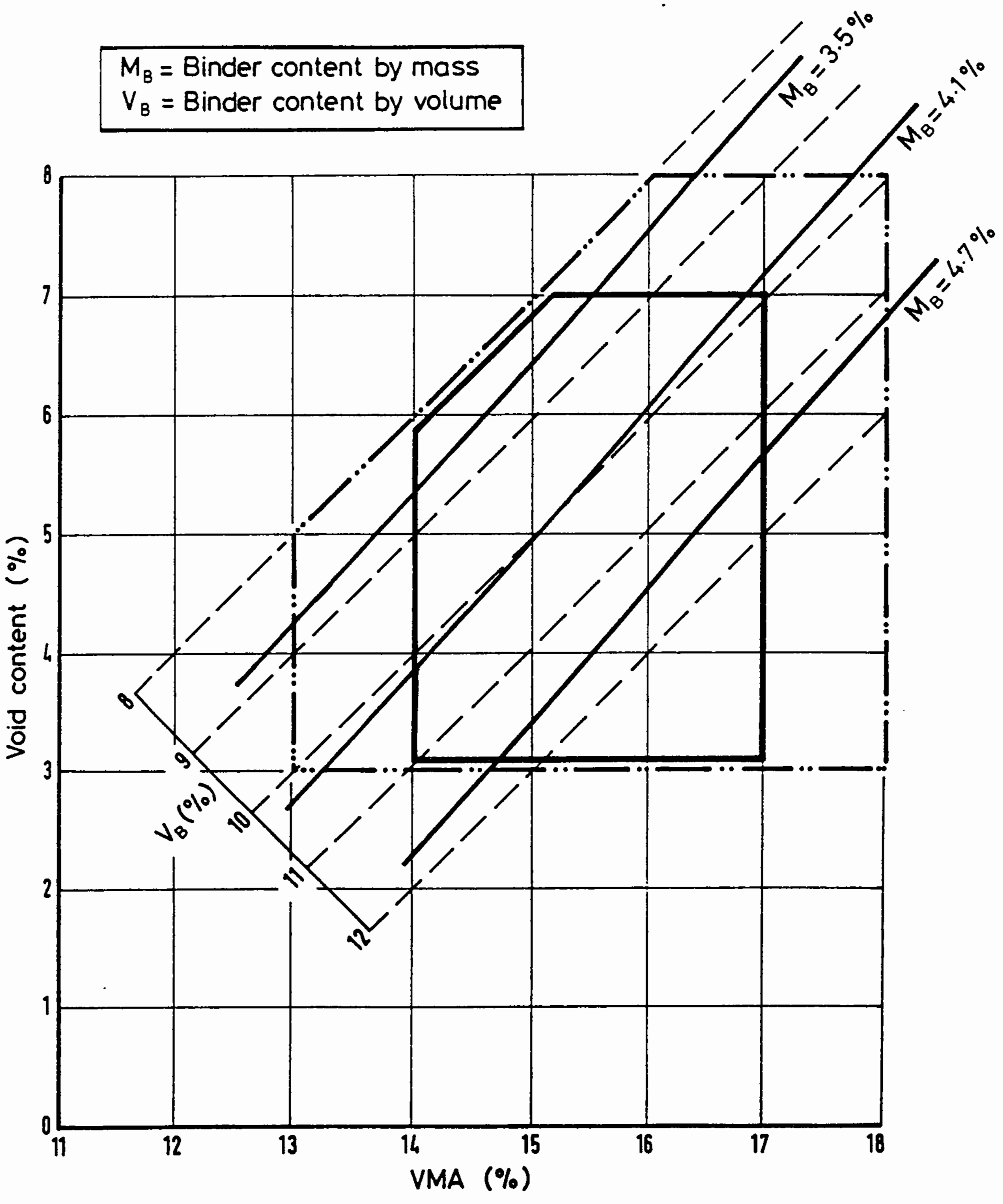


Figure 8.10 The recommended range of acceptable volumetric proportions for n=0.5, M_B=3.5% which compact to 2% voids at 100PRD.

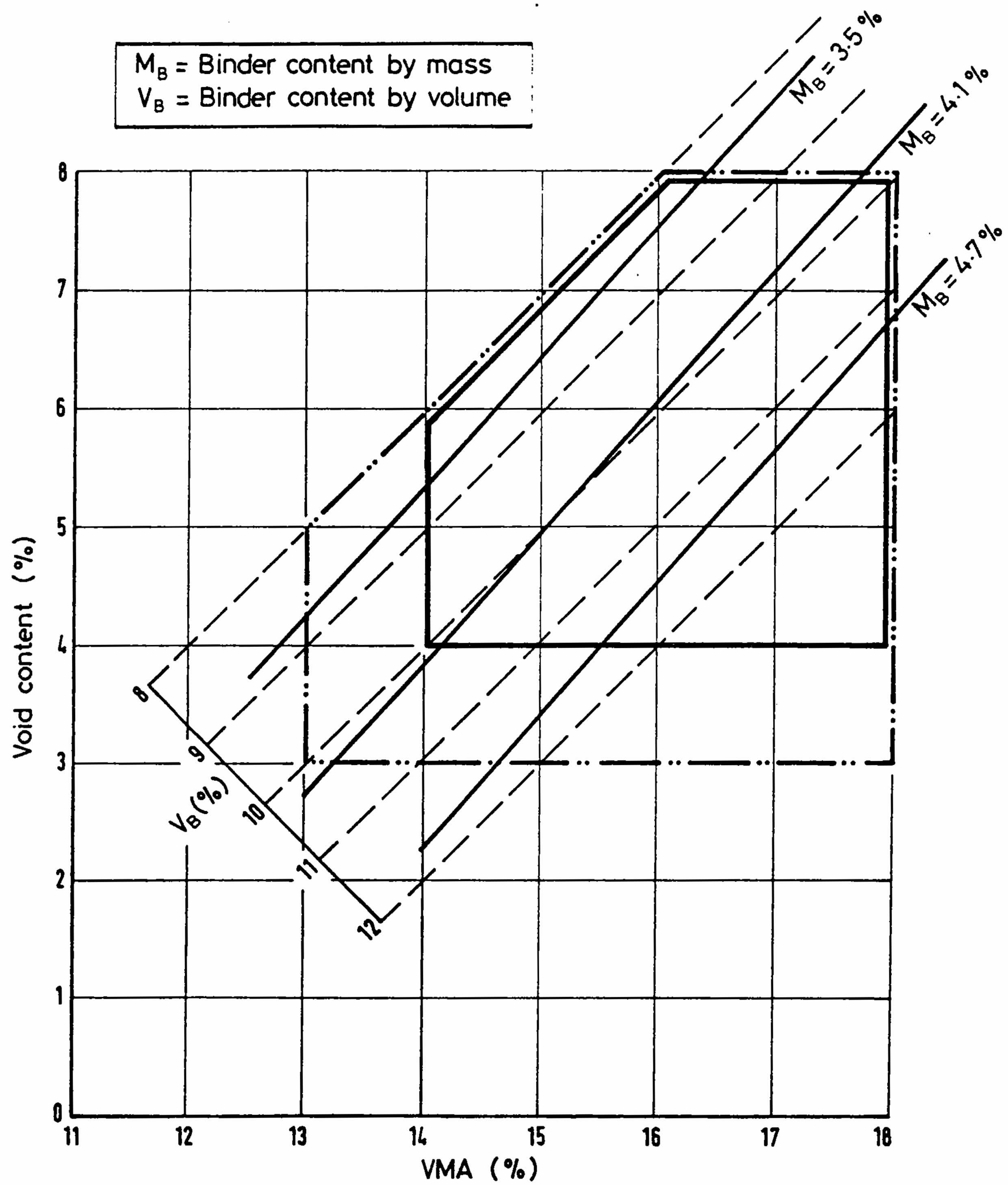


Figure 8.11 The recommended range of acceptable volumetric proportions for n=0.5 M_B=3.5% mix formulations which compact to 3% voids at 100PRD.

3% voids at the datum reference. In each case the lower bound value of the voids criteria is always coincident with the 95PRD level of compaction.

8.4.3.1. Adjusting the mix constituents

The aforementioned criteria for the volumetric proportions are based on an individual mix formulation which is considered to best represent the role of the aggregate and its influence on the volumetric composition of the mix. Any adjustments to the mix constituents will alter the volumetric proportions, but these must be considered relative to the datum formulation. By varying aggregate gradations and binder contents, an engineer can structure the volumetric composition as he desires, ranging from mastic type mixes to open graded friction course materials. Considering the mix variables in the design method, if a binder content of 4.1% is mixed with the $n = 0.5$ grading and compacted to 100PRD, the void content of the resultant specimen will be less than at the datum conditions, which is purely a factor associated with the binder content. Similarly, if the grading exponent is increased, this will open up the matrix increasing the void content of the specimen. Therefore, the densest gradation at the highest state of compaction should be taken to represent the potential of the aggregate which is given by the $n = 0.5$, $M_B = 3.5\%$, 100 PRD mix. Volumetric proportions calculated from this specimen provide fundamental information necessary to make judgements of the gradation and binder content required if the aggregate type is to be used in roadbase mixtures.

The Arabian mix design trials highlighted the importance of assessment of the volumetric proportions of mix formulations when compacted to a very high state. If a specimen exhibits zero voids at 100PRD, then it is likely that this particular mix formulation would be unsuitable for use in a heavy loaded pavement, especially if high ambient temperatures were anticipated. Modifications would be required to the grading, and or binder content to alleviate the problem.

The analysis of void contents associated with roadbase mixtures compacted to refusal density suggest that all PRD values should be quoted with accompanying details of volumetric proportions. Figure 8.12 illustrates the possible range of volumetric proportions which could result from the changes in gradation and binder content within the limits specified by the design procedure. The datum void content is indicated, which is 2% for the example shown. The possible void contents range from below zero to $3\frac{1}{2}$% . It is in the region where apparently 'negative' void contents manifest themselves, which can distort results in a PRD test. If a severe compactive effort is delivered to a rich mixture, then as the void content approaches a value of zero, the binder/fines mortar will begin to migrate out from the specimen, which is the phenomenon of a bleeding or flushing mixture. If this condition occurs in a PRD test, then the resulting specimen may be deficient in binder and fines which would lead to an incorrect measure of the mix density. All PRD values should be quoted with a corresponding void content and referred to some datum value as advocated in this report. Each aggregate type should be allocated a 'source datum' in terms of void content at the n=0.5, M_B = 3.5% 100 level, categorising the aggregate characteristics. Void contents of PRD specimens should then be assessed in terms of the source datum, allowing rational, engineering based analyses of site cores to be made.

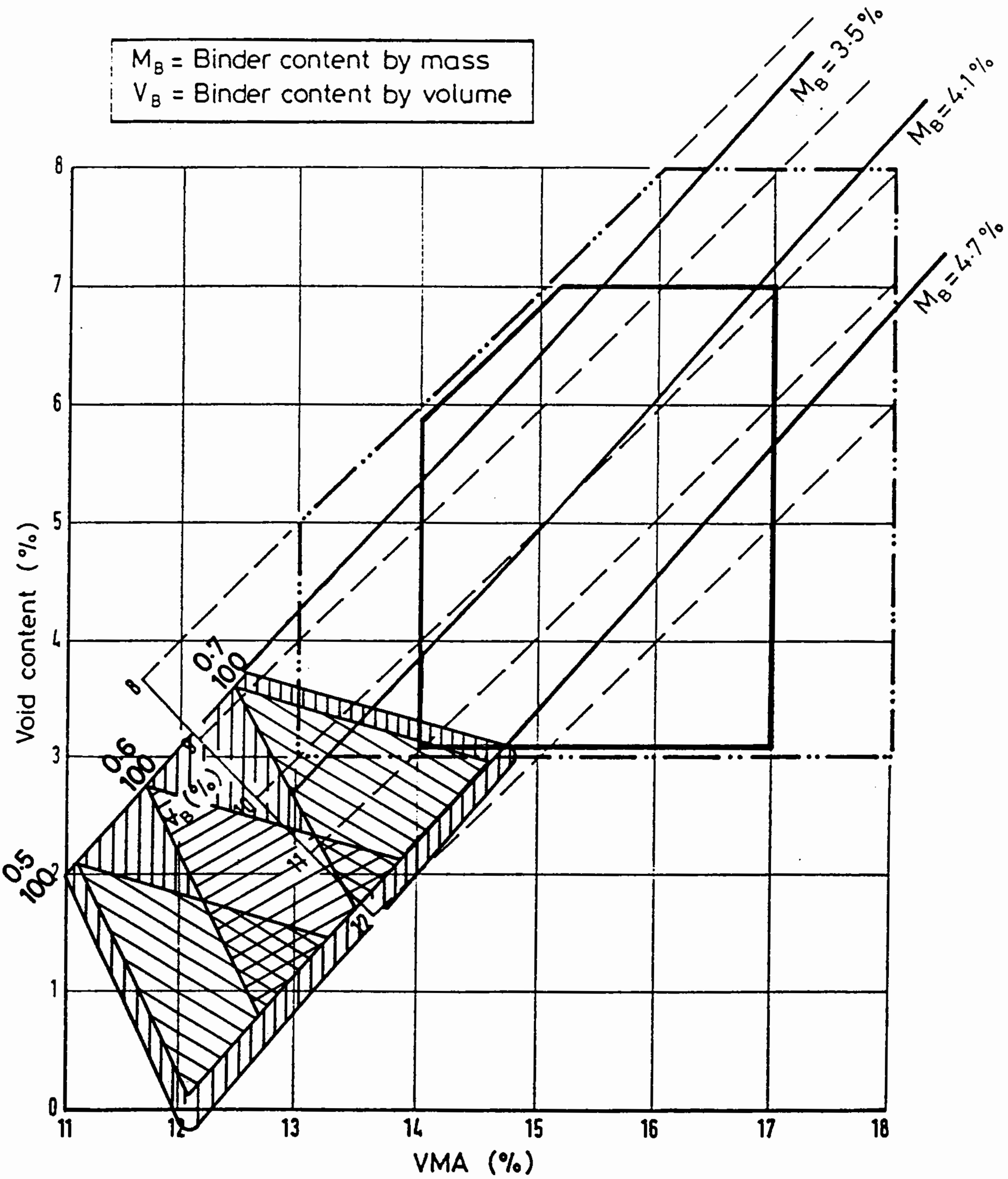


Figure 8.12 The possible range in volumetric proportions at the 100PRD level for mix formulations adhering to a recipe specification.

8.4.4. Summary comments

The preceding discussion indicates the importance of having some knowledge of the volumetric composition of bituminous materials in both mix design specimens and site cores. The problems associated with obtaining accurate values of the volumetric proportions suggest that it would be ill-advised to attach too much emphasis to these values, and that air voids and VMA's should be used as guidelines only for explaining or predicting mix performance.

In the mix design procedure the analysis of the volumetric proportions can be used as a first stage screening operation to eliminate mix formulations which do not adhere to the general range of volumetric compositions associated with good mechanical properties. In all cases, the 100 PRD specimens will be rejected according to the criteria, as these are over compacted, and are not representative of newly laid material. However, the volumetric proportions of these specimens, particularly the air void content, gives fundamental information concerning the suitability of the mix formulation for use in pavement construction.

Specimens which are under-compacted will also fail to meet the suggested specifications and will be rejected from the design procedure. It is the level 3 compaction specimens which may fall into this category, but the volumetric proportions can give an indication of the sensitivity to variations in compactive effort and attention must be given therefore to the volumetric composition of the same mix formulations at compaction level 2. It is conceivable that the conclusion arising from the assessment of volumetric proportions could change site compaction requirements according to the material which is to be laid. Although knowledge of the volumetric composition of different specimens provides necessary information for the assessment of any mix, it is emphasised that these values should be used as guidelines to performance, with final decisions being based on the results of mechanical tests.

8.5 MEASUREMENT OF MECHANICAL PROPERTIES

The mechanical properties of bituminous materials which may conveniently be quantified through laboratory testing in the context of a mix design procedure, are elastic stiffness and resistance to permanent deformation. At the time of these investigations, the resistance to fatigue cracking requires estimation through a prediction model in the absence of a rapid laboratory test method.

The elastic stiffness should be measured using the Repeated Load Indirect Tensile test, at a material temperature of 20°C and a load rise time of approximately 0.12 seconds. The deformation on the horizontal axis of the specimen is measured by two LVDTs, which are placed in contact with the material within their response range. Using specimens which have not been cored, that have a non-uniform surface texture on the circumference, with considerable surface voids, presents a problem for the point contact required for the LVDTs. An angled contact between void and LVDT may result in slippage of the spring loaded core, hence distortion of the calculated stiffness values. This effect can be eliminated with a minor modification to the LVDT of substituting the point contact core tip with a disc contact of approximately 8mm diameter. This facilitates a tangential contact between the specimen and LVDT preventing any possible movement of the LVDT core during testing. The modified arrangement is shown in Figure 8.13.

The preferred test for the assessment of the deformation resistance of bituminous materials is the Repeated Load Axial test, where a load corresponding to a stress of 100kPa is repeatedly applied to the specimen, alternating with rest periods. The format of the loading regime is approximately a square wave with the load applied for one second followed by a one second rest period. This type of deformation test is advocated in preference to the static creep test as it is able to distinguish differences in deformation resistance characteristics of nominally similar mix formulations. The NAT facilitates testing of 100mm and 150mm diameter specimens in both RLIT and RLA tests, but the choice of 150mm diameter specimens allows a new permutation in the configuration of the RLA test.

The standard test configuration for the analysis of resistance to permanent deformation is shown in Figure 6.5, where the loading platens conform to the specimen diameter of 100mm or 150mm. If the 100mm diameter loading platens are used in the RLA test with 150mm diameter specimens, as shown in Figure 8.14, then the test becomes non-fundamental, ie., the stress regime cannot be quantified but is more representative of the loading conditions in the field. This configuration introduces an annular ring of material around the loaded area which provides a degree of confinement to the material subject to direct axial stress. The level of confinement is directly proportional to the state of compaction of the material, which is analagous to the situation in the road where material in vertical alignment of the wheel tracks is subject to an overall compressive stress and is restrained from lateral displacement by adjacent material. Although the fundamental stress regime has been lost, the test method should be able to rank mix formulations according to their deformation resistance characteristics.

The net effect of the annular ring of confining material is to reduce the overall deformation induced in the specimen from the repeated applications of load. Figure 8.15 illustrates the change in the shape of the deformation curve for specimens tested conventionally, (100mm diameter platens acting on a sample of diameter 100mm), compared with the modified, non-fundamental arrangement of 100mm diameter loading platens acting on a specimen of diameter 150mm.

This modified test configuration is suggested as the mode by which the resistance to deformation of specimens should be assessed, as it is considered to mobilize the material characteristics in a manner which approximates to field conditions and it eliminates the coring sequence from the design procedure.

8.5.1 Criteria of acceptability

This mix design procedure has aimed at manufacturing representative specimens of roadbase materials and measuring the salient mechanical properties in the context of field performance. This does not result in the selection of the mixture which ranks higher than others in end product properties but an identification of a series of mix formulations which satisfy criteria applied to the mechanical properties.

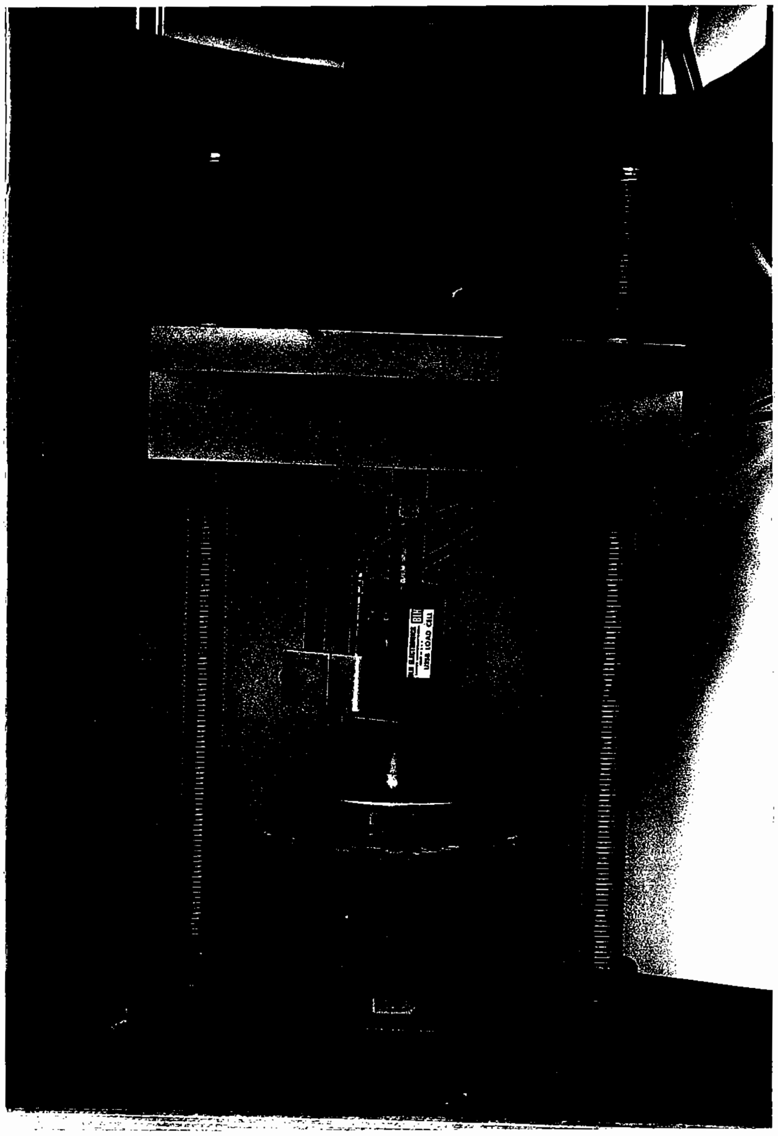


Figure 8.14 Modified Repeated Load Axial test. 100mm diameter loading platen acting centrally on 150mm diameter specimens.

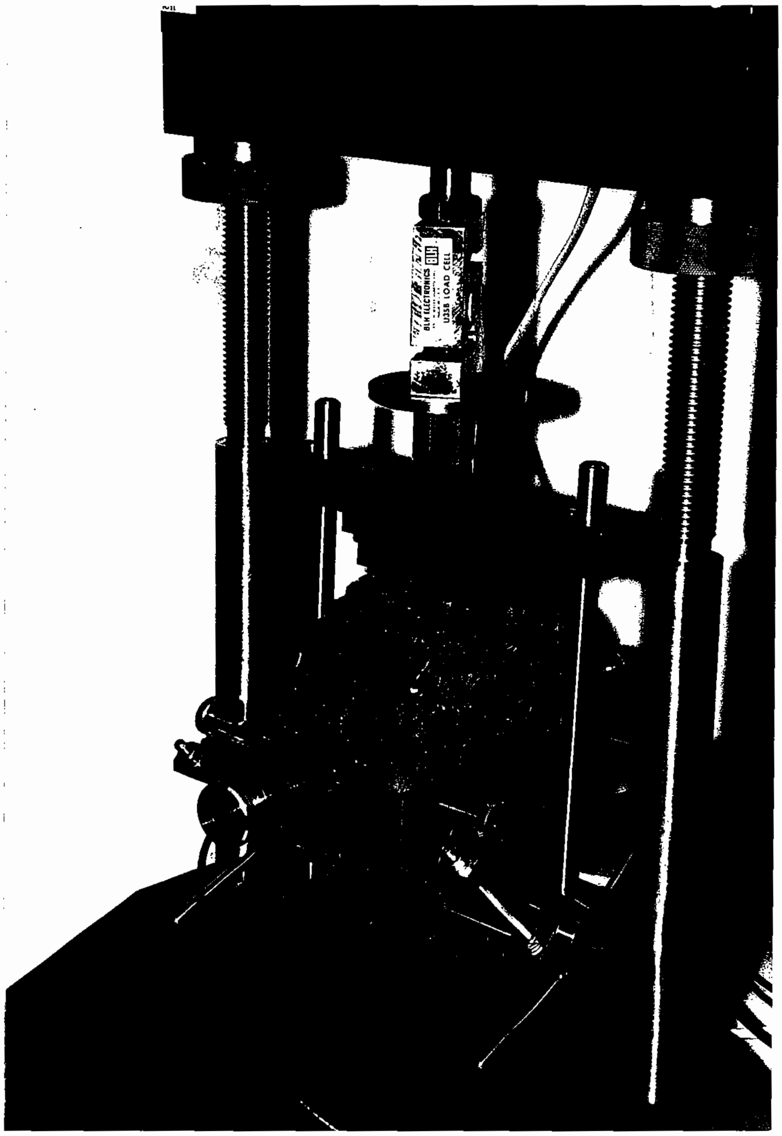


Figure 8.13 Disc contact on the LVDT for use in the RLIT test on specimens with moulded surfaces.

Fig 8.13.

8.14

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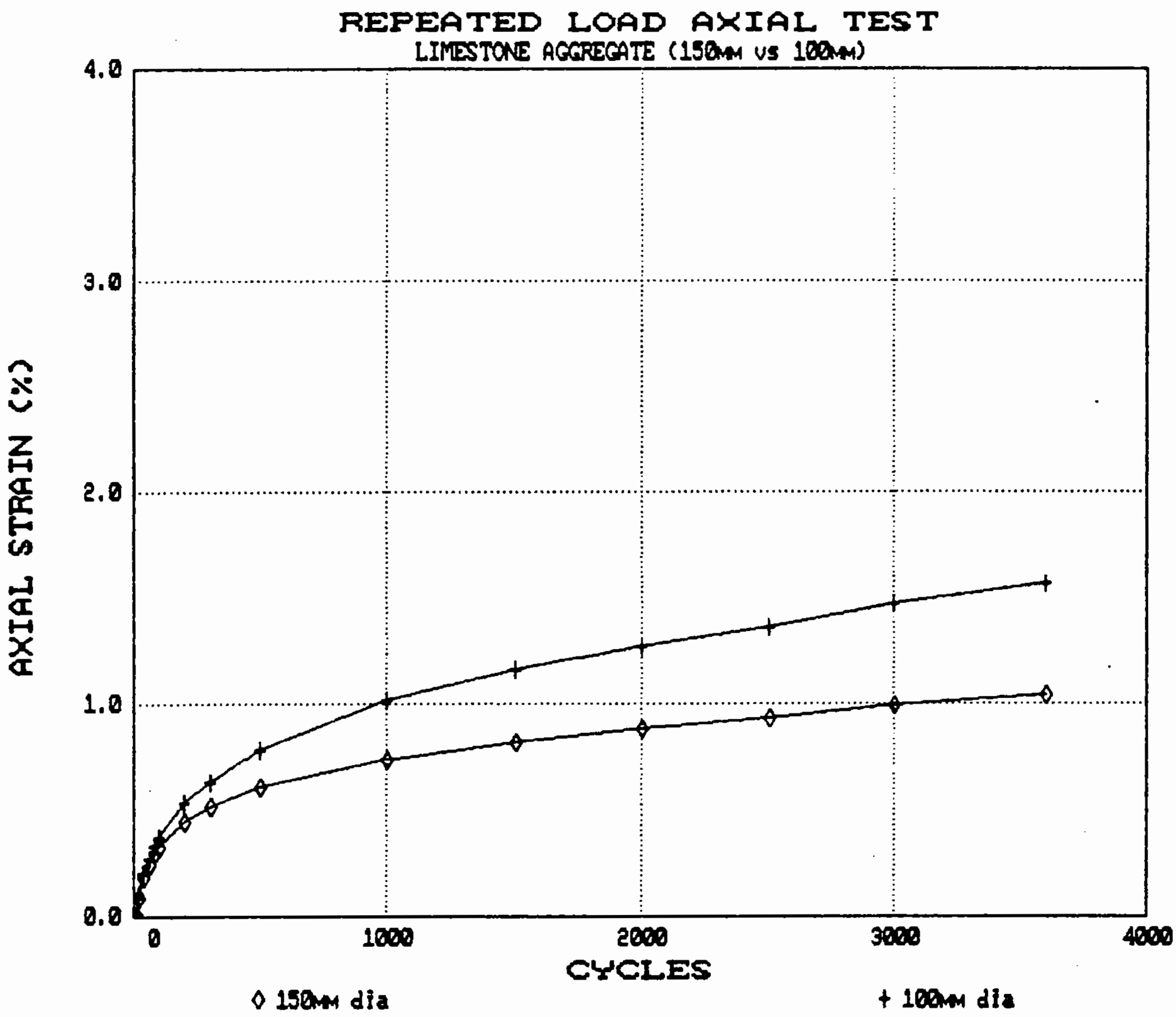

Figure 8.15 Example plot illustrating the possible effect of the use of 150mm diameter specimens in the RLA test.

The first stage of specimen analysis through an assessment of the volumetric proportions has been described, where a range of values of Vv and VMA associated with good compaction and satisfactory mechanical properties identifies the mix formulations which are most likely to exhibit optimum performance from the full selection of mixtures originally manufactured. The second stage is the evaluation of mechanical properties, which facilitates a ranking of the mix formulations in terms of performance. The application of acceptance criteria to the results of the tests.can be used to establish which mix formulations are satisfactory and which should be eliminated from consideration.

8.5.1.1 Elastic stiffness

The criterion applied to the elastic stiffness parameter must relate to the test conditions, as temperature and rate of loading have significant influences on the elastic response of the material and to whether the response can be considered to be elastic. An environment of 20°C coupled with a load rise time of 0.12 seconds may be considered to give a predominantly elastic response from a bituminous specimen when loaded under such conditions.

An indication of typical stiffness values can be obtained through prediction methods which are based on the volumetric proportions of the mix and the stiffness of the binder. Figures 8.16 and 8.17 show how the predicted mix stiffness changes according to variations in the mix composition and resultant volumetric proportions. Each plot is based on an original binder penetration of 100. Taking a mid-range value of the parameters involved in the mix design procedure (an aggregate specific

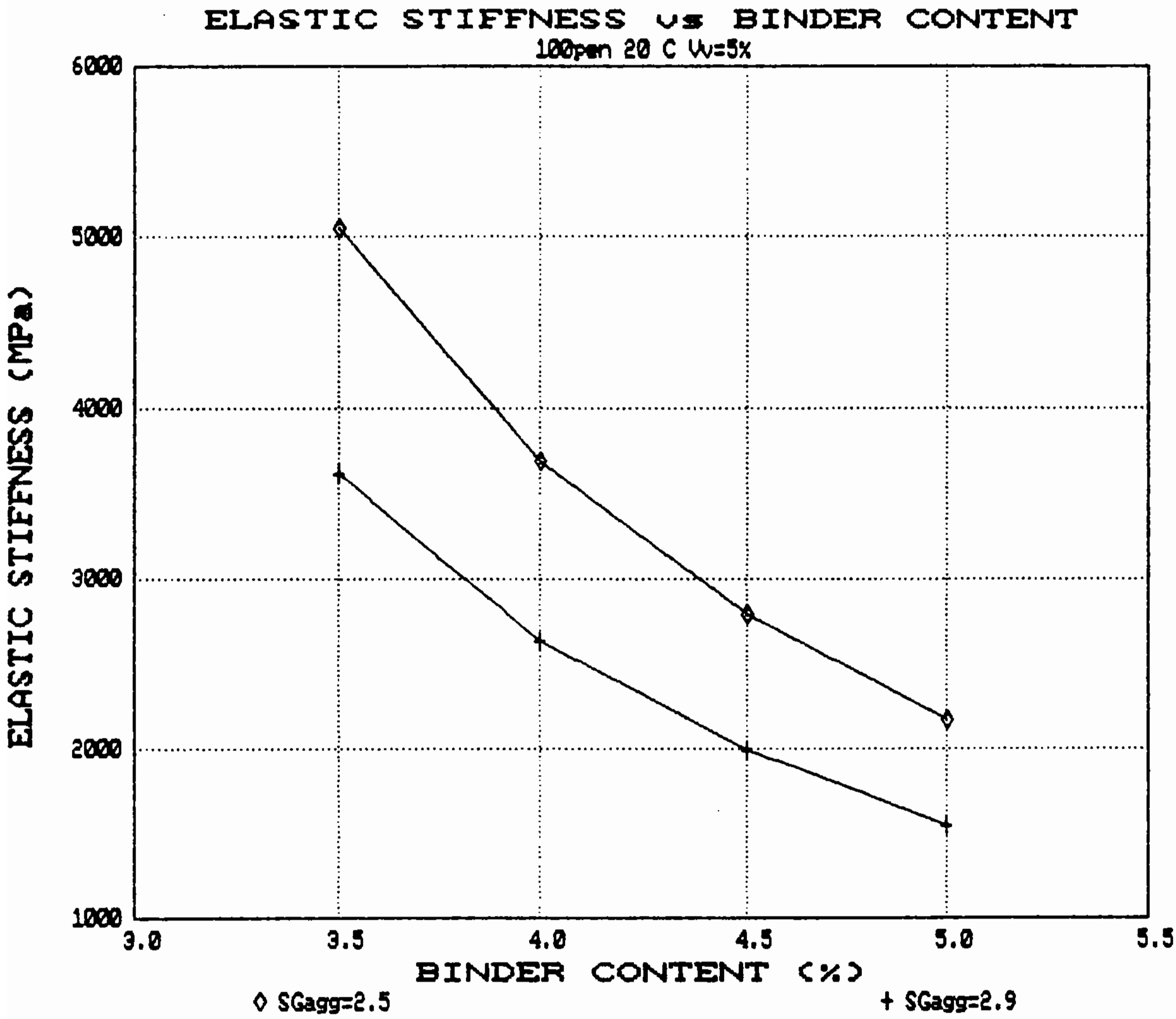

Figure 8.16 Predicted mix stiffness vs binder content.

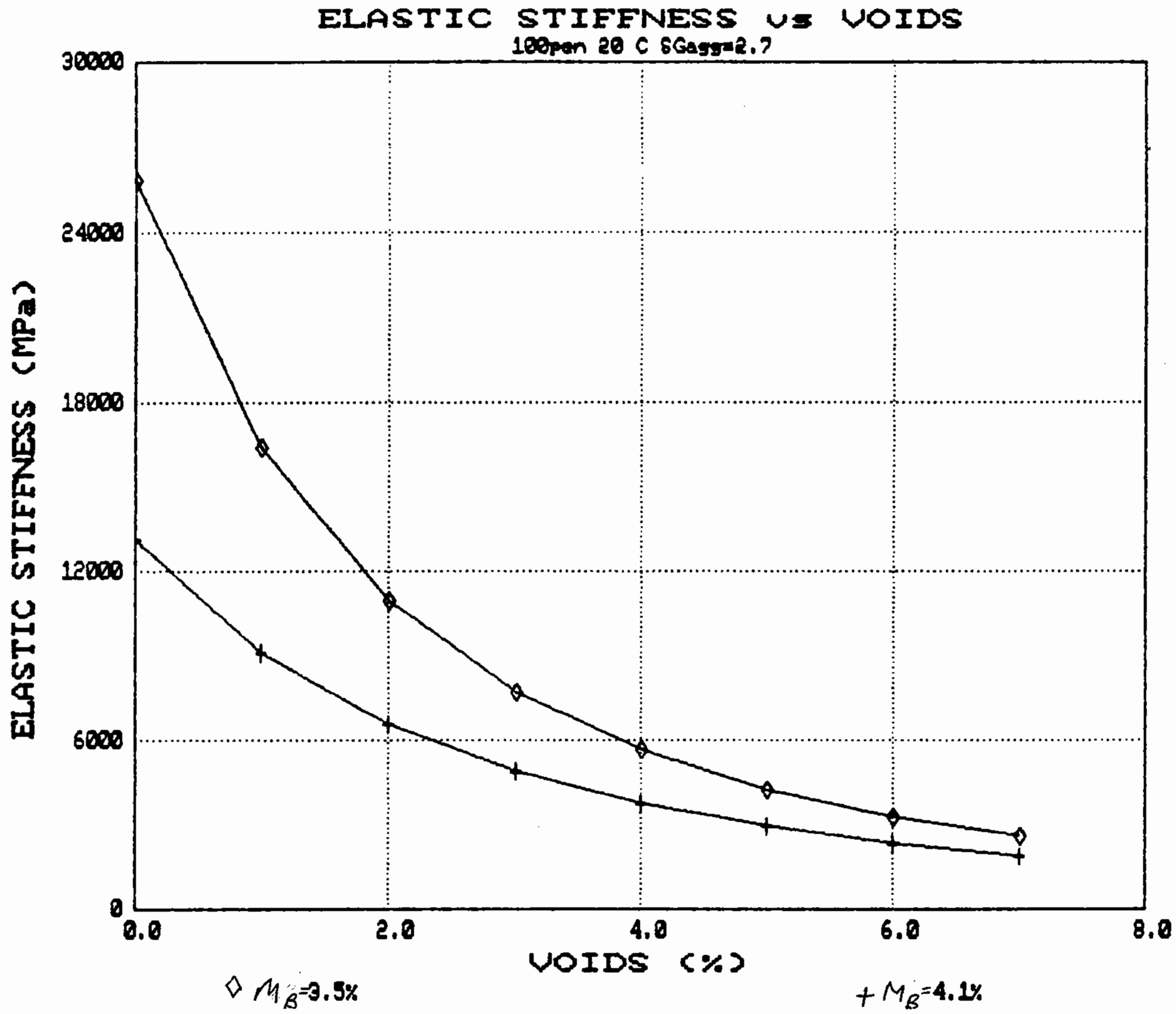

Figure 8.17 Predicted mix stiffness vs void content.

gravity of 2.7, binder content of 4.1% and a void content of 6%) then a stiffness of around 2500MPa is predicted. This value does not take into account the structure of the aggregate matrix, which must influence the stiffness value, or variations in temperature of the materials and loading time of the applied pulse. The latter variables will be encountered during the testing procedure of a number of specimens as absolute accuracy cannot be maintained throughout testing of this nature. The influence of loading time (typically 0.12 seconds in NAT testing) is shown in Table 8.3, where the range of loading time encompasses that which could be expected from the NAT.

Table 8.3 The effect of variations in Loading time
on the predicted mix stiffness.

Pen (original)	Binder (%)	Voids (%)	Temp (°c)	Loading time (seconds)	Sb (MPa)	Sm (MPa)
100	4.1.	6	20	0.110	6.677	2423
100	4.1	6	20	0.115	6.472	2376
100	4.1	6	20	0.120	6.282	2331
100	4.1	6	20	0.122	6.224	2318
100	4.1	6	20	0.125	6.104	2290
100	4.1	6	20	0.130	5.936	2250

It can be seen that within the limits of the range of loading times, the predicted mix stiffness varies by approximately 3.5% around a mid-value. This does not constitute a significant effect and would not be likely to influence the scatter of results which could normally be expected. Variations in material temperature have greater implications for mix stiffness as shown by the predicted values in Table 8.4.

Table 8.4 The effect of changes in temperature on the predicted mix stiffness.

Pen (original)	Binder (%)	Voids (%)	Temp (°c)	Loading time (seconds)	Sb (MPa)	Sm (MPa)
100	4.1.	6	18	0.122	8.850	2888
100	4.1	6	19	0.122	7.422	2589
100	4.1	6	20	0.122	6.224	2318
100	4.1	6	21	0.122	5.206	2070
100	4.1	6	22	0.122	4.354	1847

In this case the predicted mix stiffness varies 20% either side of the mid-range value indicating the importance of testing the specimens at a constant temperature. The predicted values of stiffness demonstrate that for the given conditions and volumetric composition, the expected values should lie in the range of 2000MPa to 2500MPa. In order to make better use of resources and maximise material performance a minimum stiffness of 3000MPa would be an appropriate target. Due to the nature of the RLIT test, a tolerance must be applied to this value in the order of around 500MPa making the minimum acceptable stiffness for a roadbase material of 2500MPa at 20°c and a loading time of approximately 0.122 seconds.

8.5.1.2 Resistance to permanent deformation

The deformation resistance of bituminous materials has proven to be the singularly most difficult parameter to predict, and although test methods such as uniaxial creep have been available to quantify deformation under a static load, the interpretation of results from such tests has been in terms of relative assessments between similar specimens. It is important that some value can be allocated to deformation tests to introduce a performance based specification with respect to field ruttability.

The modelling of the stress regimes encountered in a road pavement under moving traffic is almost impossible. The varied frequency and magnitude of loading, inconsistency of rest periods and changing material temperatures represent the 'real' situation which somehow must be quantified in the artificial environment of the laboratory. This project has formed an investigation into the development of a mix design procedure for roadbase materials, which in pavement construction are the underlying bituminous layers forming the main structural component of the highway. These layers do not receive direct loading from traffic but are subject to a dispersed stress regime via the wearing course. Reproduction of the contact stress between wheel and pavement would be inappropriate for assessment of roadbase performance and give an excessive stress application on a small laboratory prepared sample tested at an elevated temperature.

Criteria of acceptability for RLA testing should be taken from a library of data acquired from the testing of site material from existing pavements which are known to have a good performance history, utilising cores from the wheel tracks and material from non-loaded areas such as the lane centre. At present, no such library exists and such a mode of data acquisition would require extensive co-operation from highway authorities in a co-ordinated effort to address the problem. By maintaining the test format, the results of deformation characteristics from existing pavement material could be used to establish a datum of acceptability which could be modified to provide a criterion to be implemented at the mix design stage.

Two parameters require analysing when assessing the resistance to deformation in a controlled manner such as the RLA test:

i) The ultimate value of strain at the end of the test.

ii) The rate of strain during the test.

The first parameter is easy to quantify as it is represented by an individual numerical value, but an evaluation of the second parameter is more difficult. A typical deformation plot resulting from the RLA test (150mm diameter configuration) is shown in Figure 8.18, where the deformation curve can be generalised into three regions. The first region, or primary deformation, represents a combination of seating the loading platens, secondary compaction of the material and an elimination of the elastic response, resulting in rapid early deformation. Secondary deformation can to attributed to a restructuring phase of the material where the aggregate matrix is forced together until intimate contact between aggregate particles is achieved. During this phase the material can be described as 'locking up', and the properties of the aggregate in terms of resistance to permanent deformation are beginning to take a dominant role. The tertiary phase of the plot represents the viscous flow of binder within the aggregate matrix, which can facilitate further restructuring and is analagous to the creep phenomenon associated with plastic deformation of bituminous mixtures. All three phases must be addressed when considering the rate of strain during the testing period and quantified by some means. A possible method to achieve this would be through expressing the quotient of the difference in strain over the tertiary period and the level of strain at the end of the secondary phase. This could be expressed by an equation:

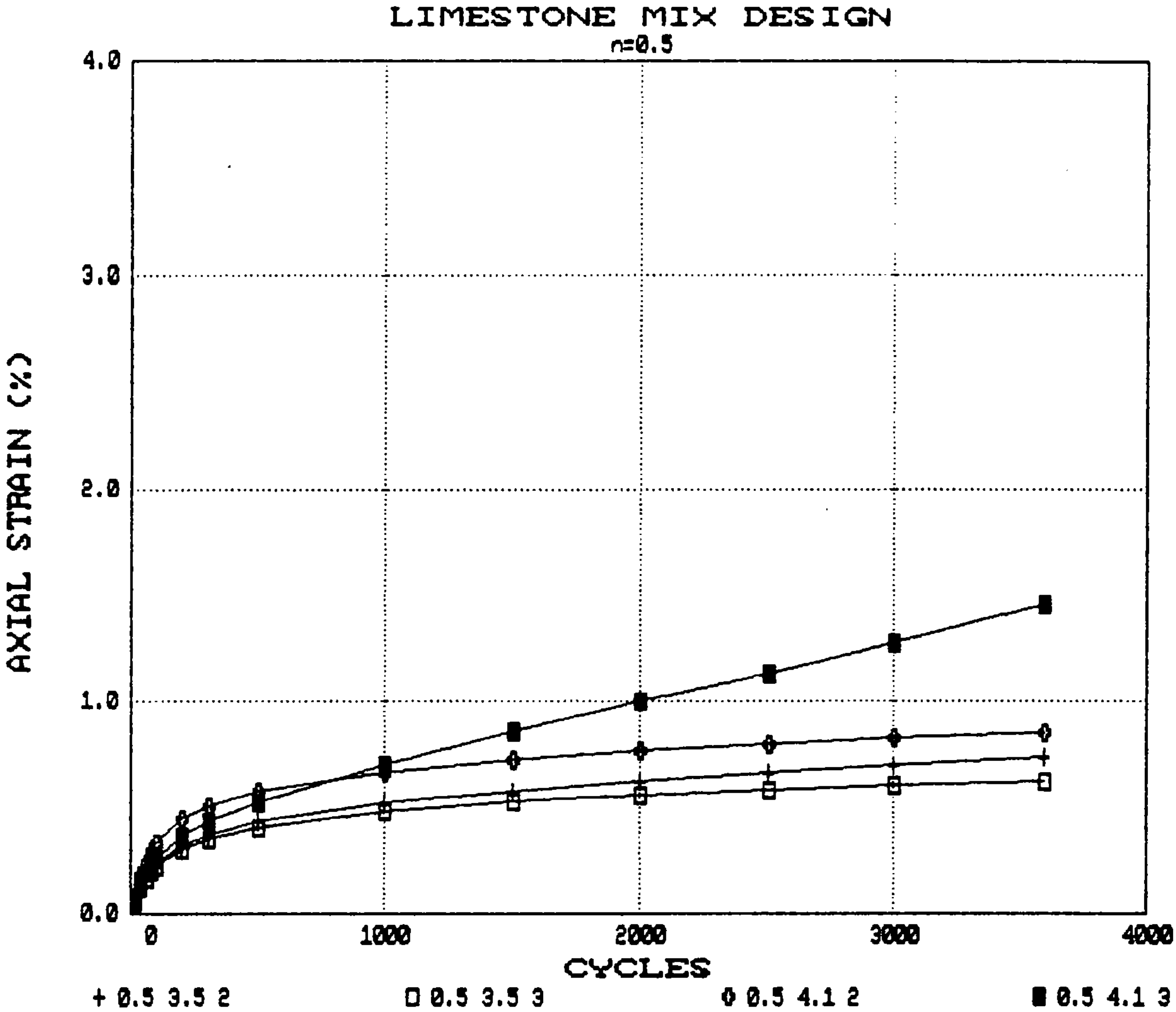

Figure 8.18 Typical results from the RLA test.

$$\text{strain rate index} = \left(\frac{E_{3600} - E_{1000}}{E_{1000}}\right) \qquad 8.5$$

where E_{3600} = level of strain in specimen after 3600 cycles

E_{1000} = level of strain in specimen after 1000 cycles

A limiting value of approximately 0.35 applied to the quotient would ensure that any specimens with an increasing rate of strain in the tertiary phase would be identified as deformation susceptible mixtures.

The analysis of the shapes of deformation curves has received little attention and further investigation is required in order to quantify this parameter in engineering terms. Visual examinations of the plots can easily determine which formulations begin to exhibit failure characteristics, but to remove the subjectivity of this analysis, a rational and numerical quantification must be introduced. This must take account of the strain rate in the tertiary phase and the amount of strain induced throughout the first and second phases. It is important to distinguish between mixtures which have similar ultimate strains at the termination of the test, but have different strain rates during the test.

An early draft of the design method (12) suggested a tentative recommendation that the maximum strain at the end of the RLA test for a satisfactory mix formulation should not exceed 1%. This value was applied to the 100mm diameter test configuration where it was found that specimens frequently failed to satisfy the criterion. The modified test utilising 150mm diameter specimens with 100mm loading platens (Figure 8.14) has displayed a greater adherence to the criterion over the range of mix formulations investigated. It must be stressed that the application of the 1% strain criterion to the RLA test is based on engineering judgement and this value requires corroboration through testing of existing material.

CHAPTER NINE
RESULTS

The following three chapters provide a selection of the salient results which have been obtained through the development of the mix design procedure. Results from the granite and limestone aggregates are included in this chapter, followed by a comparison of the mix formulations which result from the same constituents when designed by the Marshall method and the procedure which has been described herein. Finally, a concise investigation into the assessment of a durability index concerning materials with different aggregate types is described and evaluated in terms of a mix design procedure.

Work concentrated on the limestone aggregate, with comparisons of results obtained through the granite aggregate.

9.1 QUANTIFICATION OF AGGREGATE TYPE

The type of aggregate used in a bituminous mix formulation influences the volumetric composition resulting from the compaction of the material. The effect of this influence can be assessed in terms of the void content of the mix corresponding to the $n = 0.5$, $M_B = 3.5\%$ formulation when compacted to 100 PRD. The values obtained from the limestone and granite aggregates are displayed in Table 9.1.

Table 9.1 Volumetric proportions of 'datum' mix formulations using limestone and granite aggregates.

Aggregate type	Mix formulation	Density (g/ml)	V_v (%)	V_B (%)	VMA (%)	PRD (%)
Limestone	0.5, 3.5, 1	2.512	0.6	8.6	9.2	100
Granite	0.5, 3.5, 1	2.602	1.7	8.9	10.5	100

The difference in the void content for what is nominally the same mix formulation with two aggregate types can be seen to be in excess of 1%. This emphasises the need for specifications on volumetric proportions according to the packing characteristics of the aggregate. Using the charts described in Section 8.4, the void contents which were obtained from the two aggregates must be approximated to the nearest integer, and specifications selected from the appropriate chart. In the case of a mid-point value, such as 0.5%, 1.5% etc., easy interpolation of the volumetric proportions criteria could be made between the two relevant charts. For the aggregates used, the specifications for the limestone would correspond to Figure 8.9 (1% voids at 100 PRD), and specifications for granite to Figure 8.10 (2% voids at 100 PRD).

9.1.1 Validity of Approach

The void content of the 100 PRD specimens is calculated from densities obtained through gravimetric means as described in Chapter 8. Figure 8.7 illustrated the relationship between densities of specimens obtained with sealed and non-sealed surfaces, and suggested the error which can be introduced on specimens with void contents greater than 6.5% if density calculations are based on unsealed values. Because of the structure of the specimens resulting from the fabrication method it is advocated that a sealant is not applied to the specimens prior to the weighing operation. This approach may introduce errors on some specimens, particularly those compacted to level 3, but at the 100 PRD level where such a high compactive effort is applied to the material resulting in a high density, the calculated volumetric proportions should be unaffected. Therefore it is considered that the void content obtained from the 100 PRD specimens are representative values which may confidently be used to categorise different aggregate types.

9.2 THE MIX DESIGN PROCEDURE

Twenty seven specimens were manufactured using the PRD apparatus with 100 pen binder and limestone aggregate according to the compositional variations illustrated in Figure 8.5. The maximum densities for mix formulations at each binder content were calculated using equation 8.4, the results of which are shown in Table 9.2. These values are independent of the aggregate grading.

Table 9.2 Theoretical maximum densities of mix formulations

M_B %	SG agg	TSG mix
3.5	2.67	2.527
4.1	2.67	2.504
4.7	2.67	2.481

Table 9.3 Volumetric Proportions of specimens manufactured for the mix design procedure (Limestone Aggregate)

Spec No	Grading Exponent (n)	M_B (%)	Compaction Level	Density (g/ml)	Vv (%)	(VB) (%)	VMA (%)	PRD (%)
1	0.5	3.5	1	2.512	0.6	8.6	9.2	100
2	0.5	3.5	2	2.391	5.4	8.2	13.6	95.2
3	0.5	3.5	3	2.366	6.3	8.1	14.4	94.2
4	0.5	4.1	1	2.498	0.2	10.0	10.2	100
5	0.5	4.1	2	2.459	1.8	9.9	11.7	98.4
6	0.5	4.1	3	2.371	5.3	9.5	14.8	94.9
7	0.5	4.7	1	2.482	0.0	11.4	11.4	100
8	0.5	4.7	2	2.450	1.3	11.3	12.6	98.8
9	0.5	4.7	3	2.390	3.7	11.0	14.7	26.3
10	0.6	3.5	1	2.496	1.2	8.6	9.8	100
11	0.6	3.5	2	2.403	4.9	8.2	13.1	96.3
12	0.6	3.5	3	2.354	6.8	8.1	14.9	24.3
13	0.6	4.1	1	2.495	0.3	10.1	10.4	100
14	0.6	4.1	2	2.402	4.0	9.8	13.8	96.2
15	0.6	4.1	3	2.308	4.2	9.7	13.9	96.1
16	0.6	4.7	1	2.490	0.0	11.5	11.5	100
17	0.6	4.7	2	2.467	0.6	11.4	12.0	99.0
18	0.6	4.7	3	2.395	3.4	11.0	14.4	96
19	0.7	3.5	1	2.481	1.8	8.7	10.5	100
20	0.7	3.5	2	2.406	4.8	8.3	13.1	96.9
21	0.7	3.5	3	2.352	6.9	8.1	15.0	94.8
22	0.7	4.1	1	2.502	0.1	10.1	10.2	100
23	0.7	4.1	2	2.408	3.8	9.7	13.5	96.2
24	0.7	4.1	3	2.331	6.9	9.4	16.3	93.2
25	0.7	4.7	1	2.488	0.0	11.5	11.5	100
26	0.7	4.7	2	2.427	2.2	11.2	13.4	97.8
27	0.7	4.7	3	2.351	5.3	10.8	16.1	94.7

9.2.1 Analysis of volumetric proportions

The resulting volumetric proportions and relative levels of density for each specimen are shown in Table 9.3. It can be seen that the approach to the fabrication of specimens has successfully produced a range of volumetric proportions encompassing those which could be expected to be achieved in the field.

The specimens manufactured with the maximum binder content of 4.7% can be seen to exhibit a density in excess of the theoretical maximum when compacted to 100 PRD. This effect illustrates the phenomenon described in Chapter 8 where the application of a severe compactive effort causes a migration of binder and fines to the surface of the specimen, resulting in a flange of material protruding from the circumference on one side of the sample. This flange can easily be broken during handling, which means that the specimen is no longer representative of its original constitutive make up, resulting in the calculation of an apparently negative void content. These specimens represent the mix formulations which would exhibit bleeding and loss of structural integrity if secondary compaction in the wheel tracks led to an over-densification of material. It is important for the design procedure to identify mixtures which compact to a state of zero voids at refusal density, in order that recommendations on binder contents can be made with respect to material stability.

Examination of the specimens compacted to 100PRD demonstrates the effect of changing the aggregate gradation and binder content, with respect to the volumetric proportions of the mixture, emphasising the need to select a datum value by which aggregate types may be categorized. At the leanest binder content of 3.5%, the change in gradation from n = 0.5 to n = 0.7 increases the void content at 100 PRD from 0.6% to 1.8%, with a corresponding increase in VMA. Increasing the binder content depresses the volume of voids in the mixture to the state at a binder content of 4.7% where the volume of binder equals the VMA. Specimens compacted to levels 2 and 3 exhibit volumetric proportions representative of well compacted site material, with the lowest value of PRD corresponding to 93.2% for specimen n = 0.7, M_B = 4.1%, level 2.

The tabulated results of the volumetric compositions of the specimens give further evidence of the need to quote values of the volumetric proportions alongside percentage refusal densities. Specimens 6 and 21 have very similar PRD values but exhibit different compositions due to grading and binder content. This is an important shortfall of the PRD test as the recognition of differences in volumetric proportions is a significant indicator of changes in mechanical properties.

9.2.1.1 Screening the specimens

The objective of analysing the volumetric proportions of the twenty seven specimens is to rationalise the initial formulations to a number representing those mixtures which could be expected to exhibit accepTable mechanical properties. Specimen 1, corresponding to the datum formulation of n = 0.5, M_B = 3.5%, level 1, exhibited a void content of 0.6%, which sets the criterion for volumetric proportions on the chart shown in Figure 8.9. For the case of aggregate packing to 1% voids at the datum formulation the compositional criteria are as follows:

V_V : 3% to 6%

VMA : 13% to 16%

V_B : minimum 8%

Figures 9.1 to 9.3 show the volumetric proportions plotted on the appropriate chart. The mix formulations have been divided according to the gradation exponent for clarity. In this form of assessment, all 100PRD specimens are eliminated immediately as they fail to satisfy the recommended criteria, and have not been included on the charts. The remaining eighteen specimens show a degree of scatter in and around the target area, with nine samples achieving full compliance, two falling just outside the zone of acceptance, and seven specimens clearly failing to meet the recommended volumetric proportions. In the case of the n = 0.5 grading, compliance infringement has occurred through over-compaction of specimens 5 and 8 where PRDs in excess of 98% have been achieved. As the same compactive effort was applied to these samples as specimen 2, which has a PRD of 95%, the over compaction may be indicative of the workability of the mix formulations. A similar effect has occurred with specimen 17. However, there does not appear to be a regular pattern of increasing PRD with increasing binder content over the full range of specimens. It is likely that specimens 5, 8 and 17 were compacted at too high a temperature. Although a probe is used to monitor material temperature during the cooling phase, localised differences occur throughout the volume of the mould. Compaction temperatures at levels 2 and 3 can only be approximate values, which in practice will vary either side of the recommended figure. If the material is compacted at a higher temperature

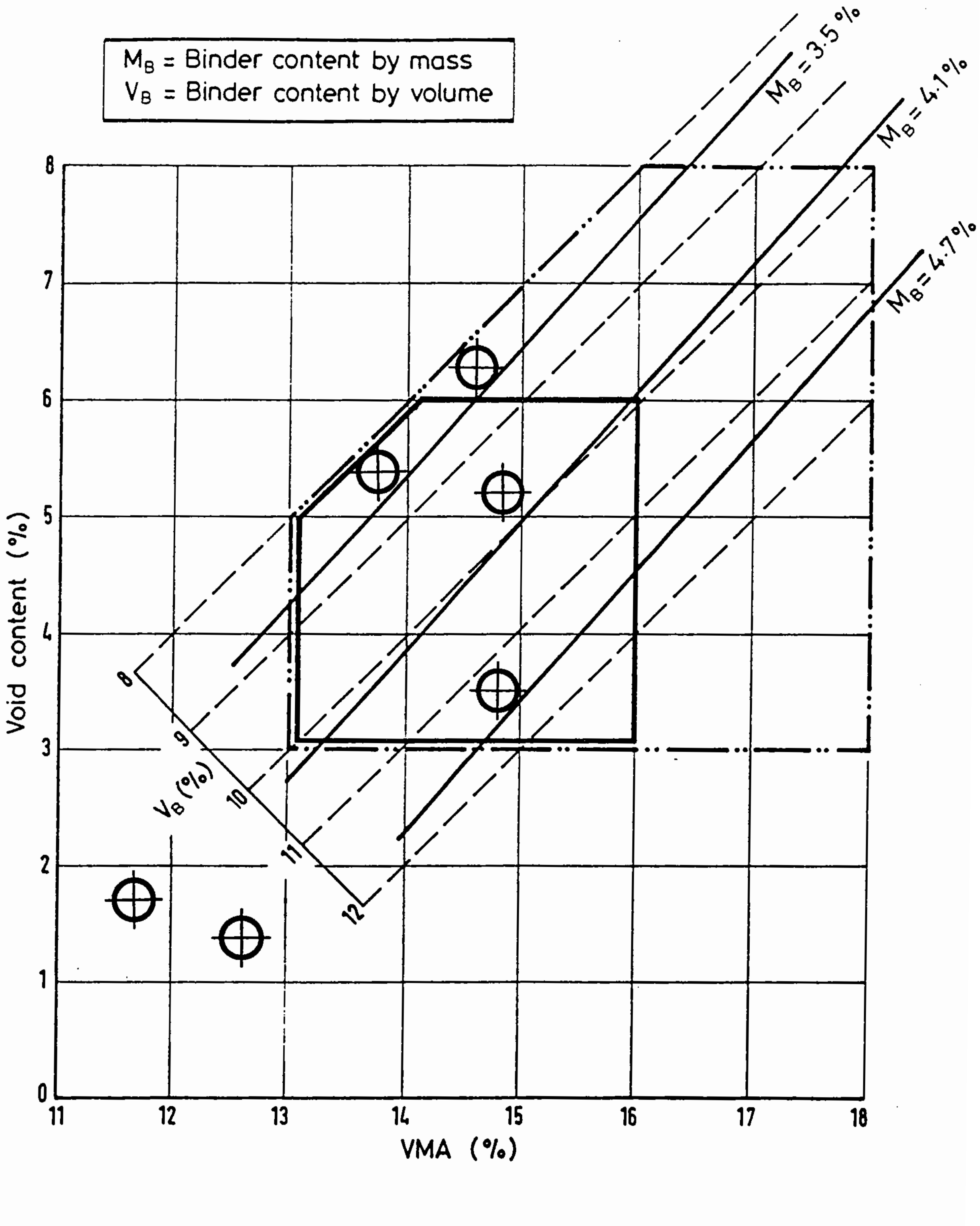

Figure 9.1 Volumetric proportions of limestone specimens: n=0.5, compaction levels 2 and 3.

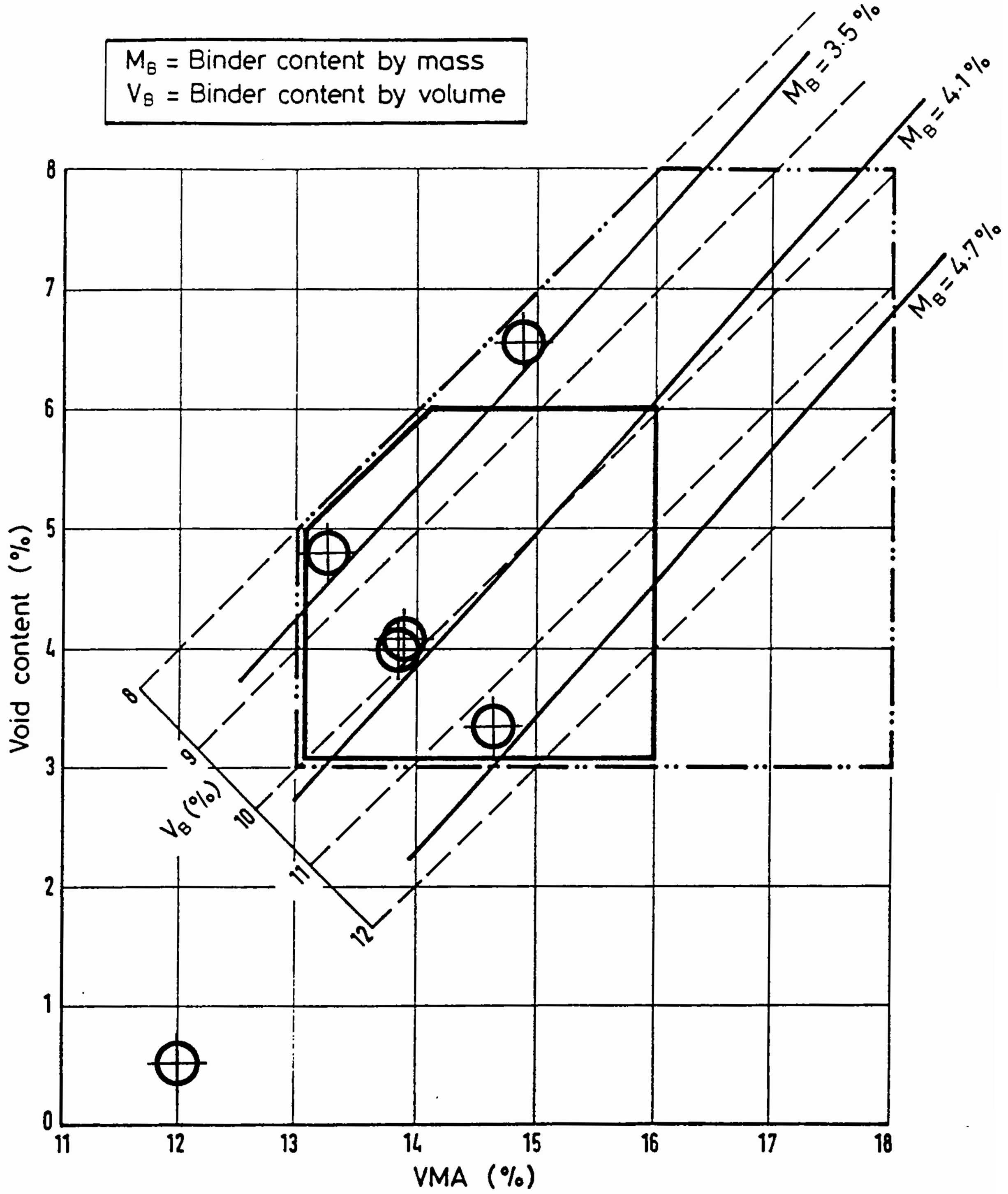

Figure 9.2 Volumetric proportions of limestone specimens: n=0.6, compaction levels 2 and 3.

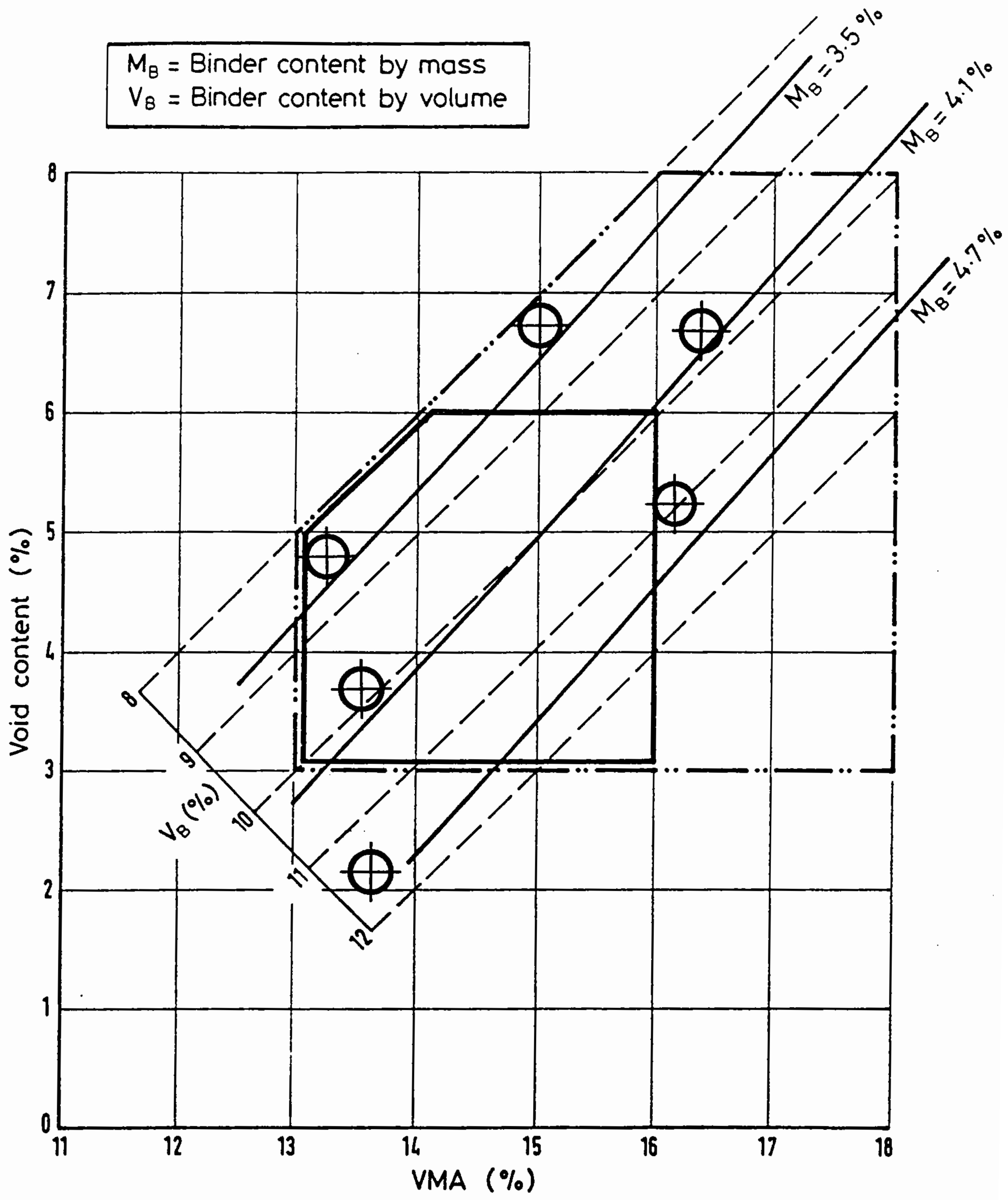

Figure 9.3 Volumetric proportions of limestone specimens: n=0.7, compaction levels 2 and 3.

than suggested, the binder viscosity will be elevated and hence, for a fixed compactive effort, the resulting density of the specimen will increase.

It is difficult to evaluate the state of over-compaction of bituminous materials through an assessment of the volumetric composition of the mixtures. 98 PRD is a value which can be achieved through field compaction, particularly with a workable mixture. It has been shown that good mechanical properties of roadbase materials are associated with low VMAs (1), but Figure 8.8 illustrates that there is a critical minimum, below which a reducing VMA detrimentally effects the mechanical performance. Identifying the optimum minimum VMA desirable appears a difficult problem, and will be associated with the aggregate type and characteristics, with the binder content as a second influential factor. Aggregates which exhibit 3% or more voids at the datum level may prove exempt from a requirement on minimum VMA, as the matrix cannot densify to a low level. The binder content would be the critical parameter in this case, where a high value could reduce the void content to a level where structural integrity is lost.

The minimum level of void content specified by the volumetric proportions charts was introduced to address the problem of over compacted mixtures. Although specimens 5 and 8 exhibit relative densities which equate to high levels of compaction, the situation which must be avoided is the potential of these formulations receiving further compaction under traffic action leading to a state of instability. Therefore, careful attention should be given to the mechanical properties of the same mix formulations at compaction level 3 which have satisfied the criteria on volumetric proportions, allowing a critical evaluation of these mixtures with reference to compaction sensitivity.

Specimens 12, 21 and 24 fail to adhere to the criteria through under compaction, where the aggregate structure is too open. If the volumetric criteria were to be taken as definitive values, then only the specimens lying in, or immediate to the zone of acceptance on the charts would be selected for mechanical testing. In order to make a full investigation for research purposes, all the specimens received RLIT and RLA tests.

9.2.2 Elastic Stiffness results

RLIT tests were performed in the NAT at 20°C and a load pulse rise time of 0.12 seconds. Tables 9.4 to 9.6 show the results of the tests, with specimens categorized according to the gradation exponent.

Table 9.4 Elastic Stiffness of specimens corresponding to grading exponent n = 0.5

Specimen No	Mix formulation	VMA (%)	Elastic Stiffness (MPa)
1	0.5 3.5 1	9.2	5100
2	0.5 3.5 2	13.6	3300
3	0.5 3.5 3	14.5	3300
4	0.5 4.1 1	10.3	4500
5	0.5 4.1 2	11.7	4400
6	0.5 4.1 3	14.8	2200
7	0.5 4.7 1	11.4	3000
8	0.5 4.7 2	12.6	2800
9	0.5 4.7 3	14.7	2200

Table 9.5 Elastic Stiffness of specimens corresponding to grading exponent n = 0.6.

Specimen No	Mix formulation	VMA (%)	Elastic Stiffness (MPa)
10	0.6 3.5 1	9.8	5200
11	0.6 3.5 2	13.2	3700
12	0.6 3.5 3	14.9	3500
13	0.6 4.1 1	10.4	3200
14	0.6 4.1 2	13.7	2600
15	0.6 4.1 3	13.9	2400
16	0.6 4.7 1	11.1	3000
17	0.6 4.7 2	11.9	2800
18	0.6 4.7 3	14.5	1700

Table 9.6 Elastic Stiffness of specimens corresponding to grading exponent n = 0.7

Specimen No	Mix formulation	VMA (%)	Elastic Stiffness (MPa)
19	0.7 3.5 1	10.3	5300
20	0.7 3.5 2	13.0	3200
21	0.7 3.5 3	15.0	2500
22	0.7 4.1 1	10.1	4000
23	0.7 4.1 2	13.5	2800
24	0.7 4.1 3	16.3	1400
25	0.7 4.7 1	11.2	3300
26	0.7 4.7 2	13.4	2000
27	0.7 4.7 3	16.1	1500

Values of elastic stiffness range from 1500MPa to a maximum of 5300MPa, with seven specimens exhibiting a value below the criterion of 2500MPa. Highest values for each formulation are coincident with specimens compacted to refusal, with lowest values represented by the level 3 compaction.

The relationship between elastic stiffness and level of compaction is shown in Figures 9.4 to 9.6, where the mix formulations are again categorized by gradation exponent for clarification purposes. Two conclusions may be drawn from the results:

i) Elastic stiffness increases as compaction level increases for a given mix formulation.

ii) For a given aggregate grading and a fixed level of compaction, elastic stiffness decreases as binder content increases.

The first conclusion is synonymous with elastic stiffness increasing with density, adhering to the philosophy that good compaction is associated with good mechanical performance. The second conclusion is not as obvious, but is again linked to the material density. In order to maintain the same void content in a given volume of material after increasing the binder content, then per unit of volume of mix there is a reduced quantity of aggregate and an increased quantity of binder. Therefore the mix density must decrease, and hence, result in a reduced elastic stiffness.

Figures 9.4 to 9.6 imply that compaction level and binder content are the most influential mix design parameters with reference to the elastic stiffness of the mix. Although increasing the gradation exponent increases the VMA for a fixed compaction level and binder content, the results do not indicate a general reduction in elastic stiffness as the aggregate grading is opened up. The change in VMA between the 0.5 and 0.7 grading exponents is less than 1.5% which, at a relative state of density of approximately 96PRD, would not be expected to significantly change the value of elastic stiffness (Figure 8.16). Although the measured values of stiffness of the 100PRD specimens do not exhibit a reduction as the aggregate grading exponent increases, at lower compaction levels, the elastic stiffnesses of the n = 0.7 specimens are less than those of the n = 0.5 specimens.

9.2.2.1 Elimination of mix formulations

Seven of the specimens fail to exceed the criterion of an elastic stiffness of 2500MPa. Of this number, four have the highest binder content of 4.7%, and the remainder are specimens with the intermediate binder content compacted to level 3. It can be seen that compaction level is a critical parameter at the 4.1% and 4.7% binder contents, but less so at 3.5%, particularly at the 0.5 and 0.6 grading exponents. As a generalisation all the lean mixtures, and well compacted mixes at the intermediate binder content adhere to the criterion, with the remaining specimens failing to exhibit satisfactory characteristics.

9.2.3 Resistance to permanent deformation

RLA tests were performed in the NAT at 40°C with repeated applications of a 100kPa axial stress. The test results are illustrated in Figures 9.7 to 9.12. Inspection of the Figures shows the variation in performance of the specimens, with some mixtures exhibiting good resistance to deformation

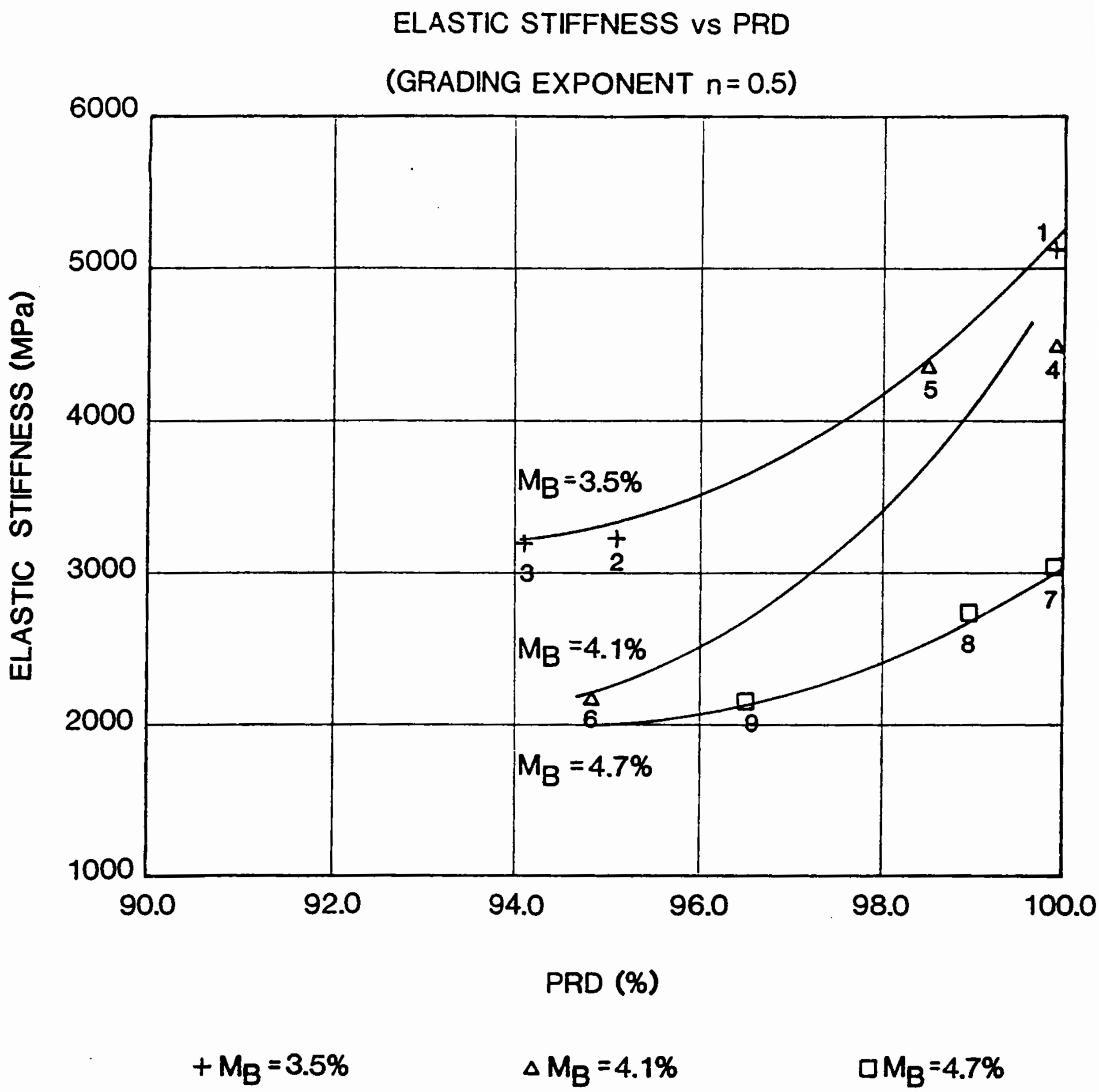

Figure 9.4 Influence of compaction on the elastic stiffness of test specimens. Limestone aggregate, grading exponent, n=0.5.

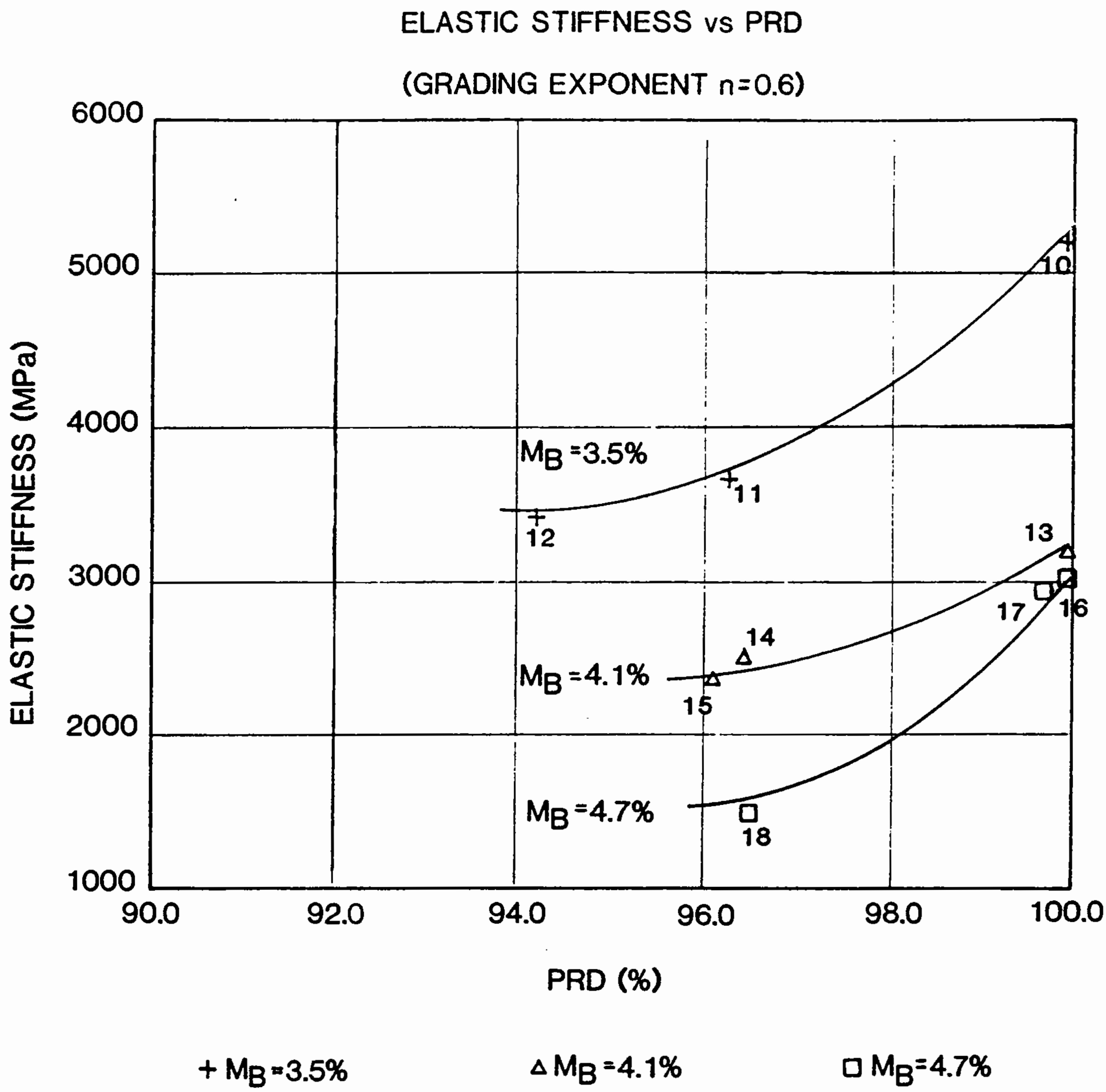

Figure 9.5 Influence of compaction on the elastic stiffness of test specimens. Limestone aggregate, grading exponent, n=0.6.

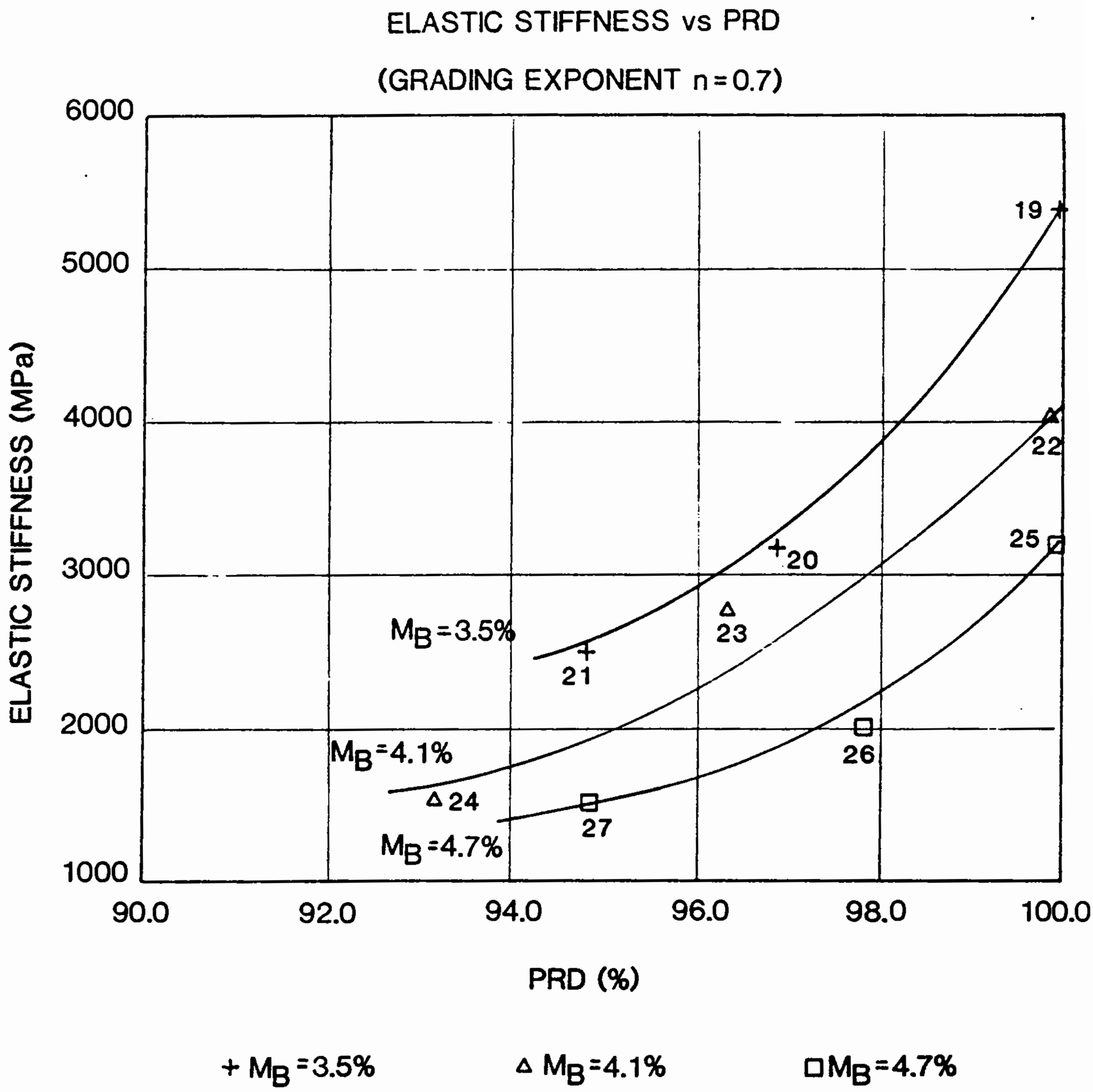

Figure 9.6 Influence of compaction on the elastic stiffness of test specimens. Limestone aggregate, grading exponent, n=0.7.

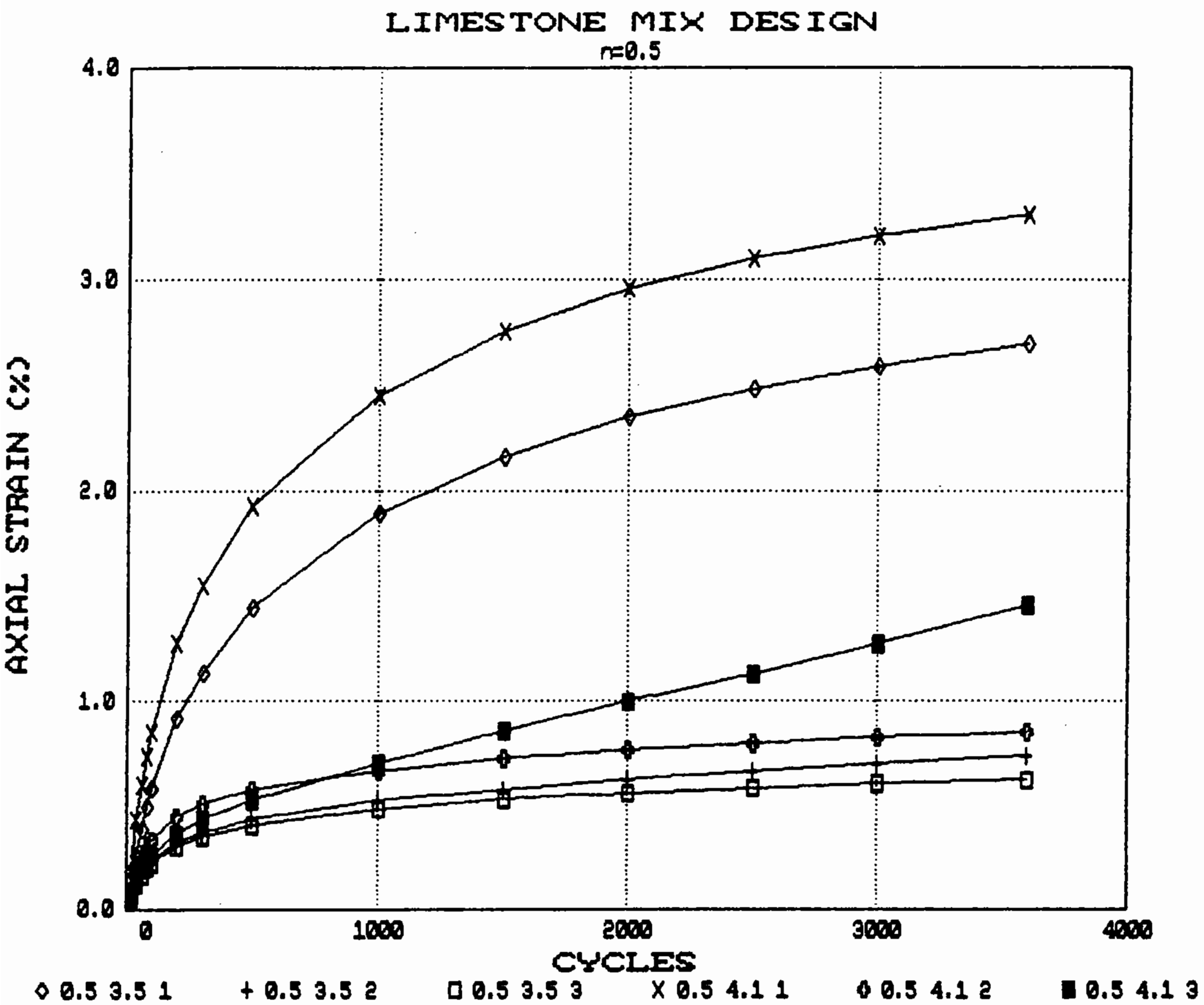

Figure 9.7 Deformation curves plotted from the results of RLA tests. Limestone aggregate, mix formulations corresponding to n=0.5, M_B=3.5% and n=0.5 M_B = 4.1%.

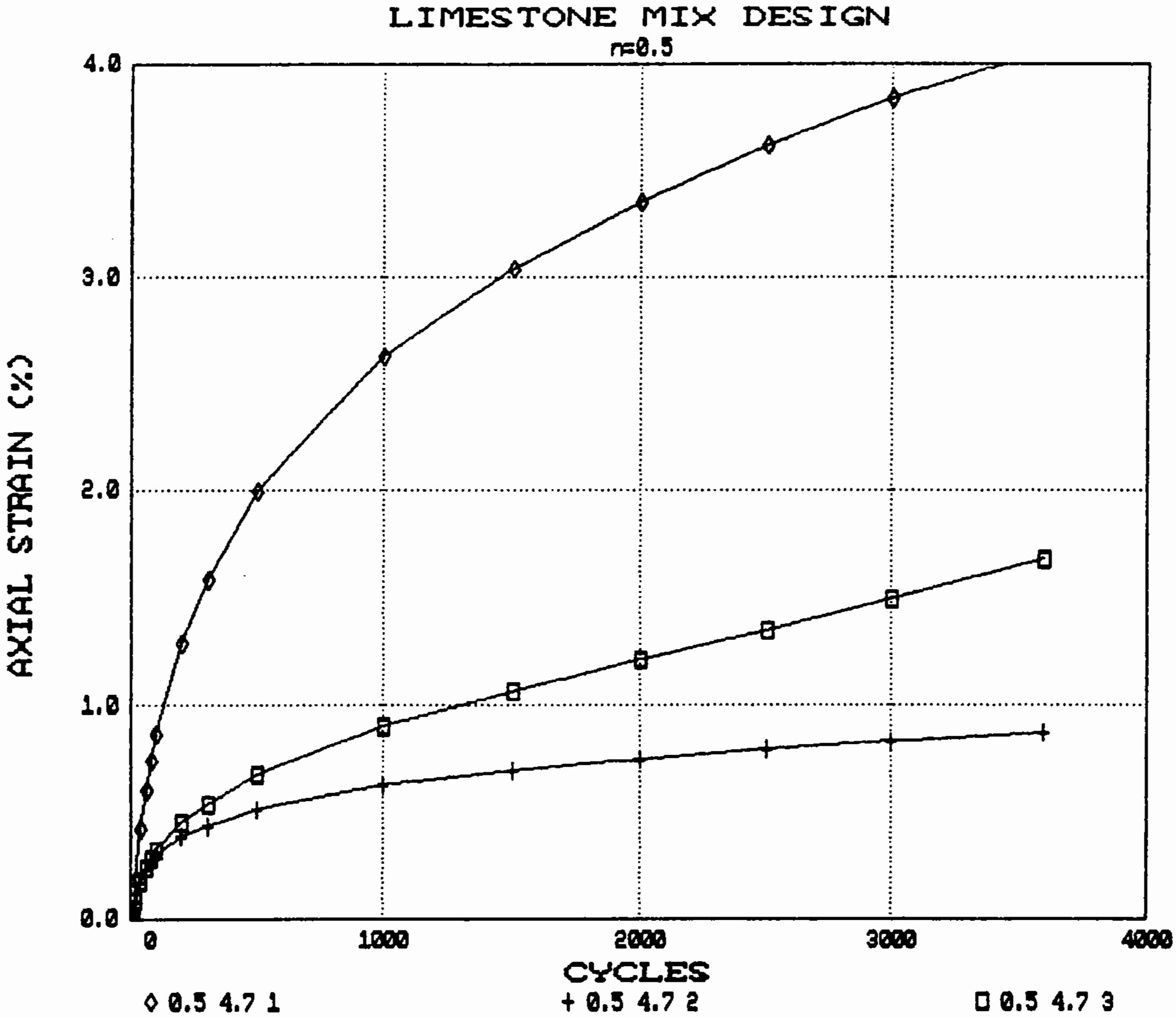

Figure 9.8 Deformation curves plotted from the results of RLA tests. Limestone aggregate, mix formulations corresponding to n=0.5, M_B = 4.7%.

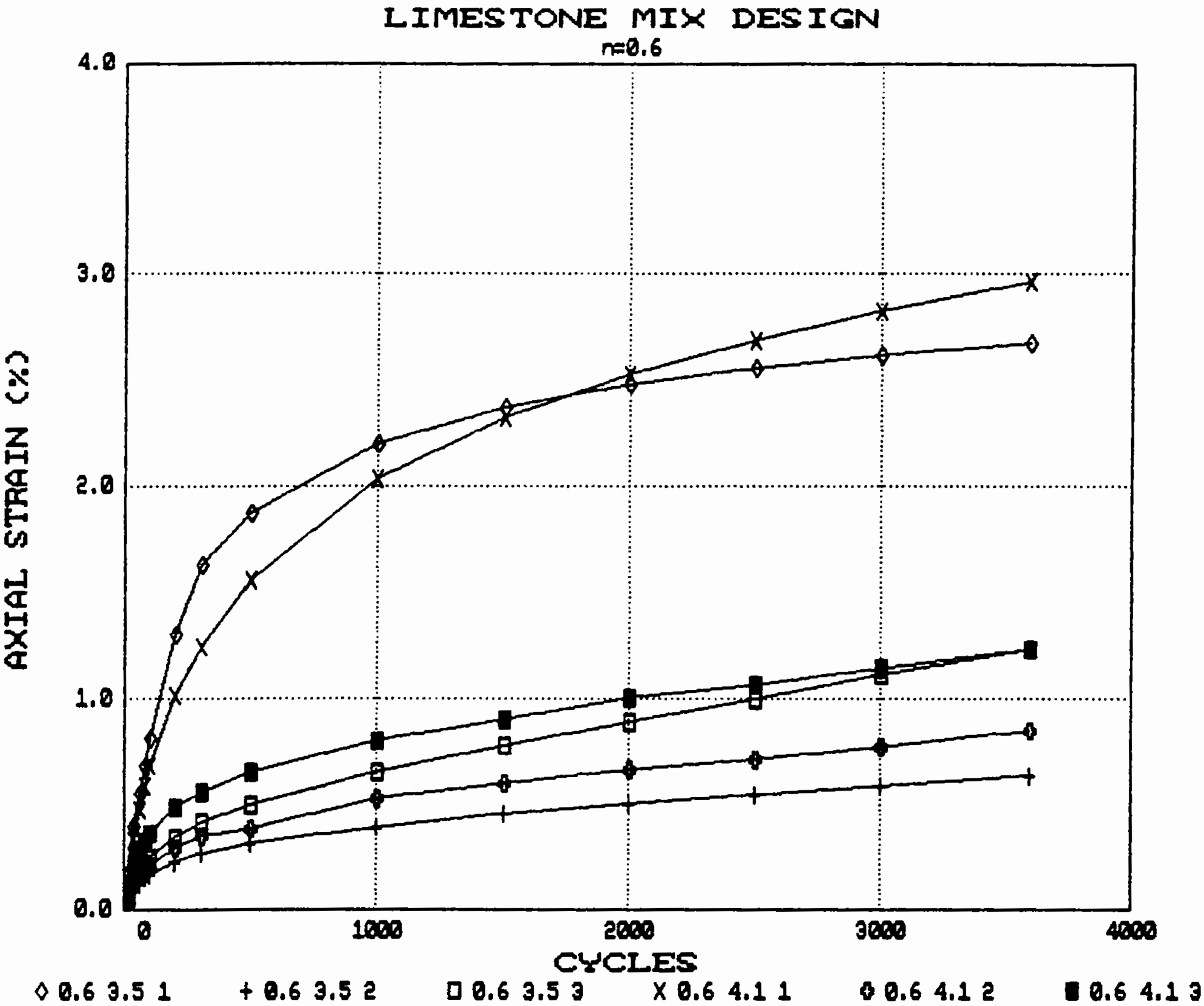

Figure 9.9 Deformation curves plotted from the results of RLA tests. Limestone aggregate, mix formulations corresponding to n=0.6, M_B = 3.5% and n=0.6, M_B = 4.1%.

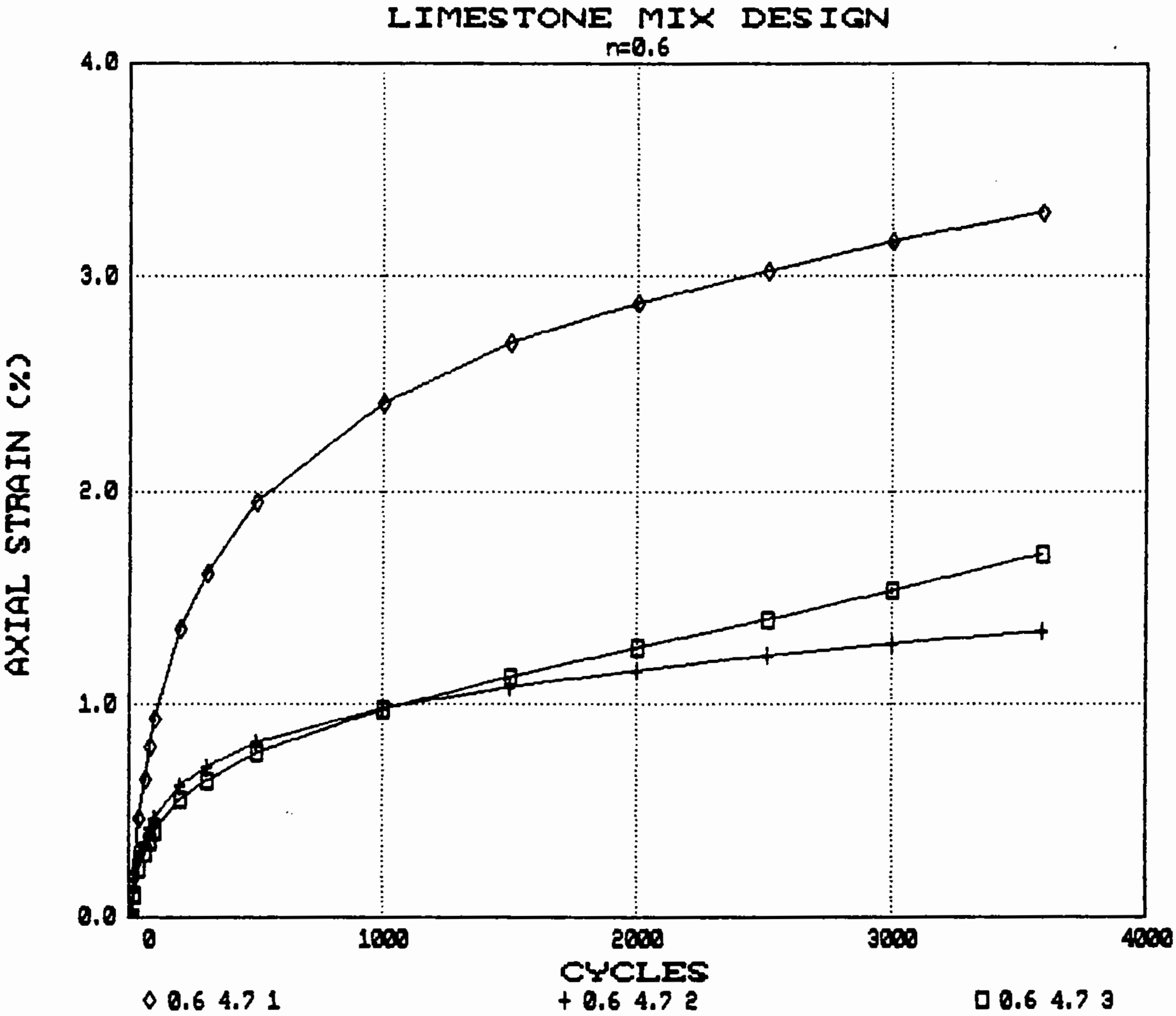

Figure 9.10 Deformation curves plotted from the results of RLA tests. Limestone aggregate, mix formulations corresponding to n=0.6 M_B = 4.7%.

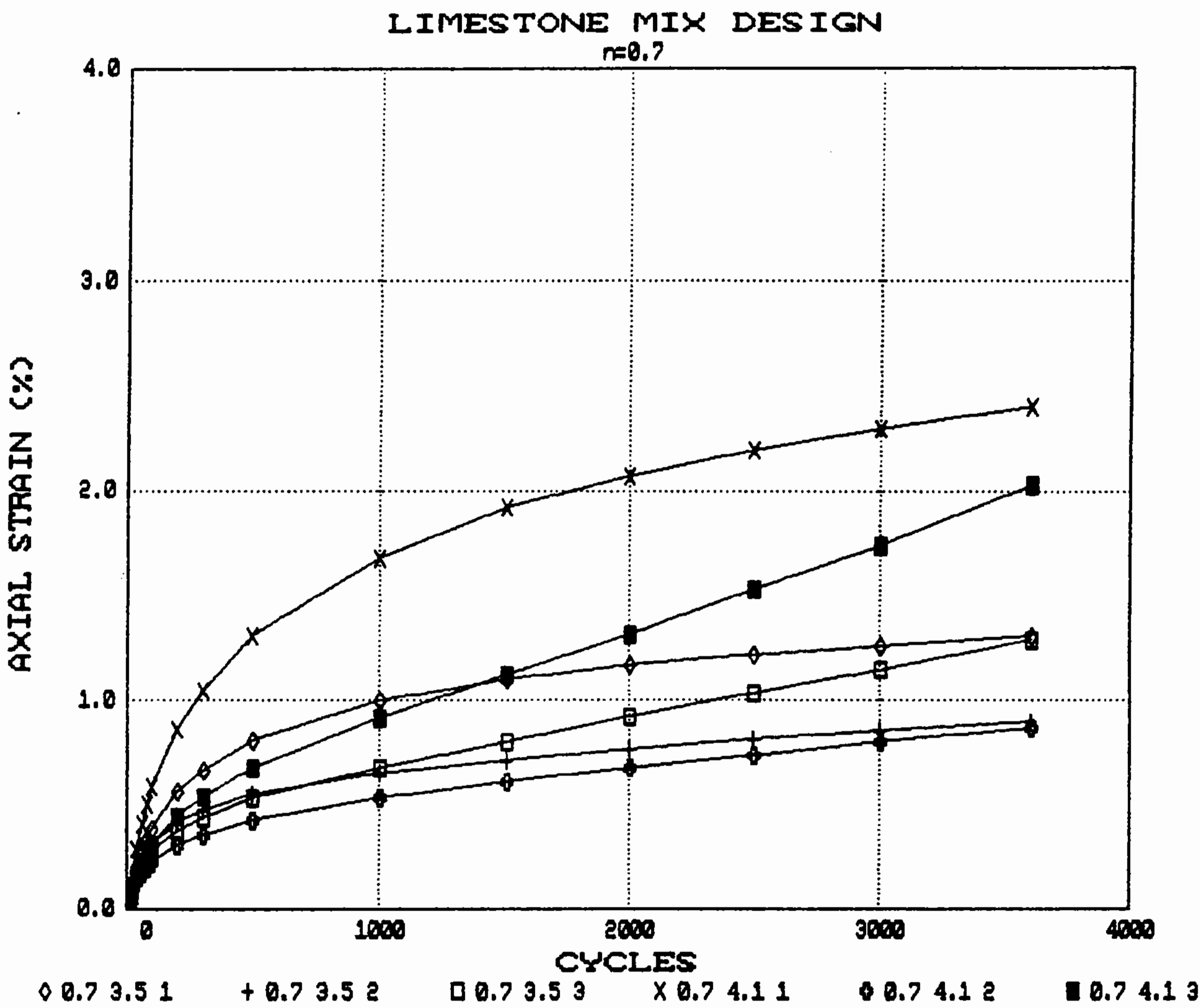

Figure 9.11 Deformation curves plotted from the results of the RLA tests. Limestone aggregate, mix formulations corresponding to n=0.7, M_B=3.5% and n=0.7, M_B=4.1%

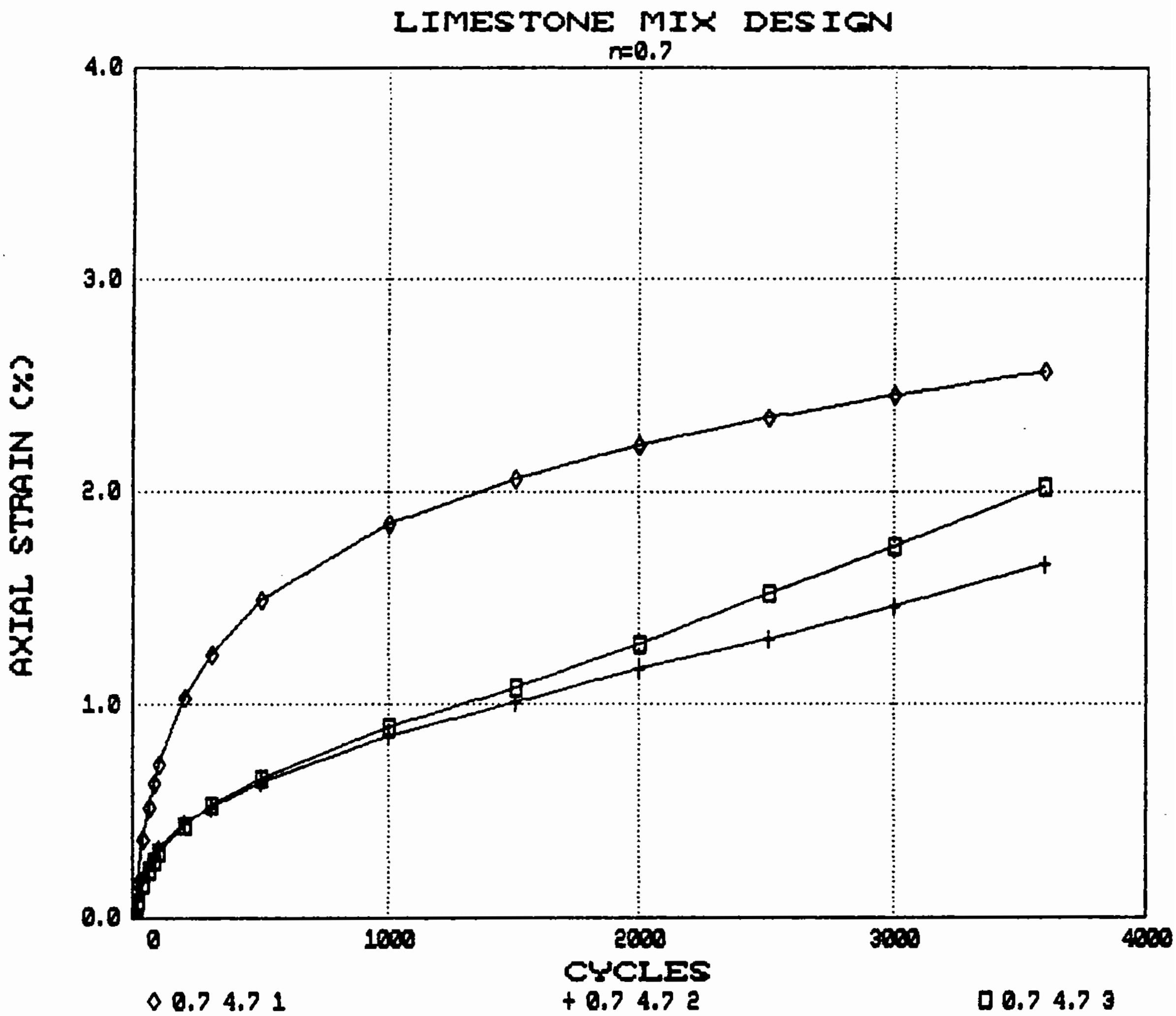

Figure 9.12 Deformation curves plotted from the results of RLA tests. Limestone aggregate, mix formulations corresponding to n=0.7, M_B= 4.7%.

and others clearly showing failure characteristics. The aggregate gradation and level of compaction are considered to have an influential role concerning this property and an evaluation of the test results should be made with this consideration in mind.

Figures 9.7 and 9.8 show the deformation curves for the specimens made at the n = 0.5 gradation exponent, with Figure 9.7 exhibiting the results of the 3.5% and 4.1% binder contents, and Figure 9.8 dealing with the 4.7% binder content. At this grading, the specimens compacted to 100PRD have exhibited very high deformations, which illustrates the critical state of over compacted mixtures. The leanest binder content of 3.5% (specimens 2 & 3) shows the greatest resistance to deformation, with specimen 5 (n = 0.5, $M_B = 4.1\%$, level 2) also exhibiting little permanent strain at the termination of the test. At binder contents 4.1% and 4.7% at compaction level 3, it can be seen that these specimens fail to meet the criterion of 1% allowable strain at the end of the test, and also exhibit an increasing rate of strain between 1000 and 3600 load applications.

The pattern of results for n = 0.6 gradation is similar, but the results appear to be more sensitive to compaction level. The only specimens which meet the criterion on deformation resistance are the n = 0.6, $M_B = 3.5\%$ level 2 and n = 0.6, $M_B = 4.1\%$ level 2 (11 and 14) mix formulations. However, both mix types exhibit higher strain rates and greater deformations if the compaction level is reduced (specimens 12 and 15), failing to comply with the 1% strain criterion. All specimens fabricated with a binder content of 4.7% show failure characteristics.

The coarsest grading corresponding to an exponent of n = 0.7 starts to show a reduction in the resistance of deformation compared with the two denser gradations. For corresponding mix formulations the n = 0.7 grading exhibits an increased permanent strain, with only specimens 20 (0.7,3.5,2) and 23 (0.7,4.1,2) marginally satisfying the criterion. One anomaly occurs with the specimen n = 0.7, M_B = 3.5%, 100PRD, which deforms much less than any other of the specimens compacted to refusal. This is due to the void content of the mixture maintaining a satisfactory value (1.8%), which allows a degree of further compaction under applied loading without complete loss of stability.

Chapter 8 discussed a means by which the rate of strain could be quantified by considering the quotient of the difference in strain between 3600 and 1000 load applications, and the value of strain at the 1000th load application. This evaluates the accumulation of strain during the secondary and tertiary phases of the test, and it was suggested that a 'strain rate index' in excess of 0.35 may be indicative of a formulation which could ultimately exhibit failure characteristics if the test were prolonged.

Tables 9.7 to 9.10 show values of the strain rate index and permanent strain at the termination of the test for each of the specimens. A final column introduces a relationship between ultimate strain and strain rate index in terms of their product, which could be used as an acceptance criterion for deformation testing. A limiting value on the product must be controlled by the criteria which have been applied to the ultimate strain and strain rate index. This would give a value of 0.35, which is satisfied by seven of the specimens. From the preceding discussion it would appear that an evaluation of the deformation resistance characteristics of the mix design specimens requires consideration of both the strain rate index and the ultimate strain exhibited at the termination of the test. The product of these two parameters provides a liaison, but does not produce a unique value which can be used to accept a fail specimen. For example, specimen 19 which gives a product value of 0.31, exhibits an ultimate strain of 1.3%, failing to meet the criterion applied to overall deformation.

The product value suggests an overall indication of deformation resistance but still requires assessments of the two parameters which are combined in order to make a full evaluation. Adopting this approach for the mix design exercise, the mixtures comprising an aggregate grading corresponding to n = 0.5 and a low binder content exhibited the most preferential characteristics concerning resistance to permanent deformation. Changes to the grading exponent and binder content lead to a reduction in resistance to deformation, with the amount of reduction exacerbated by low compaction.

Table 9.7 Ultimate strain and strain rate index for specimens at gradation exponent n = 0.5.

Specimen	Mix formulation	A : Strain at end of test (%)	B : Strain rate index	Product (A x B)
1	0.5 3.5 1	2.67	0.37	0.99
2	0.5 3.5 2	0.74	0.29	0.22
3	0.5 3.5 9	0.62	0.21	0.13
4	0.5 4.1 1	3.31	0.29	0.98
5	0.5 4.1 2	0.85	0.19	0.17
6	0.5 4.1 3	1.46	0.77	1.12
7	0.5 4.7 1	4.06	0.46	1.87
8	0.5 4.7 2	0.87	0.25	0.22
9	0.5 4.7 3	1.68	0.63	1.06

Table 9.8 Ultimate strain and Strain rate index for specimens at gradation exponent n = 0.6.

Specimen	Mix formulation	A : Strain at end of test (%)	B : Strain rate index	Product (A x B)
10	0.6 3.5 1	2.68	0.18	0.48
11	0.6 3.5 2	0.64	0.44	0.28
12	0.6 3.5 3	1.23	0.66	0.81
13	0.6 4.1 1	2.96	0.38	1.12
14	0.6 4.1 2	0.83	0.42	0.35
15	0.6 4.1 3	1.23	0.36	0.44
16	0.6 4.7 1	3.30	0.30	3.6
17	0.6 4.7 2	1.35	0.27	0.36
18	0.6 4.7 3	1.71	0.54	0.92

Table 9.9 Ultimate strain and strain rate index
for specimens at gradation exponent n = 0.7

Specimen	Mix formulation	A : Strain at end of test (%)	B : Strain rate index	Product (A x B)
19	0.7 3.5 1	1.31	0.24	0.31
20	0.7 3.5 2	0.90	0.26	0.23
21	0.7 3.5 3	1.29	0.63	0.81
22	0.7 4.1 1	2.40	0.35	0.84
23	0.7 4.1 2	0.87	0.43	0.37
24	0.7 4.1 3	2.03	0.92	1.87
25	0.7 4.7 1	2.57	0.31	0.80
26	0.7 4.7 2	1.67	0.72	1.20
27	0.7 4.7 3	2.02	0.96	1.94

9.2.3.1 Volumetric composition and resistance to permanent deformation

The twenty seven mix matrix of the design procedure produces a variation in volumetric composition which should encompass the optimum mix formulation achievable for available resources. The data provides useful information on the effect of density and binder content on mix properties. Figure 9.13 illustrates the effect of VMA on the level of strain measured at the end of the RLA test. Although considerable scatter is shown by the results, it seems reasonable to conclude that optimum resistance to deformation occurs at a VMA of 13% - 14%. Much of the scatter is due to variations in the binder content, which for a given state of aggregate packing, has significant influence on the volumetric composition. Figures 9.14 to 9.16 examine the influence of binder content and gradation exponent on the axial strain induced in the specimens. By considering the ordinate of 1% as a failure criterion, the

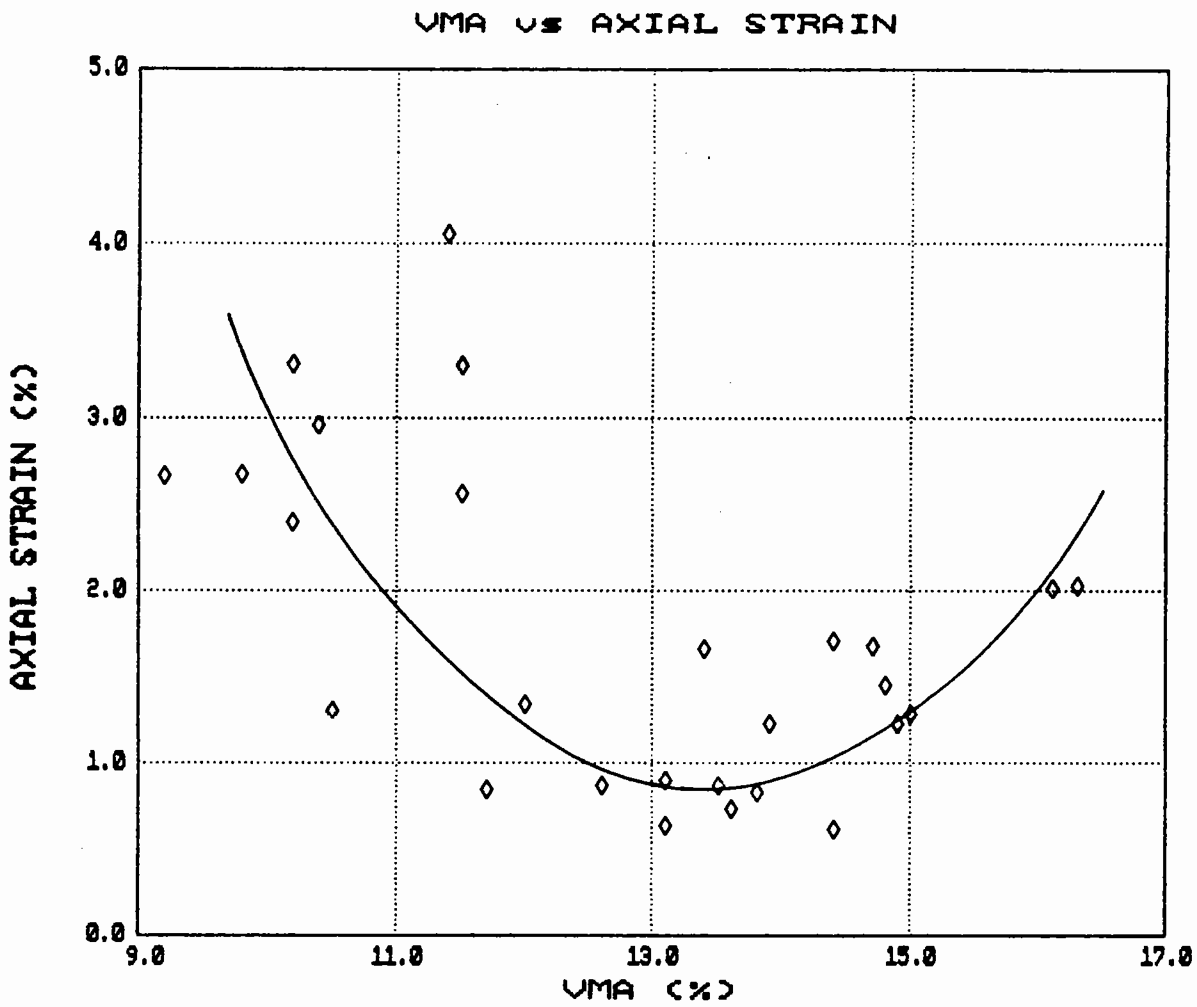

Figure 9.13 The influence of compaction on the resistance to permanent deformation for the 27 mix design specimens.

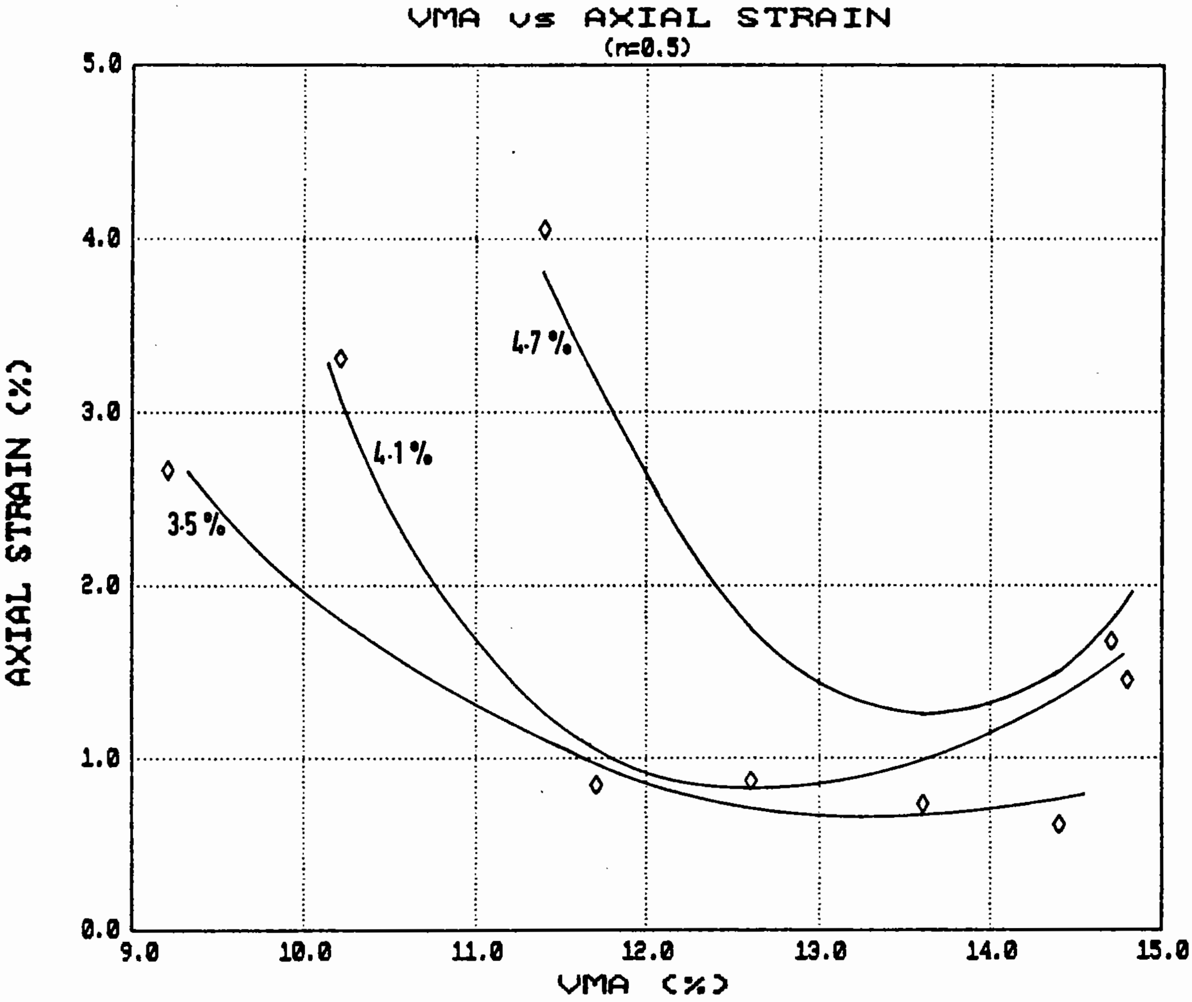

Figure 9.14 Influence of compaction on the resistance to permanent deformation. Limestone aggregate, n=0.5.

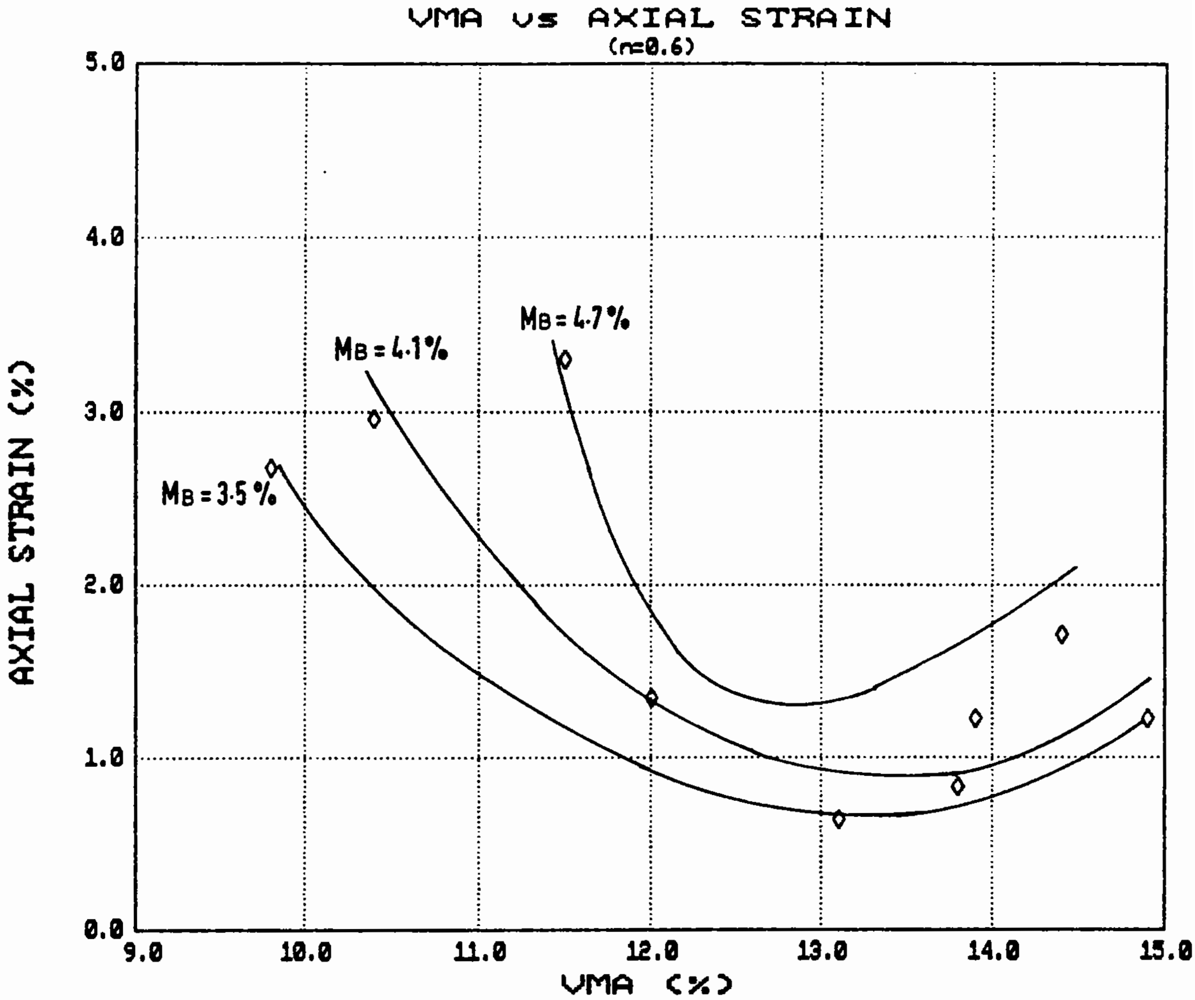

Figure 9.15 Influence of compaction on the resistance to permanent deformation. Limestone aggregate, n=0.6.

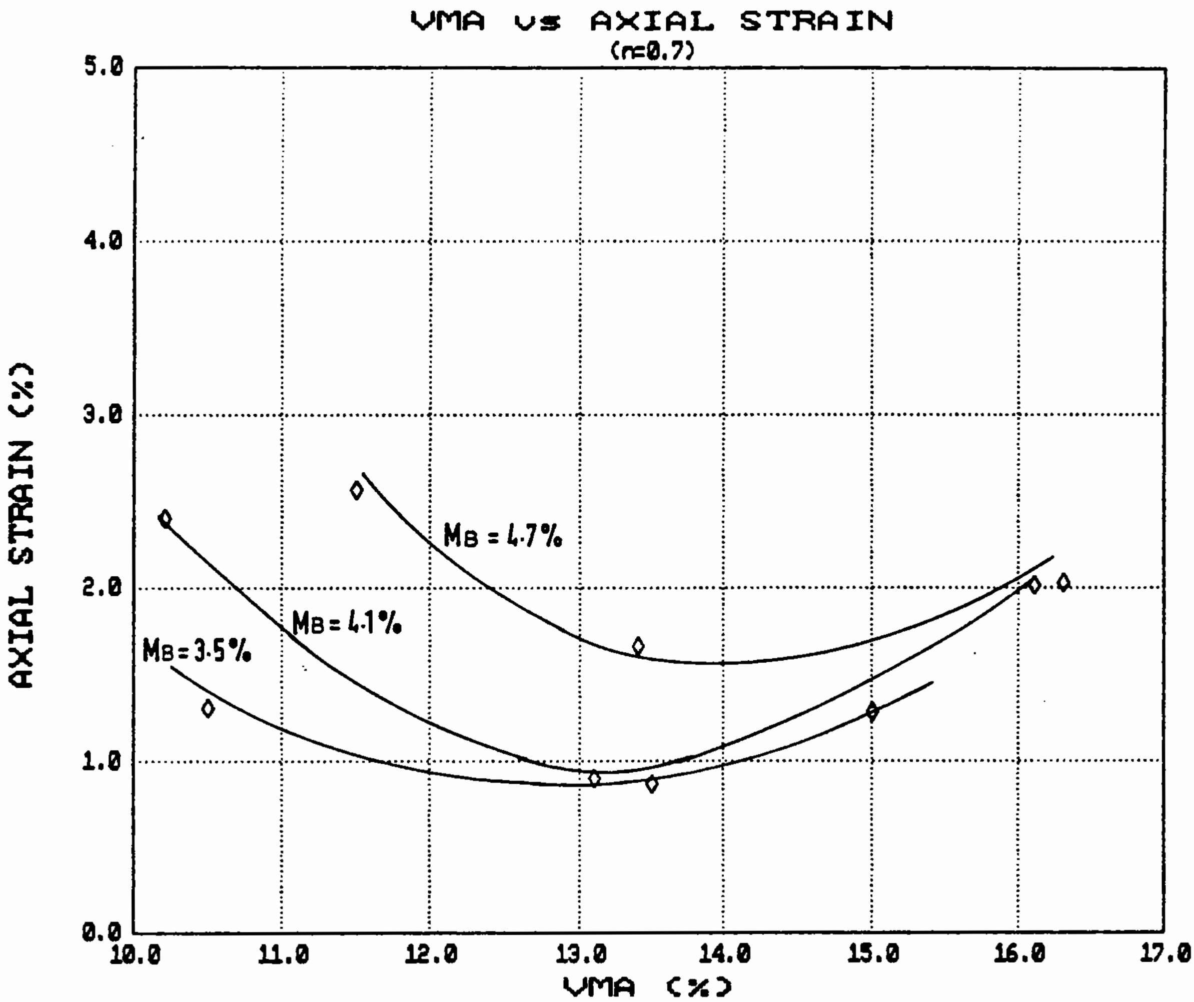

Figure 9.16 Influence of compaction on the resistance to permanent deformation. Limestone aggregate, n=0.7.

sensitivity of mix formulations to variations in density may be observed. In each case over, and under-compaction adversely effects deformation resistance, with increasing binder content reducing the tolerable range of VMA which meets the specification. The greatest tolerance occurs with the n = 0.5 M_B = 3.5% mixes where a VMA of 11.0%, corresponding to a PRD in excess of 99%, must be achieved before deformations exceed the failure criterion. At each gradation exponent, the high level of binder content, 4.7% is least likely to adhere to the specification.

9.2.4 Fatigue Resistance

Laboratory tests to establish fatigue relationships of specific mix formulations require the testing of a large number of specimens. Consequently, for practical mix design purposes the resistance to fatigue cracking was estimated using a prediction model. The input parameters required in order to use the prediction method are the volume of binder in the mix, (V_B) and the initial softening point of the binder (SP_i) (51).

Concerning the properties of elastic stiffness and resistance to permanent deformation, the mix formulations which exhibited optimum performance corresponded to those with the densest aggregate grading and the leanest binder content. Fatigue lives are mainly influenced by the quantity of binder in the mix, with an increasing binder content leading to greater fatigue lives. As the mix formulations which correspond to high values of elastic stiffness and good resistance to permanent deformation have binder contents of 3.5%, the fatigue lives of these mixtures will be relatively low. Any increase in binder content will start to compromise on deformation resistance and elastic stiffness.

Figure 9.17 shows fatigue lines based on specimens n = 0.5, M_B = 3.5% level 2 and n = 0.6, M_B = 4.1% level 2 (2 and 14), both of which exhibit satisfactory properties as measured by the NAT, and which represent the tolerances on grading and binder contents within which performance specifications are achieved. Specimen 14 can be seen to have an improved fatigue life compared with specimen 2 by a factor of approximately 2 for a fixed tensile strain, due to the increased quantity of binder in the volumetric composition. However, consideration must be given to the higher elastic stiffness of specimen 2, which will result in a reduced tensile strain on the underside of the material for a fixed stress application. The n = 0.5, M_B = 3.5% level 2 formulation also exhibits preferred deformation resistance with a lower strain rate index, to the mixture with a higher fatigue life. All these parameters must be considered when recommending the optimum mix formulation resulting from the design procedure. The fatigue life of the lean mixture of specimen 2 can be improved by increasing the thickness of the roadbase layer, which becomes an economic factor, and emphasises the relationship with pavement design.

The mix design procedure can identify a range of mix formulations with satisfactory mechanical properties according to end-product specifications. The criteria applied are general and do not take account of different pavement designs, which influences the roles of different properties. The fatigue life of material can be significantly effected by layer thickness and hence pavement

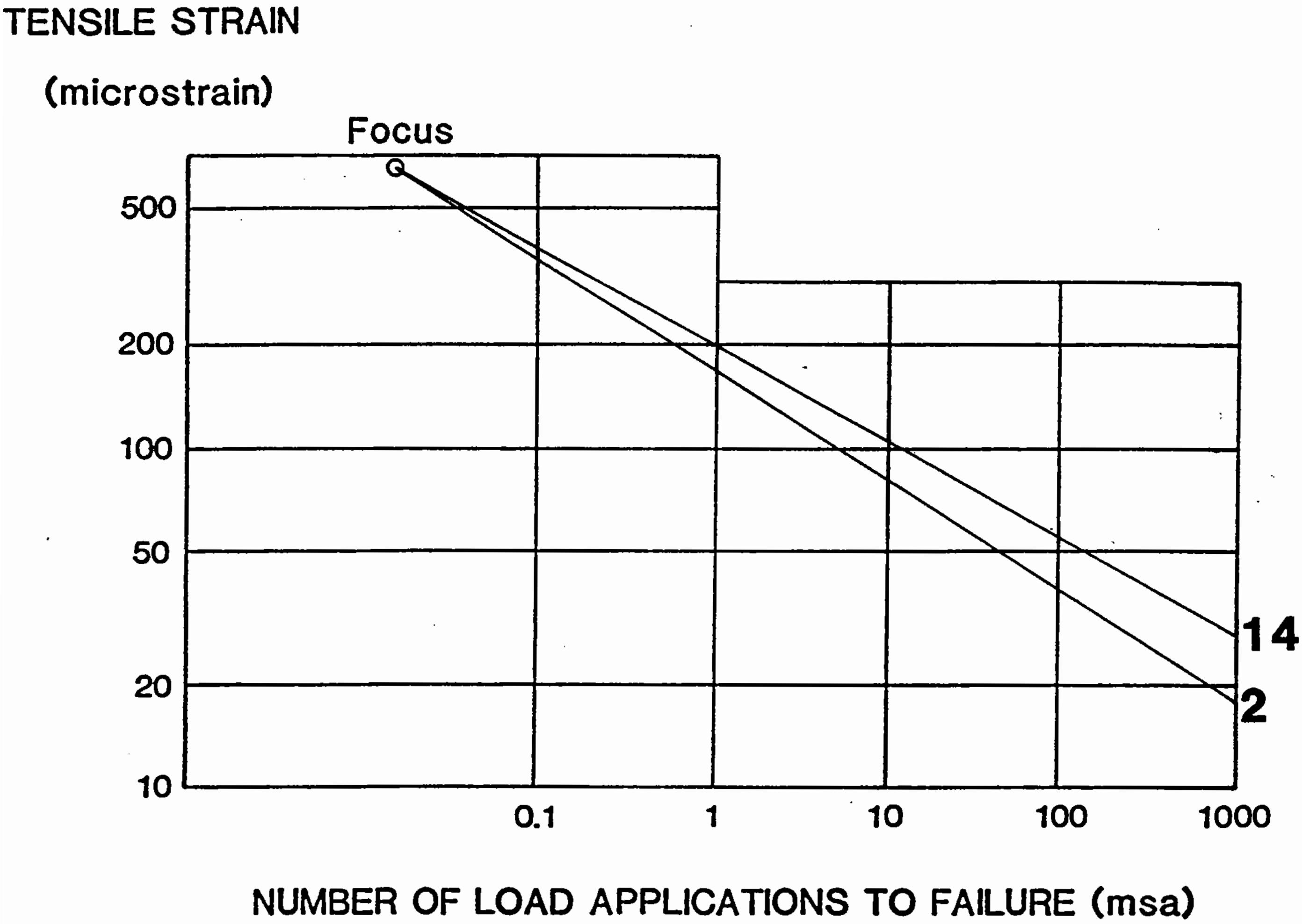

Figure 9.17 Fatigue lines for specimens 2 and 14.

design, which in turn should consider the material properties generated through mix design.

9.3 DISCUSSION

The new approach to mix design, performed on a limestone aggregate and a conventional straight run 100 pen binder has been able to identify a series of mix formulations which satisfy criteria in terms of end product mechanical properties. Mix formulations comprising the densest gradation under consideration, n = 0.5, and the leanest binder content of M_B = 3.5% were shown to exhibit adequate volumetric proportions, and mechanical properties in excess of the recommended levels. The selection of a target gradation and binder content from the procedural results, must take account of tolerances required by the material supplier. The n = 0.5 grading allowed the greatest variation in compactive effort whilst still retaining sound mechanical properties, and exhibited the least sensitivity to changes in binder content. From these observations the following recommendations can be made concerning the use of these materials in roadbase construction.

1) The material supplier should target on an aggregate gradation corresponding to a curve specified by:

$$P = 94\,\frac{(d^{0.5} - 0.274)}{(D^{0.5} - 0.274)} + 6 \qquad (9.1)$$

where P = percentage of material passing a sieve of size d mm

D = maximum particle size in mm

(2) The target binder content should be 3.8%

(3) The material should be compacted on site to a minimum of 95 PRD and not exceeding a maximum of 97 PRD

The selection of binder content at 3.8% averages the lean and intermediate levels of binder content employed in the mix design exercise. This facilitates small variations either side of this value without detrimentally effecting the elastic stiffness or deformation resistance qualities. The value also compromises on the fatigue lives shown in Figure 9.17. The importance of compaction level is recognized, with critical levels of PRD introduced to try to ensure against over and under compaction of the material. A 2% range of allowable compaction may appear too severe, but good rolling practice should be able to adhere to such recommendations without excessive difficulty.

Finally specification compliance should concentrate on the mechanical properties of the site laid material. Variations in mixture composition in terms of aggregate grading and binder content are inevitable, as are variations in mechanical properties, but the design procedure should identify mix formulations capable of tolerating small deviations without compromising material performance.

The major problem is the accuracy to which a quarry can blend aggregate stockpiles to an individual grading curve and maintain such a blend for a considerable tonnage of material. Practical problems such as these must be addressed in the implementation of a mix design procedure and material specifications selected with such problems under consideration.

9.4 GRANITE AGGREGATE

The design procedure was performed on the granite aggregate from Bardon Hill Quarry in Leicestershire. Table 9.1 illustrated the difference in packing characteristics between the limestone and granite aggregates, with the dominant angular structure of the granite resulting in a higher void content at the datum level of compaction.

Table 9.10 and Figures 9.18 and 9.19 show selected results of the volumetric proportions and the measured mechanical properties of some of the specimens, which illustrate the effect of changing the mix variables. A more detailed description of results is given in Appendix B.

Table 9.10 Volumetric proportions of specimens manufactured in the mix design procedure (granite aggregate).

Grading exponent(n)	MB (%)	Compaction Level	Density (g/ml)	Vv (%)	V_B (%)	VMA (%)	PRD (%)	Elastic Stiffness (MPa)
0.5	3.5	1	2.602	1.7	8.9	10.6	100	4600
0.5	3.5	2	2.449	7.5	8.4	15.9	94.1	3600
0.5	4.1	2	2.466	5.9	9.9	15.8	95.3	3800
0.5	4.7	2	2.509	3.3	11.5	14.9	97.8	2500
0.6	3.5	2	2.402	5.9	8.5	14.4	96.1	4000
0.6	4.1	2	2.431	7.3	9.7	17.0	93.5	3500
0.6	4.7	2	2.454	5.5	11.3	16.7	95.7	2800
0.7	3.5	2	2.457	7.2	8.4	15.6	95.7	2300
0.7	4.1	2	2.442	6.8	9.8	16.6	95.1	2200

The results show a similar pattern to that exhibited by the limestone specimens, with optimum mechanical properties coinciding with the densest aggregate gradation and the leanest binder content. The curves resulting from the RLA tests suggest that the use of a binder content of 4.7% causes a significant reduction in the resistance to permanent deformation of mixtures fabricated with granite aggregate. This effect appears more severe than observed with the limestone specimens. The use of a coarse gradation corresponding to an exponent of n = 0.7 also appears to cause the breakdown of the structural integrity of the matrix in a more exaggerated manner than was manifested in the limestone mix design.

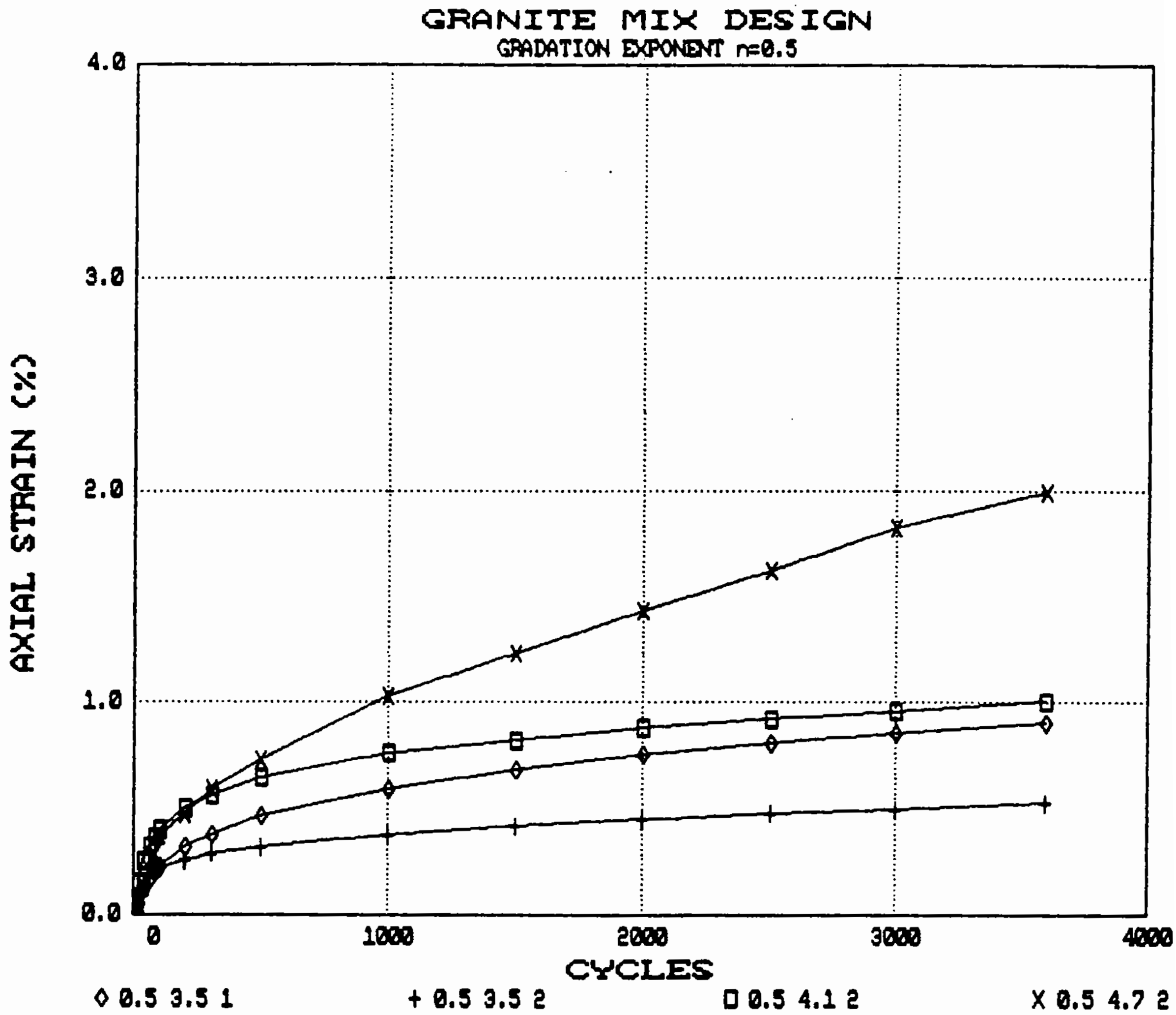

Figure 9.18 Deformation curves plotted from the results of RLA tests. Granite aggregate, selected mix formulations.

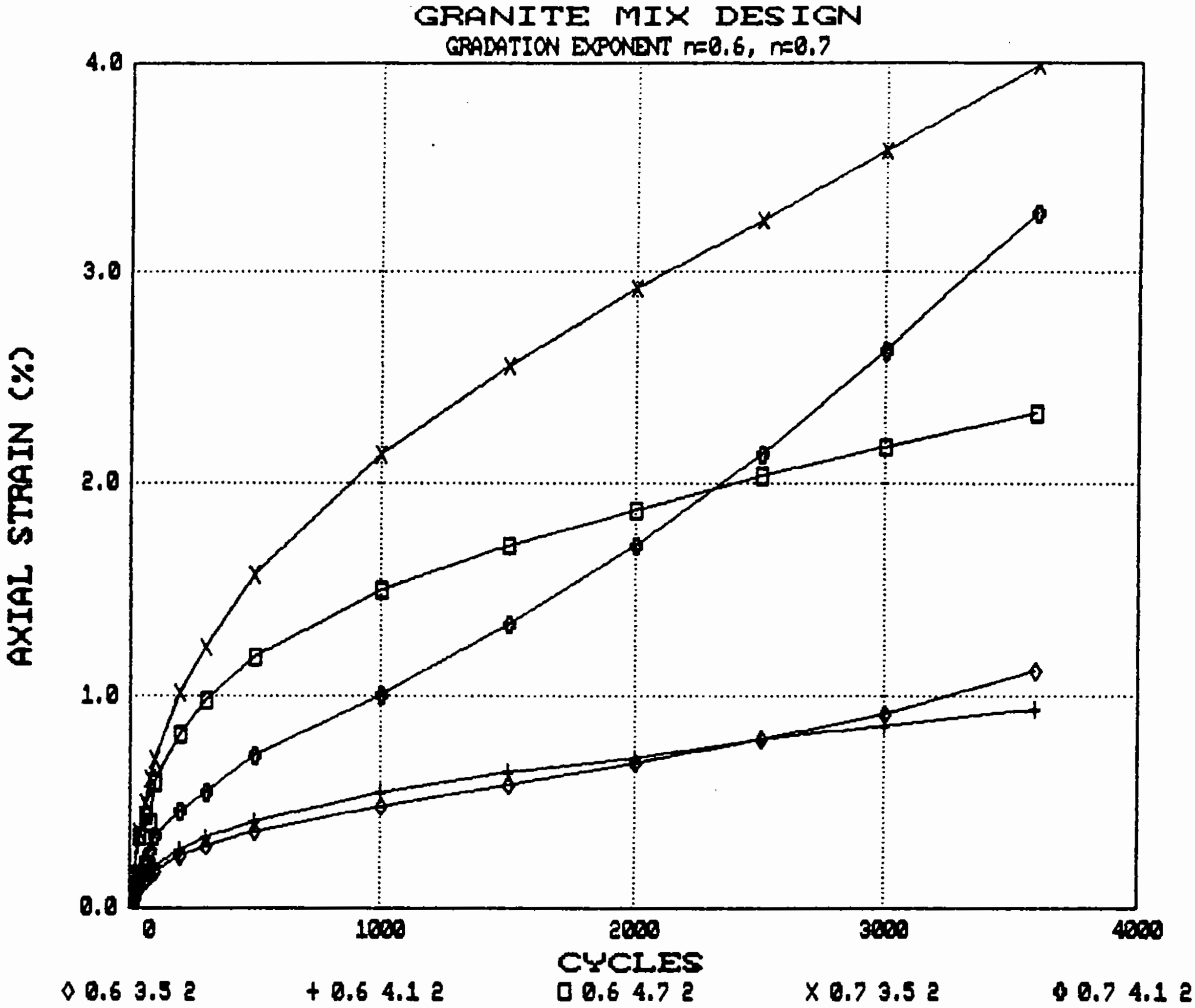

Figure 9.19 Deformation curves plotted from the results of RLA tests. Granite aggregate, selected mix formulations.

The elastic stiffness values obtained from RLIT testing adhere to the expected format of a decreasing stiffness proportional to an increasing VMA. The range of elastic stiffness values is very similar to that resulting from the limestone specimens, with failed mix formulations corresponding to combinations of high binder contents and low levels of compaction, although it is worthy of note that all specimens compacted at levels 2 and 3 at the n = 0.7 gradation exponent failed to exhibit values of elastic stiffness in excess of 2500MPa.

Properties associated with the packing charateristics of the aggregate perform an influential role on the compaction of the material. The surface roughness and inter-particle friction lead to a lower PRD for a given compactive effort for the granite specimens. This suggests that compaction control should be another variable parameter which should be governed by the aggregate type used in the mix. This point provides further evidence of the significance of the type of aggregate which is used in the manufacture of bituminous roadbase materials. Properties associated with the aggregate have been shown to influence the volumetric proportions of the mixtures, and the degree of compaction in response to a given compactive effort. These observations further emphasise the need to individualise design requirements according to the particular source of material, as opposed to the all-embracing specifications currently recommended in British practice.

9.5 DENSE GRADATIONS ON THE FINE SIDE OF THE MAXIMUM DENSITY CURVE

The advocated philosophy of the mix design method has been to maintain gradations on the coarse side of the maximum density, or 'Fuller' curve. Chapter 8 discussed the ideology of ensuring a dominant coarse aggregate skeleton by compromising on the proportion of fine aggregate within the grading, facilitating the manufacture of lean mixtures.

Results from the mix design procedure using two types of aggregate have suggested that mix formulations which exhibit preferential mechanical properties correspond with a combination of the densest grading and leanest binder content. Although it is clear that the procedure is able to identify the optimum mix formulations under consideration, it must be questioned whether the range of mixtures included within the design procedure comprise the optimum mix formulation achievable for the given source of material.

In order to fully qualify the approach to mix design advocated in this research, it was necessary to investigate the effect of changing the aggregate grading exponent to a value fine of the $n = 0.5$ curve. The limestone aggregate was selected for this investigation, and gradations corresponding to exponents of $n = 0.3$ and 0.4 were included for evaluation. The format of three binder contents was repeated, although specimens were manufactured at only two compaction levels.

Volumetric proportions and measured values of elastic stiffness for the specimens manufactured at the fine gradings are indicated in Table 9.11.

Table 9.11 Volumetric proportions and elastic stiffness values of specimens manufactured using grading exponents n = 0.3 and n = 0.4.

Grading Exponent	MB (%)	Compaction Level	Density (g/ml)	V_V (%)	V_B (%)	VMA (%)	PRD (%)	Elastic Stiffness (MPa)
0.3	3.5	1	2.476	2.0	8.9	10.9	100	7000
0.3	3.5	2	2.380	5.8	8.5	14.3	96.1	8500
0.3	4.1	1	2.472	1.3	10.4	11.7	100	6300
0.3	4.1	2	2.414	3.6	10.1	13.7	97.6	6200
0.3	4.7	1	2.456	1.0	11.8	12.8	100	3700
0.3	4.7	2	2.421	2.4	11.6	14.0	98.5	3400
0.4	3.5	1	2.479	1.9	8.9	10.8	100	7600
0.4	3.5	2	2.379	5.9	8.5	14.4	95.9	8000
0.4	4.1	1	2.470	1.3	10.4	11.7	100	7500
0.4	4.1	2	2.418	3.4	10.1	13.5	97.8	5600
0.4	4.7	1	2.460	0.8	11.8	12.6	100	4700
0.4	4.7	2	2.441	1.6	11.7	13.3	99.2	4200

Figures 9.20 and 9.21 illustrate the relationship between the gradation exponent and VMA for values of 'n' ranging from 0.3 to 0.7. The effect of compaction level is also assessed with Figure 9.20 showing the relationship for specimens compacted to refusal, and Figure 9.21 dealing with level 2 specimens.

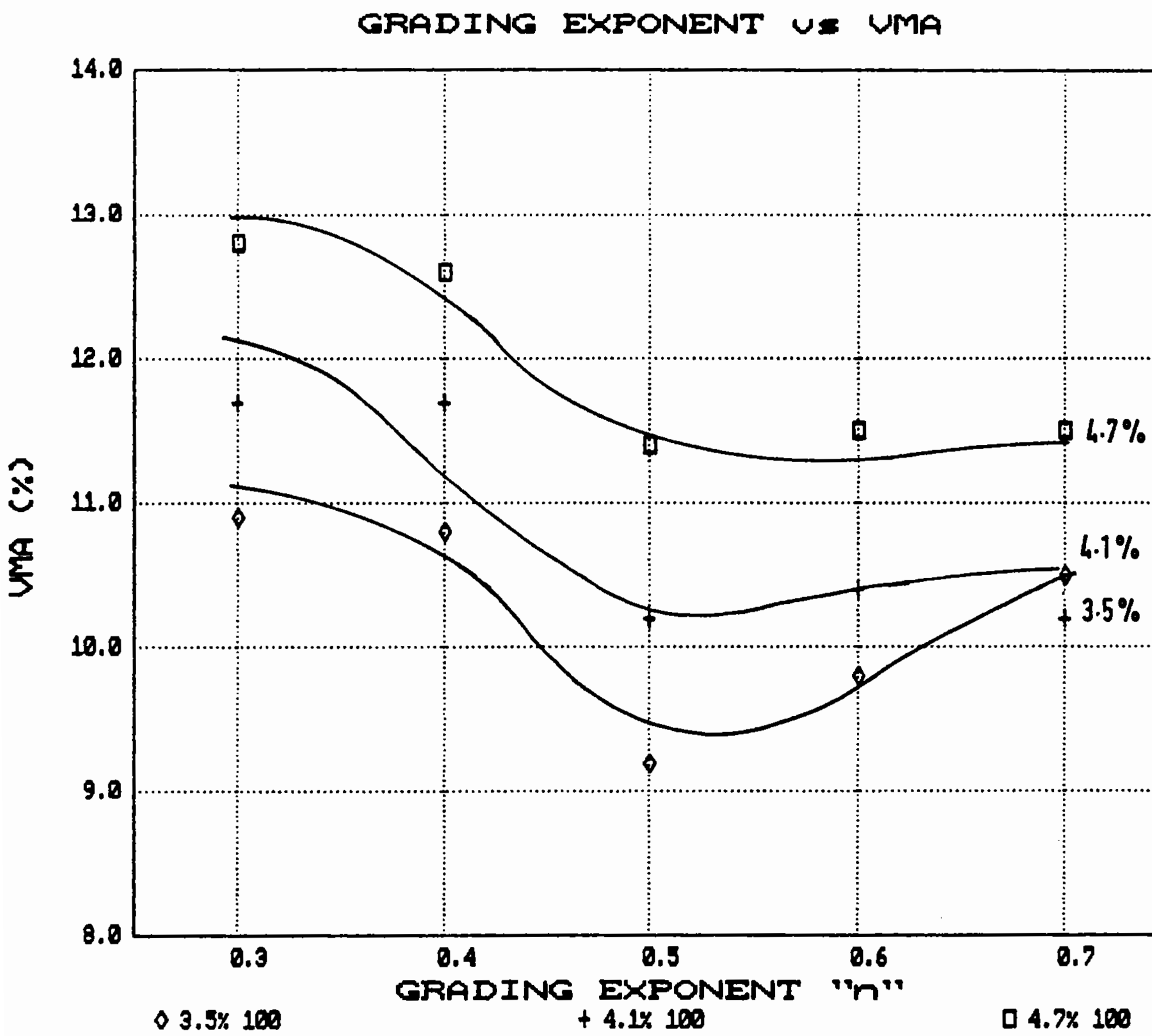

Figure 9.20 Influence of grading exponent on VMA at refusal density.

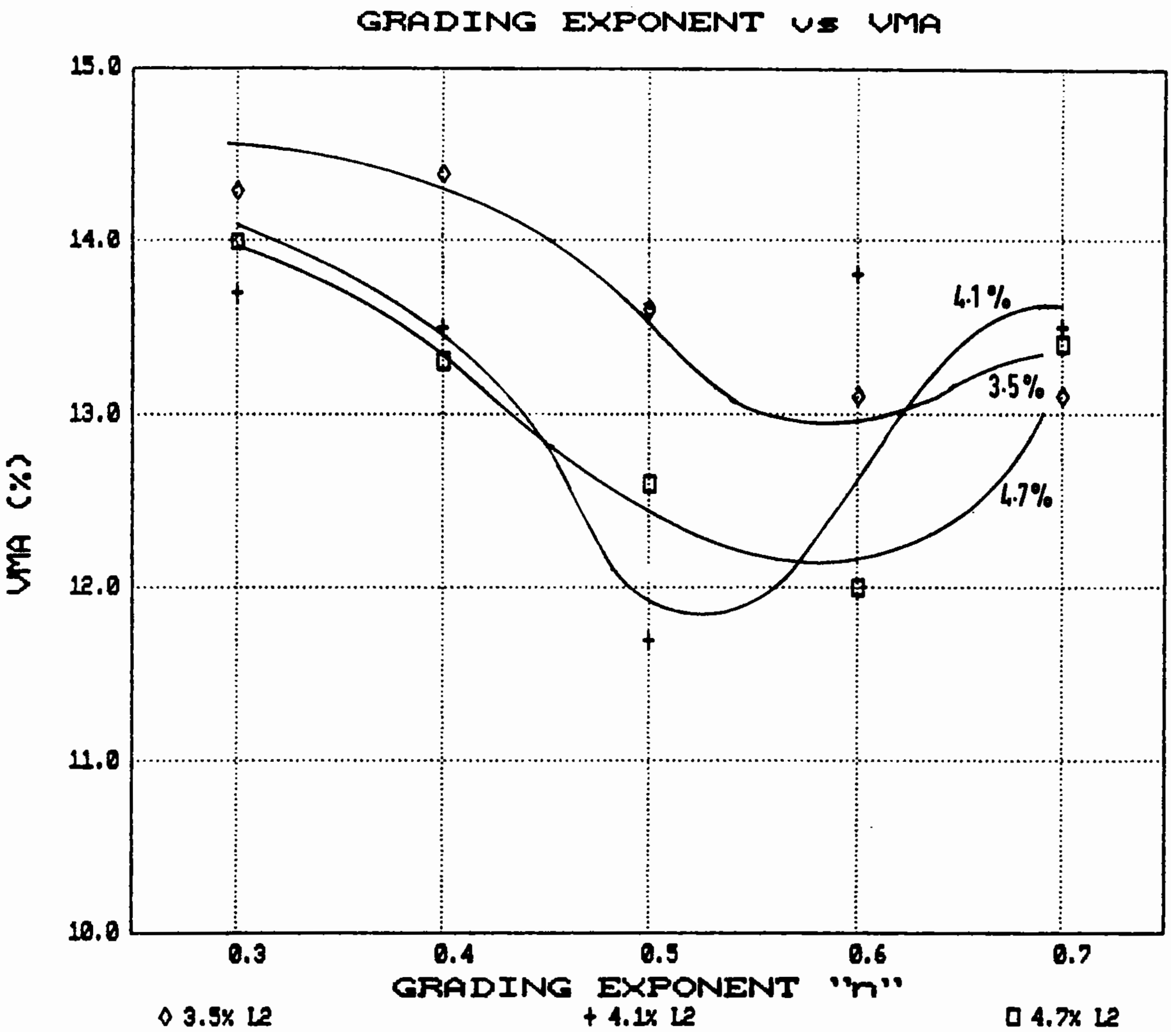

Figure 9.21 Influence of grading exponent on VMA at compaction level 2.

Two important observations may be taken from Figure 9.20. Firstly, a minimum VMA can be seen to occur at the n = 0.5 gradation for each binder content, indicating that adjusting the grading exponent to values below n = 0,5 serves to open up the aggregate matrix. Secondly, the introduction of more binder into the mixtures has the effect of increasing the VMA at the level of compaction. This can be attributed to the aggregate properties and is a direct result of the packing characteristics. The limestone aggregate has been shown to pack to a very low void content at the datum formulation and changes to the gradation do not significantly increase the void structure. Binder contents of 4.1% and 4.7% begin to dilate the stone matrix as the void content approaches 0% at refusal density.

The relationship between the gradation exponent and VMA for specimens compacted at level 2 is less well defined in Figure 9.21 as the results are taken from specimens with a range of PRDs but a minimum VMA can be seen to occur typically between the n = 0.5 and n = 0.6 exponents. At this compaction level, an increase in binder content generally corresponds with a variation in VMA for a fixed gradation, with the most pronounced changes occurring when the grading exponent is at the n = 0.5 level.

At both compaction levels, the void structure of the matrix is greater on the fine side of the maximum density curve than for gradations corresponding to the exponents of n = 0.6 and n = 0.7. It is likely that this effect results from a combination of an increased fine aggregate content dilating the stone structure, and a greater surface area of aggregate within the mix causing a greater absorption of binder. Hence, not only is the void content of the mixture increased because of a change in gradation but there is a reduced amount of lubrication from a given quantity of binder, impeding the relative compactibility of the material.

9.5.1 Mechanical properties

RLIT tests on the specimens manufactured at the n = 0.3 and n = 0.4 gradations, Table 9.11, show that these mixtures have an improved elastic stiffness to the corresponding specimens tested in the full mix design procedure (Tables 9.4 to 9.6). The elastic stiffness of a bituminous material is considered to be influenced by the volumetric proportions of the mixture and the binder stiffness, as well as material temperature and applied loading time. The volumetric compositions of the n = 0.3 and n = 0.4 mixtures do not change significantly from the coarser graded materials, but the stiffness of the binder mortar is likely to show improved qualities. Particularly fine aggregate passing the 300 micron sieve, probably contributes to the properties of the binder to a greater extent than to the properties of the gradation. Filler material, passing the 75 micron sieve almost certainly combines with the binder to produce a mortar which serves to cement the aggregate grading together. Therefore, a mix formulation comprising a high percentage of fine aggregate would probably exhibit different mechanical properties to a corresponding mixture with a reduced quantity of fines.

The introduction of more fine material into an asphaltic mixture, amalgamates with the binder to produce a stiff mortar which effects the elastic stiffness of the mix. It is this phenomenon which explains the general reduction in stiffness values as the grading exponent is moved from n = 0.3 to n = 0.7.

Although the elastic stiffness increases with the 'fine' mixtures, the aggregate matrix has been adjusted which may compromise the deformation resistance characteristics of the materials. Figures 9.22 and 9.23 illustrate the development of permanent strain in the RLA test for the specimens fabricated at the two 'fine' gradings. A comparison of these results with Figures 9.7 to 9.10 demonstrates that the use of aggregate gradings on the fine side of the Fuller curve result in mixtures of inferior resistance to permanent deformation to mix formulations based on a gradation exponent corresponding to n = 0.5 and n = 0.6. However, the mixtures with the high binder contents of 4.7% exhibit improved qualities against their coarser counterparts, but structural integrity of these specimens is lost when refusal density is approached.

9.5.2 Conclusions

The brief investigation into fine gradations in bituminous mixtures confirmed the mix design philosophy which has been followed in this project. Although there is an increase in the elastic stiffness of materials with a fine grading, there is a reduction in deformation resistance when compared with coarser materials manufactured at low binder contents. To fully coat and lubricate the aggregate at n = 0.3 and n = 0.4 gradations, necessitates a higher binder content than the n = 0.5, 0.6 gradings, which immediately creates concern over the deformation resistance qualities. This factor led to the problems analysed in the situation encountered in the Middle East, where severe rutting problems arose through an over-densification of a binder rich material.

The results underline the effect of small changes to the aggregate grading in terms of the material properties and the importance of understanding the binder-aggregate interaction.

9.6 INCREASING THE FILLER CONTENT

The isolation of individual aggregate gradations requires a specification on the quantity of material passing the 75 micron sieve. The value which has been advocated as part of this design procedure is 6%, which represents a typical amount of filler present in dense roadbase mixtures. Investigating the properties of materials manufactured with increased levels of fine material exhibited a higher elastic stiffness under the test conditions than the coarser graded materials but a reduced resistance to permanent deformation. By adjusting the lower boundary in the grading equation, the aggregate gradation may be altered to incorporate a high filler content, whilst maintaining the coarse aggregate structure. This approach should lead to mix formulation with optimum resistance to deformation and improved elastic stiffness. This in turn should improve the fatigue life of such a mixture through the reduction in the magnitude of tensile strains at the underside of the material.

A series of mixtures using granite aggregate with a filler content of 10%, was manufactured at the three grading exponents and three binder contents. These mixtures are similar to Heavy Duty Macadam (2) mixes which are commonly placed during new construction and re-construction works on principal highways in the UK. Volumetric proportions, and elastic stiffness values of the specimens are shown in Tables 9.12 to 9.14.

Table 9.12 Volumetric proportions and elastic stiffness values of specimens manufactured using granite aggregate, F = 10%, n = 0.5.

Mix formulation	Density (g/ml)	V_v (%)	V_B (%)	VMA (%)	PRD (%)	Elastic Stiffness (MPa)
0.5 3.5 1	2.541	4.0	8.7	12.7	100	8400
0.5 3.5 2	2.402	9.3	8.2	17.5	94.5	3000
0.5 4.1 1	2.526	3.6	10.2	13.8	100	4900
0.5 4.1 2	2.437	7.0	9.8	16.8	96.4	4400
0.5 4.7 1	2.529	2.6	11.7	14.2	100	5900
0.5 4.7 2	2.448	5.7	11.3	17.0	96.7	3500

Table 9.13 Volumetric proportions and elastic stiffness values of specimens manufactured using granite aggregate, F = 10%, n = 0.6.

Mix formulation	Density (g/ml)	V v (%)	V_B (%)	VMA (%)	PRD (%)	Elastic Stiffness (MPa)
0.6 3.5 1	2.522	4.7	8.7	13.4	100	8500
0.6 3.5 2	2.438	7.9	8.4	16.3	96.6	4400
0.6 4.1 1	2.510	4.3	10.1	14.4	100	5800
0.6 4.1 2	2.417	7.8	9.7	17.5	96.3	3400
0.6 4.7 1	2.519	3.0	11.6	14.6	100	4000
0.6 4.7 2	2.448	5.7	11.3	17.0	97.2	4800

Table 9.14 Volumetric proportions and elastic stiffness values of specimens manufactured using granite aggregate, f = 10% n = 0.7.

Mix formulation	Density (g/ml)	V v (%)	V_B (%)	VMA (%)	PRD (%)	Elastic Stiffness (MPa)
0.7 3.5 1	2.493	5.8	8.6	14.4	100	7000
0.7 3.5 2	2.422	8.5	8.3	16.8	97.1	2600
0.7 4.1 1	2.494	4.9	10.0	14.9	100	3200
0.7 4.1 2	2.306	8.6	9.6	18.2	96.0	2500

The most significant aspect concerning the volumetric proportions is the increase in the void contents and VMAs to corresponding specimens manufactured using a filler content of 6% (Table 9.10). This effect was noted and discussed with the fine gradations of n = 0.3 and n = 0.4, and is considered to be the result of the creation of a stiff binder/fines mortar which impedes compaction for a given mixture. This effect appears to be independent of binder viscosity which dictates mixing and compaction temperatures. It can be seen from the above Tables that for a given compactive effort, the void content of the mixture decreases as the binder content increases, suggesting that the lubrication qualities of the binder are moderated as the quantity of very fine aggregate increases.

The investigation has demonstrated that an adjustment to the filler content of the mixture does influence the mix stiffness, with a higher filler content corresponding to an improved elastic stiffness, particularly at high levels of compaction. The deformation resistance properties do not appear to exhibit an overall improvement through the addition of filler material, Figures 9.24 to 9.26, although the use of a higher binder content ceases to show failure characteristics. Optimum resistance to deformation has been achieved with the n = 0.5 and n = 0.6 gradations at low binder contents.

The results of the mechanical tests performed on the mixtures with high filler contents have indicated that the elastic stiffness of such mixtures can be improved, particularly at high levels of compaction, and that good resistance to permanent deformation can be maintained at higher binder contents. However, a decreasing binder-filler ratio also appears to have the effect of reducing the compactibility of the mixture, which leads to a practical issue and explains why some UK highway authorities refuse to include heavy duty macadams within their material specifications, claiming that satisfactory levels of compaction cannot be achieved. However, modern, sophisticated,

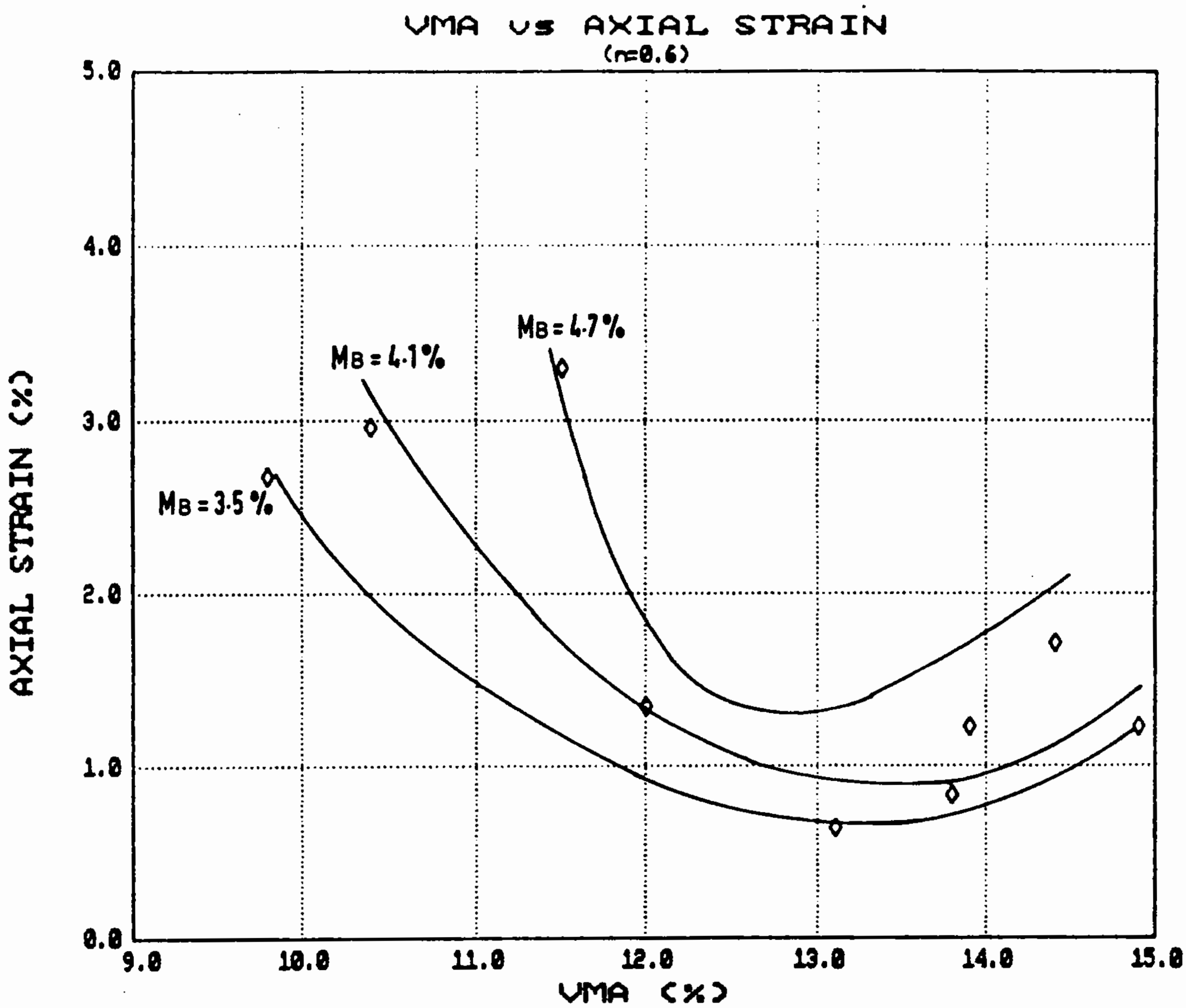

Figure 9.15 Influence of compaction on the resistance to permanent deformation. Limestone aggregate, n=0.6.

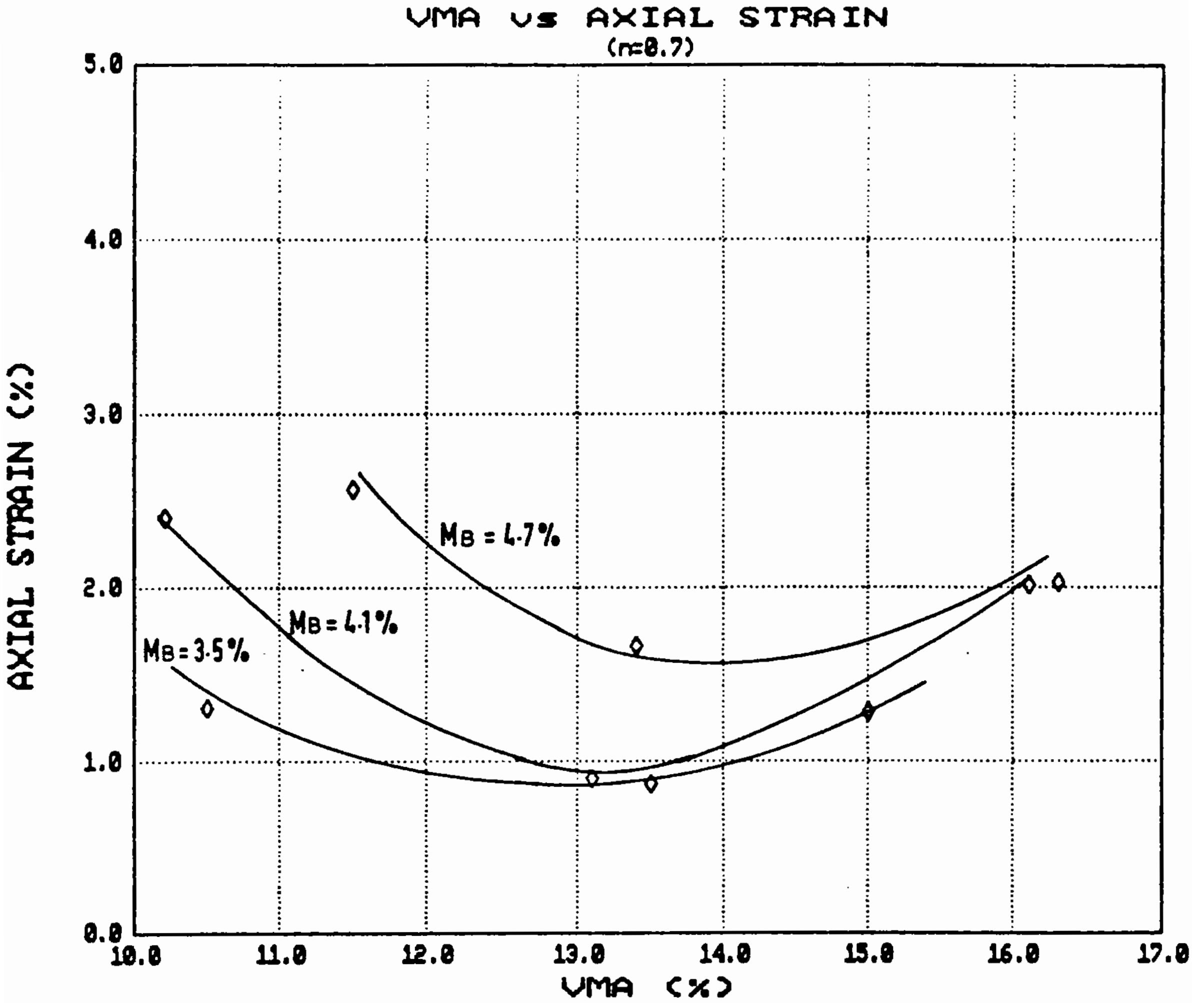

Figure 9.16 Influence of compaction on the resistance to permanent deformation. Limestone aggregate, n=0.7.

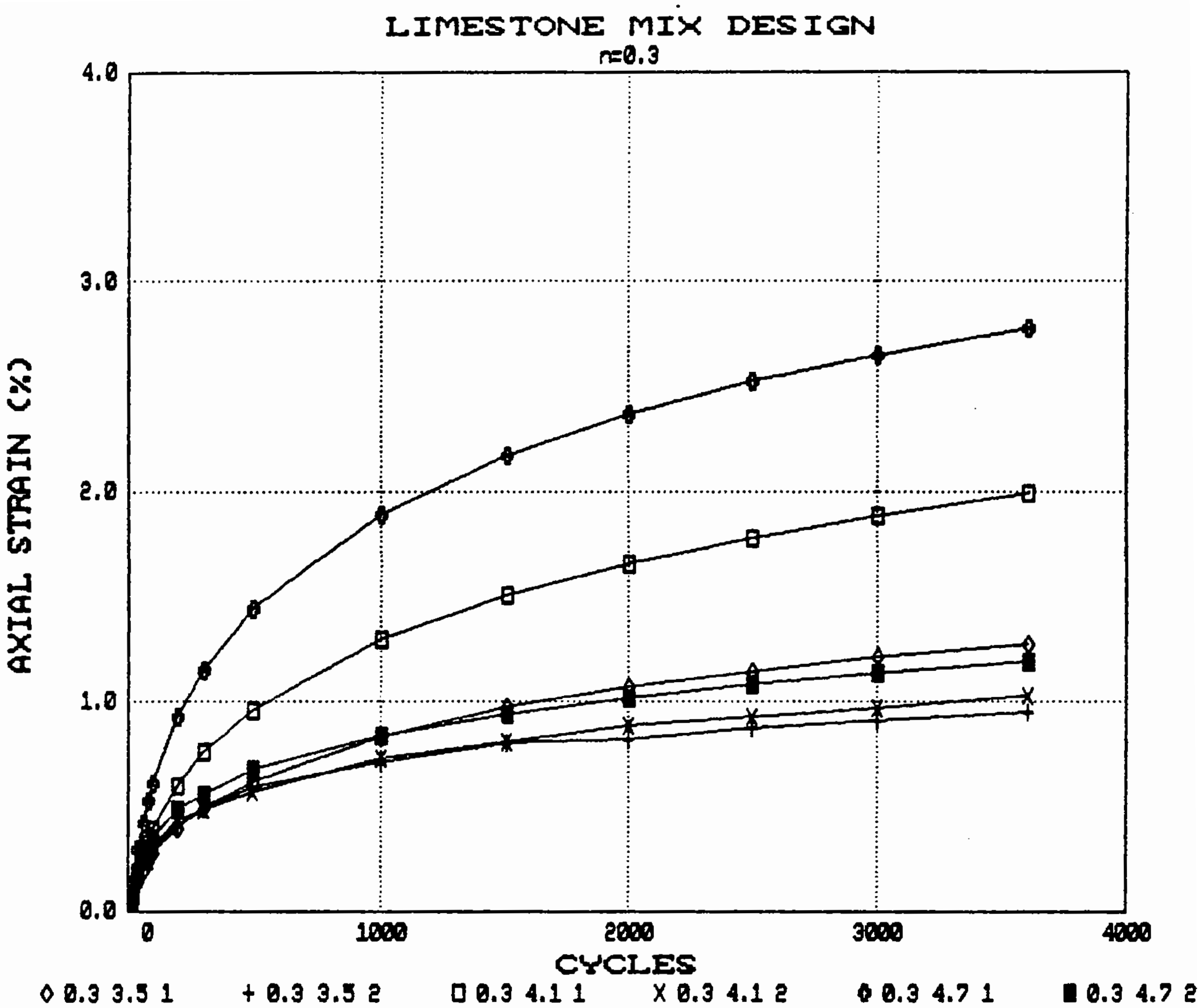

Figure 9.22 Deformation curves plotted from the results of RLA tests. Limestone aggregate, grading exponent n=0.3.

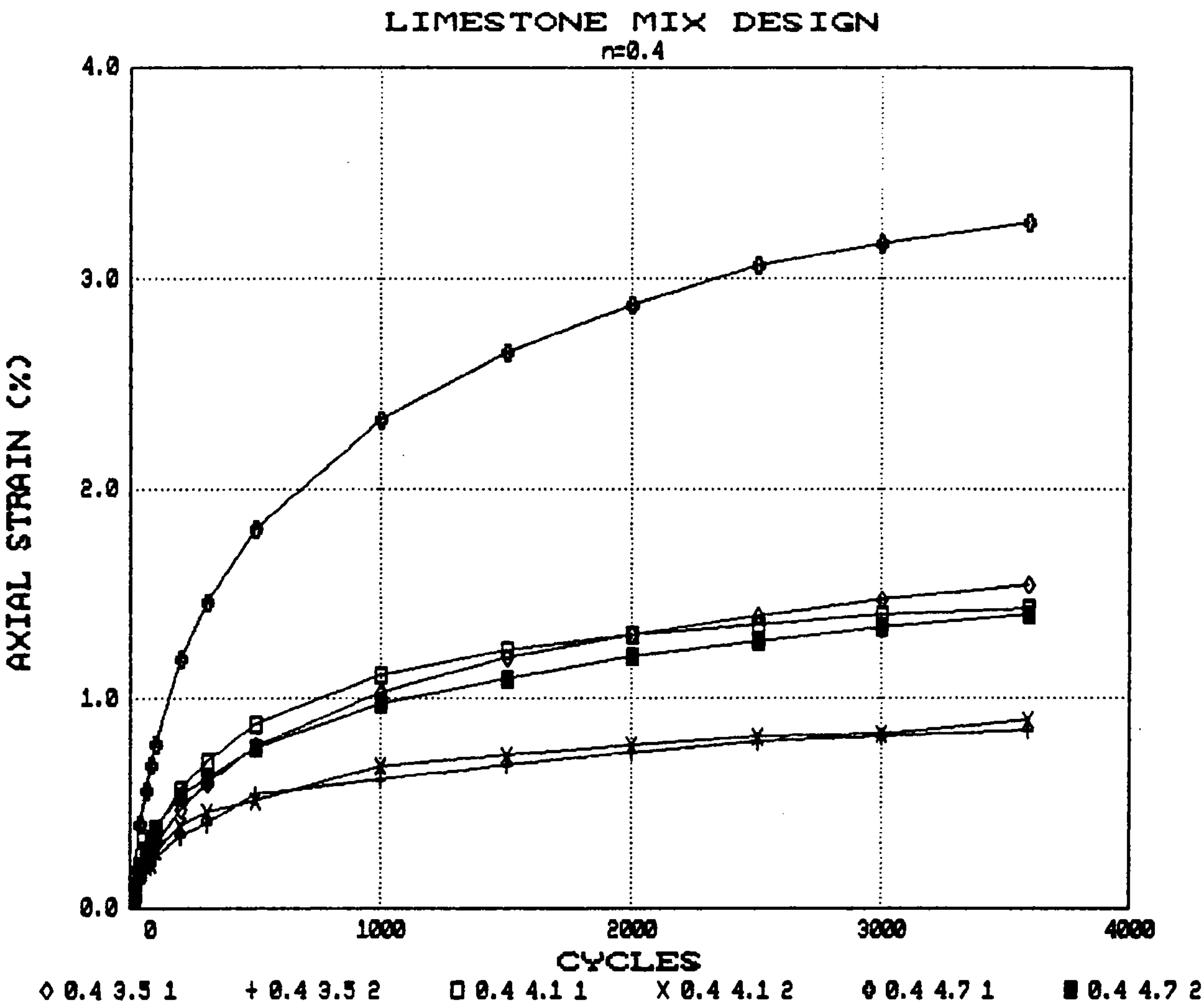

Figure 9.23 Deformation curves plotted from the results of RLA tests. Limestone aggregate, grading exponent, n=0.4.

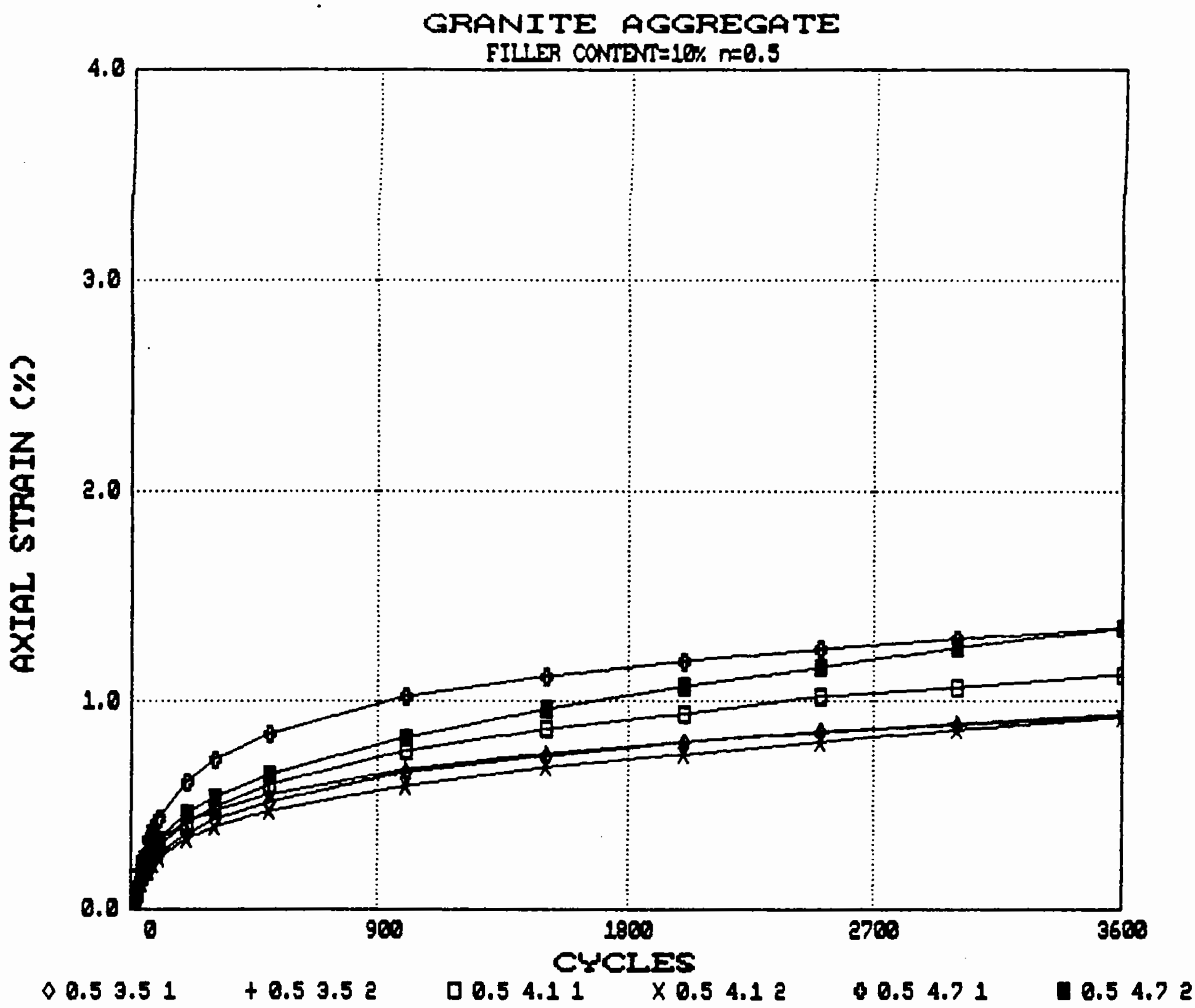

Figure 9.24 Deformation curves plotted from the results of RLA tests. Granite aggregate, grading exponent, n=0.5, filler content, F=10%.

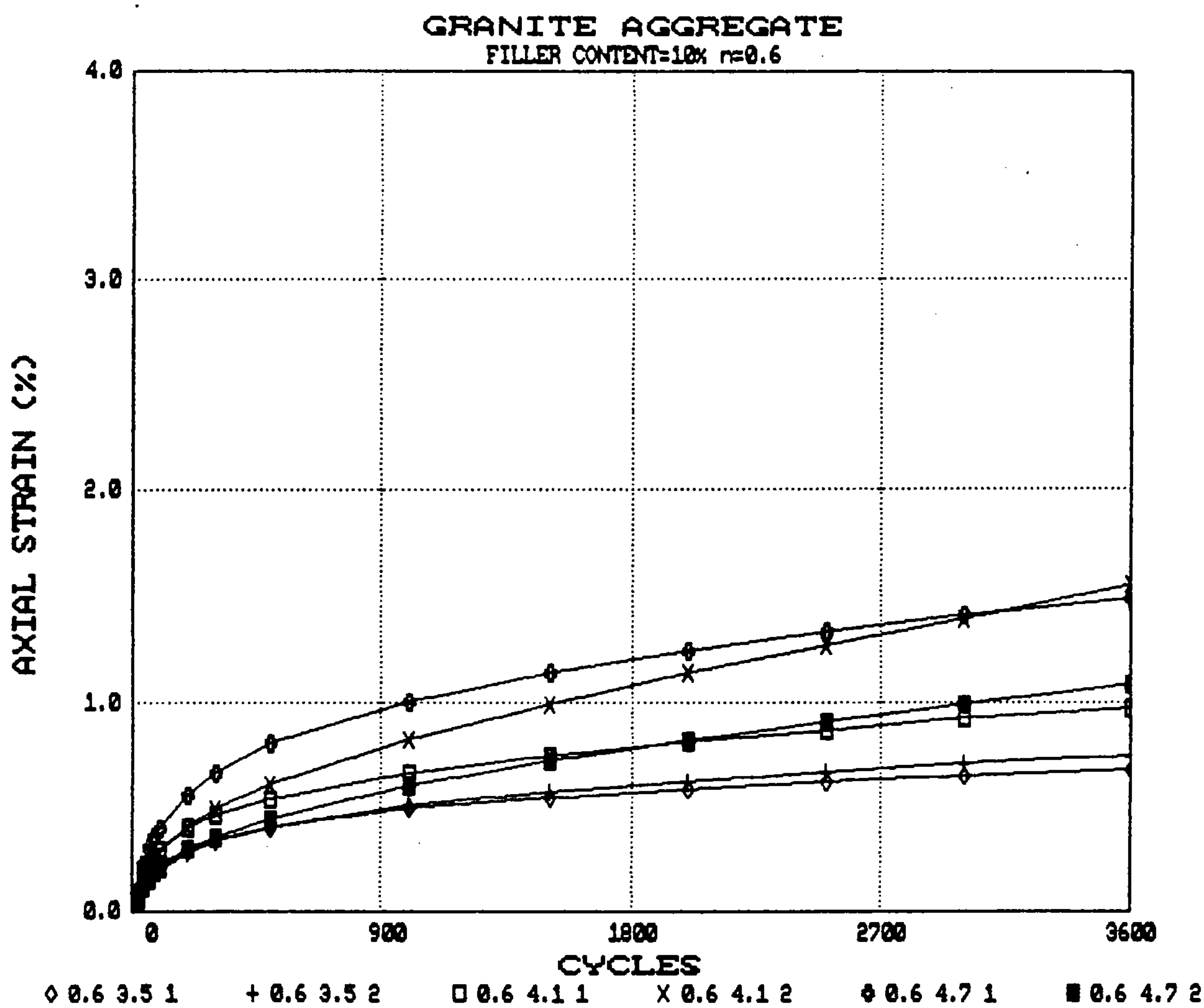

Figure 9.25 Deformation curves plotted from the results of RLA tests. Granite aggregate, grading exponent, n=0.6, filler content, F=10%.

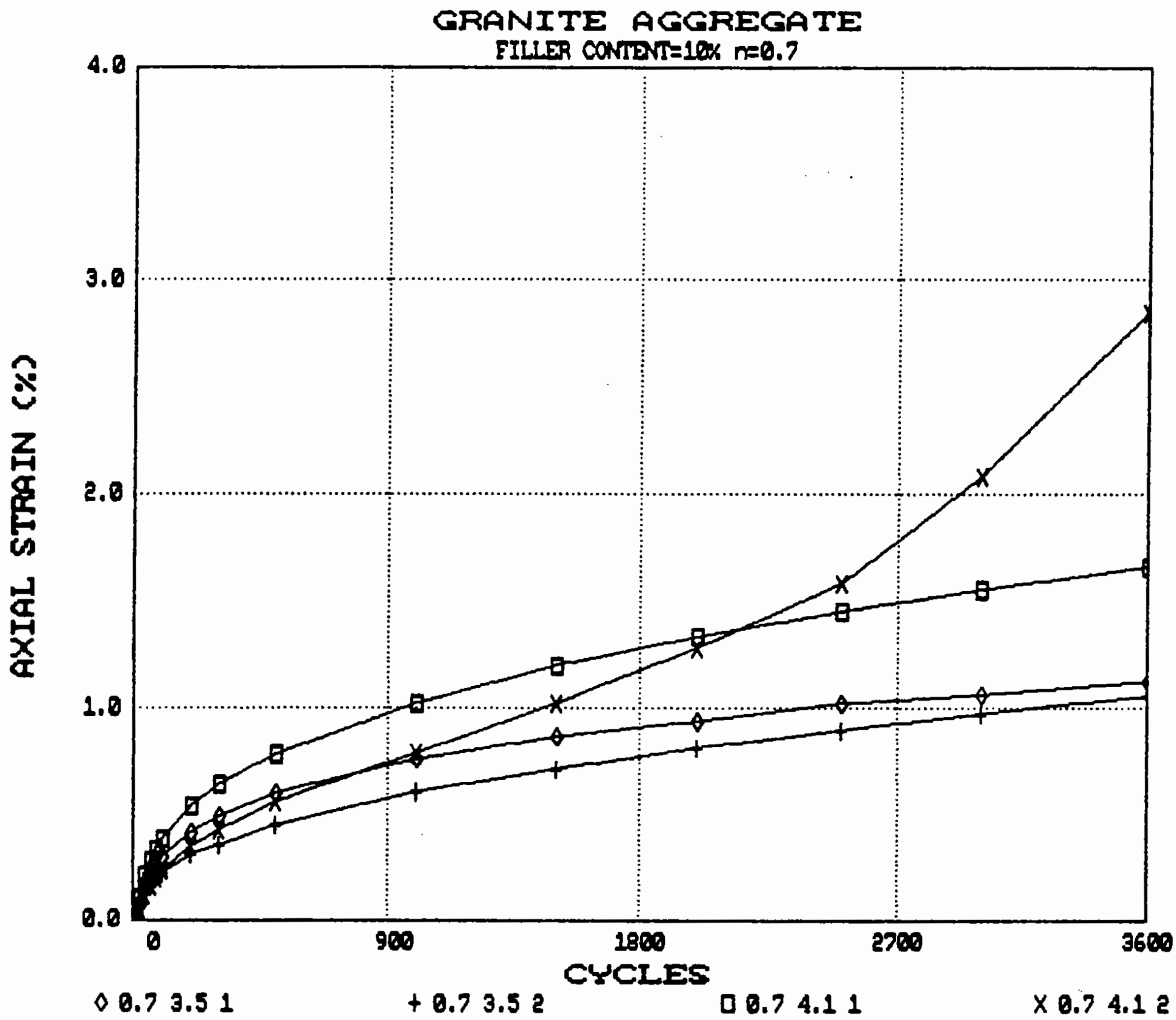

Figure 9.26 Deformation curves plotted from the results of RLA tests. Granite aggregate, grading exponent, n=0.7, filler content, F=10%.

vibratory compaction plant has shown this not to be the case but it provides further evidence that the level of compaction of bituminous materials is a variable parameter and should be specified in accordance with the type of material placed.

9.7 VERIFICATION OF MANUFACTURING PROCEDURE

The mix design procedure is capable of evaluating different mix formulations in terms of their mechanical properties and ranking the mixtures in order of preference. The acceptance criteria which have been applied to the properties of elastic stiffness and resistance to permanent deformation have been based on testing of laboratory manufactured specimens and engineering judgement.

In order to validate the specimen manufacturing procedure, a comparison must be made between the mechanical properties of laboratory specimens and site cores of the same material. To make a genuine comparison, the same generic mixture must be used in the laboratory to that placed on site. Such an investigation requires the co-operation of a local authority and a material supplier to coordinate the sampling of material at the mixing plant and the later acquisition of cores from the same batch of material which has been compacted on site. Cooperation for this exercise was achieved with Northamptonshire County Council and Mountsorrell Quarry in Leicestershire, who were involved in the laying of a 40mm Heavy Duty Macadam roadbase as part of a by-pass scheme. Material was sampled from the quarry and transported to the laboratory at Nottingham University in 'hot boxes', to minimise the cooling of the mixture. At the laboratory, the material was weighed out to provide six PRD specimens, two of which were placed in an oven at 160°C, in preparation for compaction to refusal, and the remaining four allowed to cool to 100°C for compaction at level 2.

On site, after the HDM had been laid and compacted, Northamptonshire County Council took a series of cores from the mat to check that the material complied with specification. Extra cores were taken at the request of Nottingham University, which were delivered to the laboratory at a later date. The material on site was compacted to a layer thickness of approximately 100mm, the cores from which were trimmed to a length of 70mm for testing purposes.

The exercise facilitated a comparison of the measured mechanical properties of specimens of the same generic material, some of which had been obtained from site, and the remainder manufactured using a simulative laboratory technique. Volumetric proportions were measured using gravimetric means, with elastic stiffness values and deformation resistance characteristics obtained from NAT testing. Results are displayed in Tables 9.15 and 9.16 and Figures 9.27 and 9.28.

Table 9.15 Volumetric proportions and elastic stiffness values of cores obtained from site.

Specimen No	Density (g/ml)	Vv (%)	V_B (%)	VMA (%)	PRD (%)	Elastic Stiffness (MPa)
1	2.451	2.6	9.5	12.1	98.7	7000
2	2.444	2.8	9.5	12.3	98.4	8000
3	2.448	2.7	9.5	12.2	98.5	6700
4	2.444	2.8	9.5	12.3	98.4	6000
5	2.449	2.6	9.5	12.1	98.6	5400
6	2.463	2.1	9.6	11.6	99.2	7200

Table 9.16 Volumetric proportions and elastic stiffness values of PRD manufactured specimens.

Specimen No	Density (g/ml)	Vv (%)	V_B (%)	VMA (%)	PRD (%)	Elastic Stiffness (MPa)
100A	2.483	1.3	9.7	11.0	100	10200
100B	2.484	1.3	9.7	11.0	100	11000
PRD1	2.351	6.5	9.1	15.6	94.7	9000
PRD2	2.337	7.1	9.1	16.2	94.1	9500
PRD3	2.366	6.0	9.2	15.2	95.2	10000
PRD4	2.379	5.4	9.3	14.7	95.8	10000

The laboratory manufactured material successfully produced two specimens at refusal density, and four at approximately 95PRD. The material from site

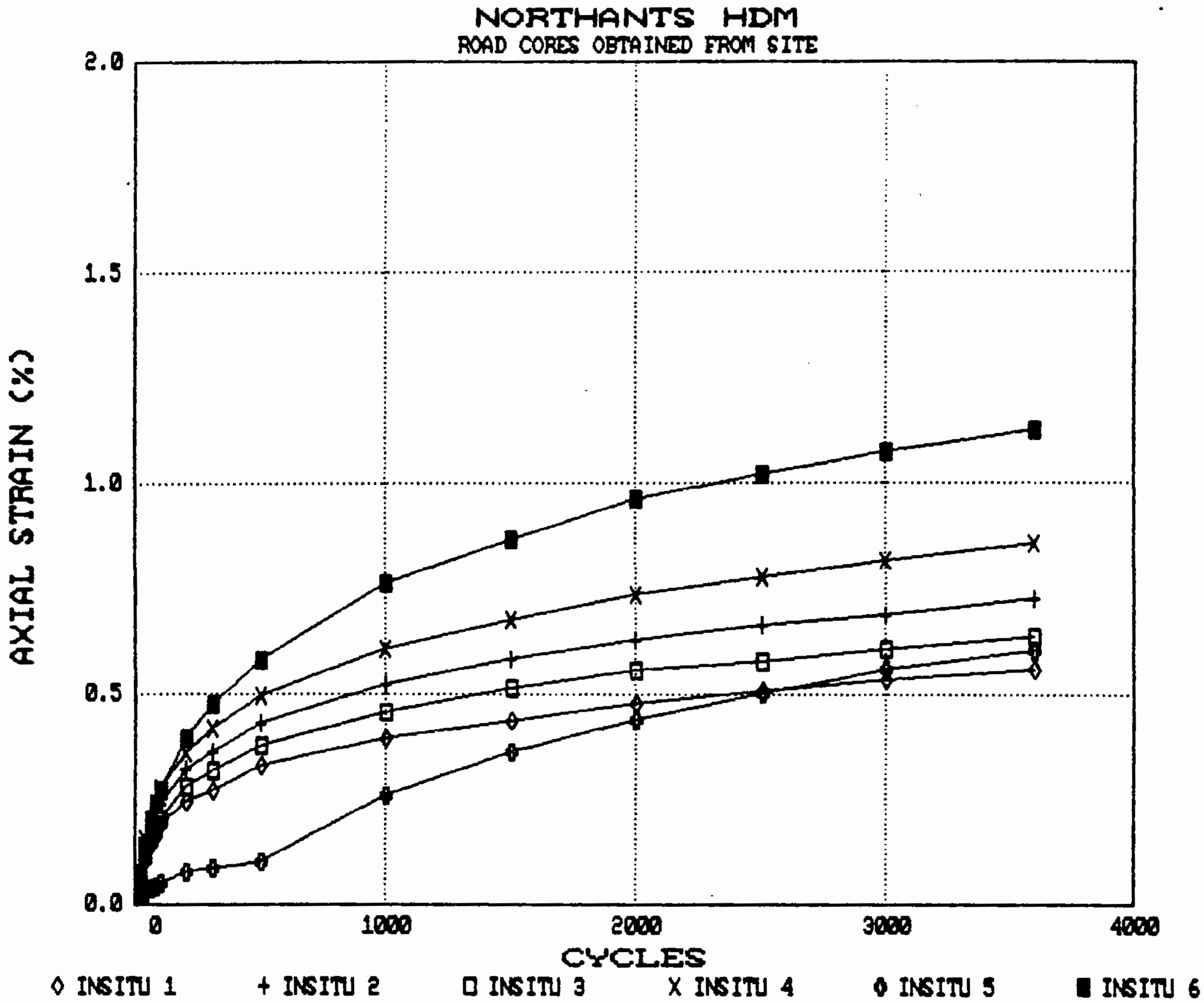

Figure 9.27 Deformation curves plotted from the results of RLA tests. HDM road cores taken from site.

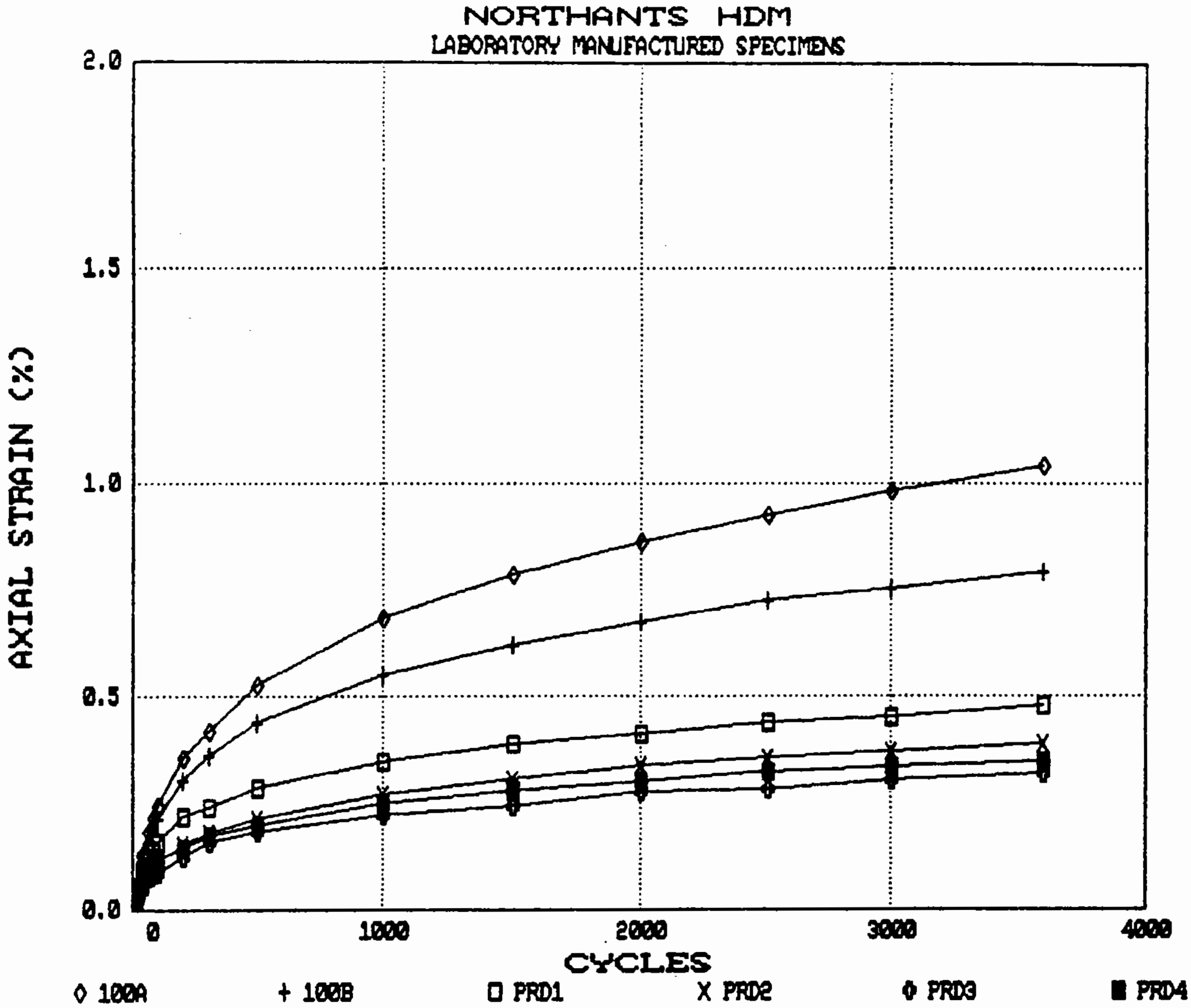

Figure 9.28 Deformation curves plotted from the results of RLA tests. HDM specimens manufactured in the laboratory.

proved to have very uniform volumetric proportions, but a high level of compaction, with all specimens in excess of 98PRD. Considering the uniformity of VMAs of the site material, there is quite a range in the elastic stiffnesses with values extending from 5.4GPa to 8GPa, with the laboratory specimens exhibiting values between 9GPa and 11GPa. Given the high level of compaction of the site material, the elastic stiffness results appear somewhat paradoxical with the laboratory PRD specimens having greater VMAs and higher elastic stiffness. A possible explanation of these results is the randomness of site coring and the variability of gradation within an individual core. Although this should not significantly effect the volumetric proportions, there could be a substantial influence on the mechanical properties, as indicated through the test results. It would be reasonable to assume however, that the range of values obtained, would encompass the typical elastic stiffness of the layer, demonstrating that the laboratory manufactured specimens exhibit elevated values of elastic stiffness.

An examination of the RLA test results in Figures 9.27 and 9.28 demonstrate that both sets of specimens have shown a good resistance to permanent deformation, with only two out of twelve test specimens exhibiting a strain exceeding 1% at the termination of the test. The results of the laboratory manufactured specimens shown in Figure 9.27,illustrate that the material compacted to refusal density, (specimens 100A and 100B) shows considerably higher strains than specimens PRD 1 to 4, which have an average compaction level of 95PRD. These results are similar to those showing the effects of over compaction in the limestone and granite mix designs when flushing of the binder has manifested at very low void contents. The general format of the

RLA test results of the road cores is similar to the laboratory specimens with curves typically lying between the 95PRD and 100PRD plots of Figure 9.28. From a relative comparison of levels of compaction between the two sets of specimens, the deformation characteristics of the laboratory specimens do not appear dissimilar to those obtained from the road cores. Specimen 'in situ 6' which has a PRD of 99.2% exhibits a very similar curve to that obtained from, 100A, a laboratory specimen manufactured at 100PRD.

One of the most significant observations which can be made from the two sets of results is the improved resistance to permanent deformation exhibited by a HDM type mix compared with conventional DBM mixtures assessed in the design procedure. The aggregate from Mountsorrell quarry is classified for engineering purposes as a granite, and could be expected to exhibit similar characteristics in a bituminous mixture to those obtained from a mix comprising Bardon Hill material, although this assumption depends upon the congruity of the aggregate packing characteristics. Comparing Figure 9.27 with Figure 9.18 illustrates that the HDM mixture has a slightly greater resistance to deformation than the n = 0.5, $M_B = 3.5\%$, Level 2 specimen from the granite mix design, and the granite specimens manufactured with a filler content of 10% (Figures 9.24 to 9.26). Elastic stiffness values are also improved, although the use of 50 pen binder in HDM mixes is the major contribution to this effect. Figure 9.28 also illustrates that materials with a higher filler content can be compacted to very high states using modern equipment and emphasises the importance of compacting material to the optimum level and not the highest level.

9.7.1 CONCLUSIONS

The investigation carried out with the kind cooperation of Northamptonshire County Council and Mountsorrell quarry suggests that the use of the PRD apparatus as a tool for manufacturing specimens of bituminous materials does produce specimens which are representative of site material. The similarity in the shape of the curves obtained from RLA testing for the laboratory and site compacted material indicates that the general deformation behaviour of field compacted material is reproduced by the compaction method used in this investigation. It may be prudent to consider that the confining effects of the mould may give rise to an elevated elastic stiffness, although further testing would be required to corroborate this supposition.

The exercise has also underlined one of the fundamental difficulties of trying to evaluate the properties of laboratory manufactured material against site material, which is the reproduction of the volumetric structure. Given the ideal case, the researcher would perform mechanical tests on specimens with identical gradings, binder contents and volumetric proportions, some of which were site material and some laboratory specimens. If the laboratory material gave repeatedly improved performance over the site material, then this could be quantified by the introduction of a factor. However, in practice this is not the case, and an engineering assessment must be employed when comparing results. In the context of a mix design procedure, an exact reproduction of the mechanical properties of site material for a given mix formulation are not necessary as long as different mixtures are quantified and ranked according to the fundamental properties which dictate pavement performance. The optimum formulation selected from the design procedure, should remain the optimum performer in the highway pavement, even if the mechanical properties do not equate exactly to the mixtures assessed at the design stage.

CHAPTER TEN

COMPARISON OF NEW PROCEDURE WITH EXISTING DESIGN METHODS

The involvement of Nottingham University in the Strategic Highway Research Program (SHRP) allowed a direct comparison of the new mix design method, developed under this project, with the ubiquitous Marshall procedure, using aggregate and binder from the SHRP library of materials. This proved a very useful departure from concentrating on UK practice, and provided an opportunity to evaluate the new design method against proven asphalt concrete designs.

10.1 MATERIALS

The materials which had been supplied to Nottingham University for inclusion in the SHRP investigation comprised a rough textured, angular granite from California, and a Venezuelan asphalt. The granite comprised five quarry bin sizes from a maximum size fraction of 28mm down to two dust components. The gradings within each bin size are shown in Table 10.1.

Table 10.1 Bin gradings of California Granite

Sieve Size		Nominal size of aggregate (mm)				
U.K. (mm)	U.S. (inch)	% Passing 37	% Passing 25	% Passing 9.5	% Passing 5	% Passing 5
37	$1\frac{1}{2}$	100	100	100	100	100
25	1	35	100	100	100	100
19	$\frac{3}{4}$	7	87	100	100	100
12.5	$\frac{1}{2}$	5	29	100	100	100
9.5	$\frac{3}{8}$	3	10	100	100	100
4.75	Nº 4	1	3	17	100	100
2.36	Nº 8	0	1	2	89	91
1.18	Nº 16	0	0	1	59	69
0.6	Nº 30	0	0	0.5	35	53
0.3	Nº 50	0	0	0	18	40
0.15	Nº 100	0	0	0	7	28
0.075	Nº 200	0	0	0	3.8	19

As with Dene Quarry aggregate, the use of two dust products helps maintain close adherence to a target gradation. For the purpose of the investigation performed using this material, the aggregate was sieved out into the individual sieve sizes shown in the left hand column of Table 10.1, allowing an exact reproduction of target gradations. The use of this particular aggregate in highway paving mixtures is known to be a good performer, as it generates a very strong mechanical interlock in the compacted matrix of material.

The asphalt is also considered to be a good binder for use in highway engineering, as it exhibits a relatively high penetration index and hence low temperature susceptibility. Therefore combination of the Californian aggregate and Venezuelan binder should provide a high quality mix, which easily satisfies the criteria applied to the mechanical properties in the mix design procedure. A summary of the material properties is given in Table 10.2.

Table 10.2 Basic properties of mixture constituents

Aggregate		Binder	
Source	California	Penetration (100g,5s,25°C)	70
Classification	Granite	R & B softening point (°C)	50
Effective S.G.	2.8	Penetration Index	- 0.37
		Mixing Temperature @ 1.5 poises (°C)	161

10.2 MARSHALL & HVEEM MIX DESIGNS

The paving materials incorporating the granite aggregate are controlled by California Transportation Department (Caltrans) specifications, who provide envelope specifications for aggregate gradations, and design asphalt contents by a modified Hveem method of mix design. The SHRP investigations were assessing two aggregate gradations, both of which meet the Caltrans specification, which can be classified as a coarse grading and a fine grading. Figure 10.1 illustrates the two grading curves and shows a comparison with the Nottingham design curves, where it can be seen that both the Caltrans gradations have a lower percentage of coarse aggregate than the three Nottingham curves.

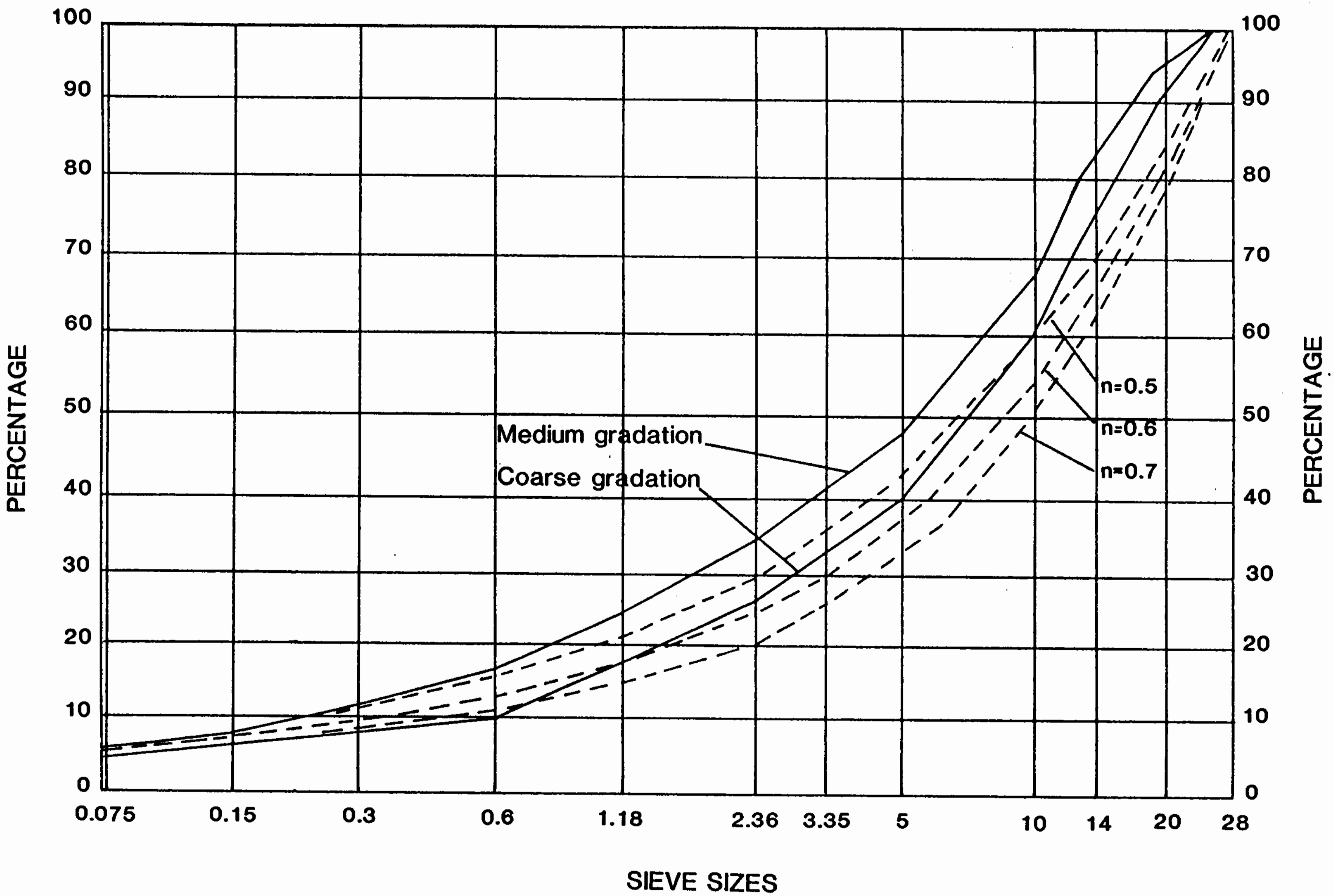

Figure 10.1 Comparison of the grading curves used in the Nottingham mix design, and the coarse and fine gradations of the SHRP.

Both gradations have been used in practice, and experience has shown the fine gradation to produce the greatest resistance to deformation. Inspection of the fine grading curve shows it to follow a similar gradation to a curve corresponding to an exponent of n = 0.4 in the gradation equation.

The SHRP investigation performed both Marshall and State of California (modified Hveem) mix design on the gradings to obtain design binder contents. Full details of the designs are given in Appendix 3, with summarized results shown in Table 10.3.

Table 10.3 Mix design results

Aggregate	Gradation	Asphalt Content (%)	
		Low	High
California	Medium	5.1	5.7
Granite	Coarse	4.9	5.5

Asphalt contents are by percentage weight of aggregate

The low and high binder contents are averaged values based on the Marshall and Hveem designs, which the SHRP study has introduced in order to investigate changes in material composition. It is immediately noticeable that almost all these values exceed the range of binder contents applied to the Nottingham method. The low binder content on the coarse grading, 4.9% by mass of aggregate, which equates to 4.7% by total mass of mix, corresponds with the highest binder content assessed in the Nottingham approach. This observation draws attention to the philosophy of design methods such as Marshall, where the procedures aim to produce a mixture with a maximised bitumen content. The potential shortcomings of this approach have been evaluated by investigations such as the trial documented in Chapter 7, and are generally accepted by highway engineers, and yet the Marshall and Hveem approaches still stand as the unequivocal methods of designing bituminous concrete.

10.3 THE NOTTINGHAM MIX DESIGN

Twenty seven specimens were manufactured according to the procedural format outlined in Chapter 8. Densities of the specimens were obtained by gravimetric means without using any sealing procedures and volumetric proportions then calculated from knowledge of properties of the mix constituents. The volumetric composition of the $n = 0.5$, $M_B = 3.5\%$, 100 PRD specimen gave the datum for specifying the target range of volumetric proportions for mixtures using the Californian granite. The calculated values are shown in Tables 10.4 to 10.6.

Table 10.4 Volumetric proportions of specimens (n = 0.5)

Spec No	Mix Formulation	Density (g/ml)	V_v (%)	V_B (%)	VMA (%)	PRD %
1	0.5,3.5,1	2.550	3.4	8.7	12.1	100
2	0.5,3.5,2	2,455	6.9	8.4	15.3	96.3
3	0.5,3.5,3	2.420	8.3	8.3	16.6	94.9
4	0.5,4.1,1	2.551	2.4	10.3	12.7	100.0
5	0.5,4.1,2	2.426	7.2	9.8	17.0	95.1
6	0.5,4.1,3	2.429	7.0	9.8	16.8	95.2
7	0.5,4.7,1	2.548	1.6	11.7	13.3	100.0
8	0.5,4.7,2	2.438	5.8	11.2	17.0	95.7
9	0.5,4.7,3	2.422	6.4	11.2	17.6	95.1

Table 10.5 Volumetric proportions of specimens (n = 0.6)

Spec No	Mix Formulation	Density (g/ml)	V_v (%)	V_B (%)	VMA (%)	PRD %
10	0.6,3.5,1	2.532	4,0	8.7	12.7	100
11	0.6,3.5,2	2,449	7.2	8.4	15.6	96.7
12	0.6,3.5,3	2.442	7.5	8.4	15.8	96.4
13	0.6,4.1,1	2.556	2.2	10.3	12.5	100
14	0.6,4.1,2	2.425	7.2	9.7	16.9	94.9
15	0.6,4.1,3	2.421	7.4	9.7	17.1	94.7
16	0.6,4.7,1	2.536	2.0	11.7	13.7	100
17	0.6,4.7,2	2.448	5.4	11.3	16.7	96.5
18	0.6,4.7,3	2.422	6.4	11.2	17.6	95.5

Table 10.6 Volumetric proportions of specimens (n = 0.7)

Spec No	Mix Formulation	Density (g/ml)	V_V (%)	V_B (%)	VMA (%)	PRD %
19	0.7,3.5,1	2.510	4.9	8.6	13.5	100
20	0.7,3.5,2	2,431	7.9	8.3	16.2	96.9
21	0.7,3.5,3	2.437	7.7	8.4	16.1	97.1
22	0.7,4.1,1	2.534	3.0	10.2	13.2	100
23	0.7,4.1,2	2.422	7.3	9.7	17.0	95.6
24	0.7,4.1,3	2.384	8.8	9.6	18.4	94.1
25	0.7,4.7,1	2.534	2.1	11.7	13.8	100
26	0.7,4.7,2	2.444	5.6	11.3	16.9	96.4
27	0.7,4.7,3	2.399	7.3	11.1	18.4	94.7

For the Californian granite the datum specimen exhibits a void content of 3.4%. This provides the most conclusive evidence that some method of quantifying aggregate packing characteristics is required prior to applying criteria to the volumetric proportions of mixtures comprising the aforesaid aggregate. Adherence to the Asphalt Institute specification on voids of 3% to 5% would require a severe compactive effort and a high binder content, such as specimens 4, 6, 13 and 16, which have been shown to be conducive to deformation in mixtures using other aggregate types. The target values of volumetric proportions for mixtures comprising the Californian granite, therefore, are recommended in Figure 8.11, with a preferential void content lying in the range of 4% to 8% and VMA between 14% and 18%. The volumetric proportions of the 27 specimens are shown plotted on the appropriate chart in Figures 10.2 to 10.4. All specimens compacted to 100 PRD fail to meet the criteria as do specimens 3, 24 and 27. An observation which may be drawn from the plots, is the similarity in many cases of the volumetric composition

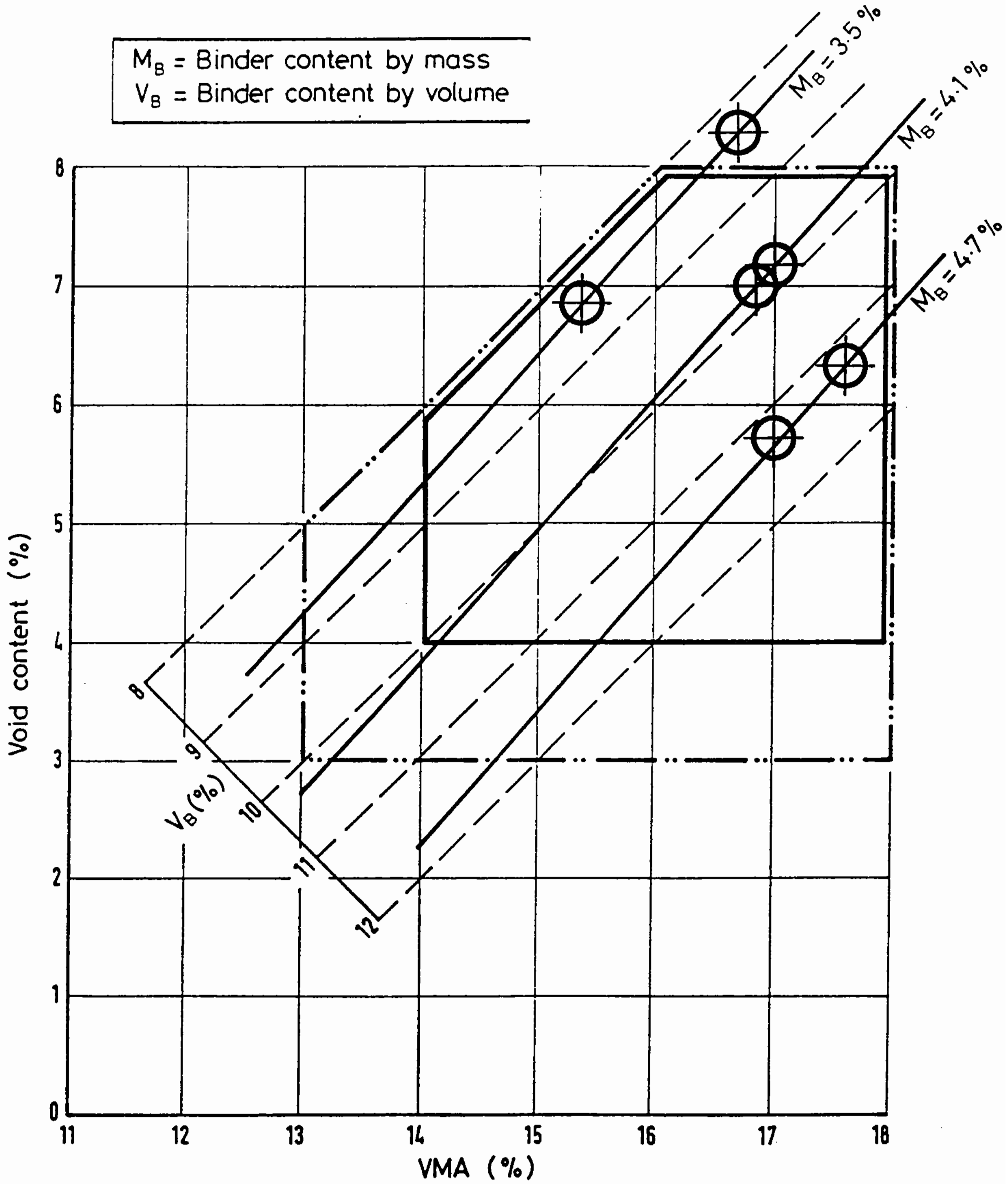

Figure 10.2 Volumetric proportions of test specimens

n=0.5, compaction levels 2 & 3

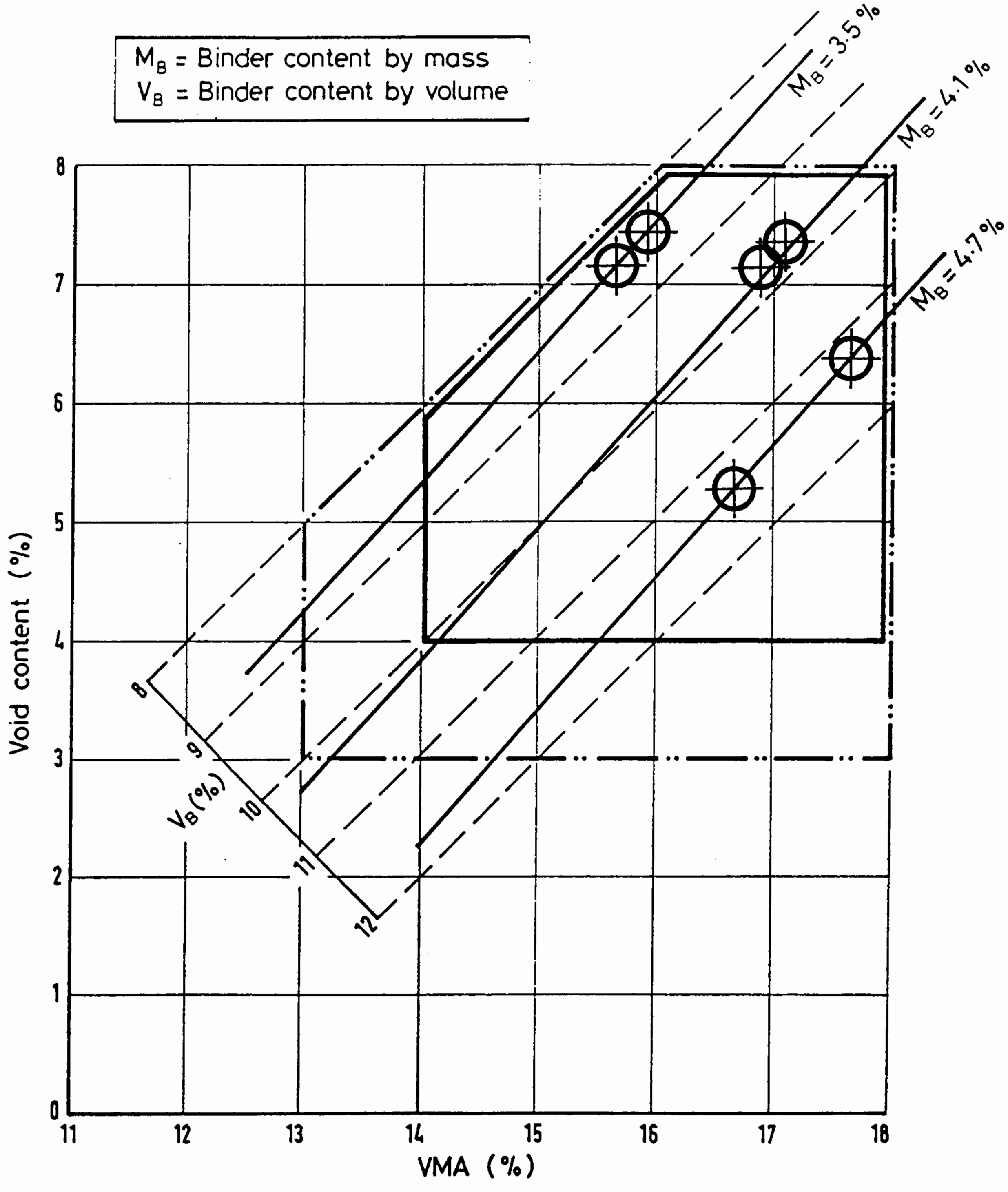

Figure 10.3 Volumetric proportions of test specimens

n=0.6, compaction levels 2 & 3

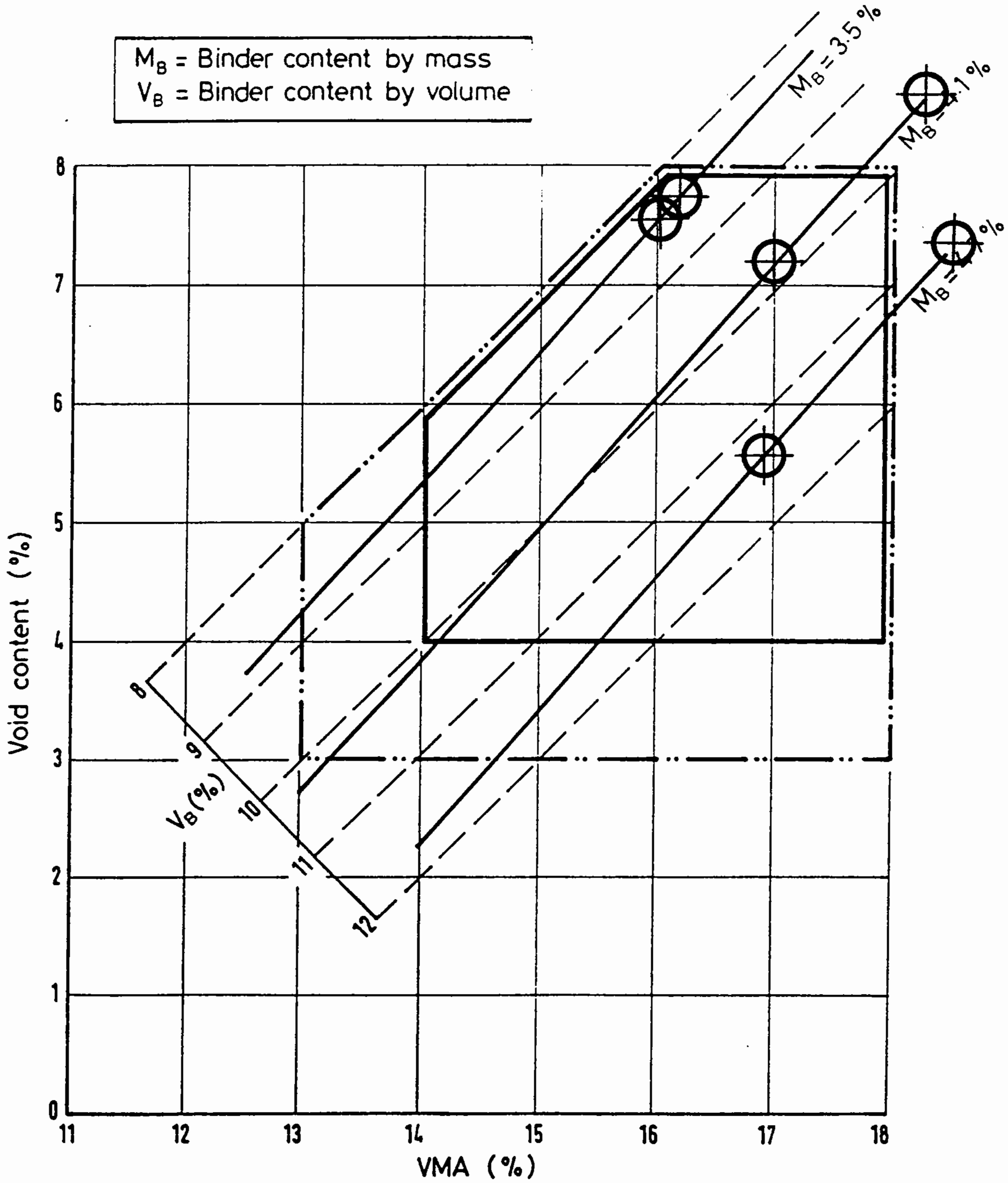

Figure 10.4 Volumetric proportions of test specimens
n=0.7, compaction levels 2 & 3

between compaction levels 2 and 3 for the same mix formulation, where the change in PRD is only a fraction of one percent. This effect did not occur with the limestone aggregate nor the Bardon Hill granite, where a distinct difference in compaction was apparent between the two levels. The higher void contents of the mixtures manufactured with Californian granite are the probable explanation of this phenomenon, creating problems with the density measuring operation. Figure 8.7 showed that the the calculated void contents of bituminous mixtures became artificially depressed if gravimetric measurements were taken without sealing the specimen surfaces, particularly at void contents higher than 6%. Because of the packing characteristics of the Californian granite, the specimens compacted at level 3 possess high void contents which may not be accurately obtained through the procedure which has been followed for density measurements. This in turn would lead to a depressed void content and hence an elevated PRD. This effect has not occurred at the highest binder content of 4.7% where thicker bitumen films on the aggregate particles will prevent the ingress of water into the specimens.

In practice, the analysis of the volumetric proportions would be used to screen the mix formulations and the specimens satisfying the applied criteria would be selected for mechanical testing. In order to make a full evaluation of the Californian granite, all specimens were subjected to the mechanical tests.

10.3.1 Mechanical Properties

Tables 10.7 and 10.8 show ranked values of elastic stiffness for specimens compacted to refusal, and the specimens compacted to the lower levels respectively. The specimens compacted to 100 PRD generally exhibit high values of elastic stiffness, particularly the n = 0.5 and n = 0.6 gradations at the lower binder contents, where all measured values are in excess of 8 GPa. The specimens representing material compacted on site in Table 10.8, show a wide range of stiffnesses with values ranging between 2000 MPa and 8800 MPa. Typically mix formulations manufactured with a binder content of 3.5% and compacted at the intermediate level exhibit the highest values of elastic stiffness, with lower values corresponding to a high grading exponent and a binder content of 4.7%. Three specimens fail to meet the criteria of a required stiffness of 2500 MPa which are the n= 0.7 formulations compacted to the lowest level.

Table 10.7 Ranked elastic stiffness values of specimens compacted to 100 PRD

Specimen No	n	M_B (%)	Compaction level	Elastic Stiffness (MPa)
4	0.5	4.1	1	11750
13	0.6	4.1	1	10000
1	0.5	3.5	1	9100
10	0.6	3.5	1	8100
7	0.5	4.7	1	4650
22	0.7	4.1	1	4600
19	0.7	3.5	1	4550
25	0.7	4.7	1	4000
16	0.6	4.7	1	3800

Table 10.8 Ranked elastic stiffness values of specimens compacted to levels 2 and 3

Specimen No	n	M_B	Compaction Level	Elastic stiffness (MPa)
2	0.5	3.5	2	8800
12	0.6	3.5	3	5500
20	0.7	3.5	2	4800
11	0.6	3.5	2	4800
8	0.5	4.7	2	4100
14	0.6	4.1	2	4100
23	0.7	4.1	2	3800
3	0.5	3.5	3	3700
9	0.5	4.7	3	3650
5	0.5	4.1	2	3510
15	0.6	4.1	3	3500
6	0.5	4.1	3	3500
26	0.7	4.7	2	3100
17	0.6	4.7	2	3050
18	0.6	4.7	3	2700
27	0.7	4.7	3	2400
24	0.7	4.1	3	2300
21	0.7	3.5	3	2000

Results of the RLA tests are shown in Figures 10.5 to 10.10, from which it may be seen that the use of this particular aggregate can produce deformation resistant mixtures. A total of five mixes exhibit an axial strain exceeding 1% at the termination of the test, which are specimens n = 0.5, M_B = 4.7% 100 PRD, n = 0.6 M_B = 4.7% 100 PRD and all of the n = 0.7 specimens when compacted at level 3.

From an inspection of the volumetric proportions, the specimens compacted to refusal at gradations of n = 0.5 and n = 0.6 with binder contents of 4.7%, have become over-compacted causing the binder to fill

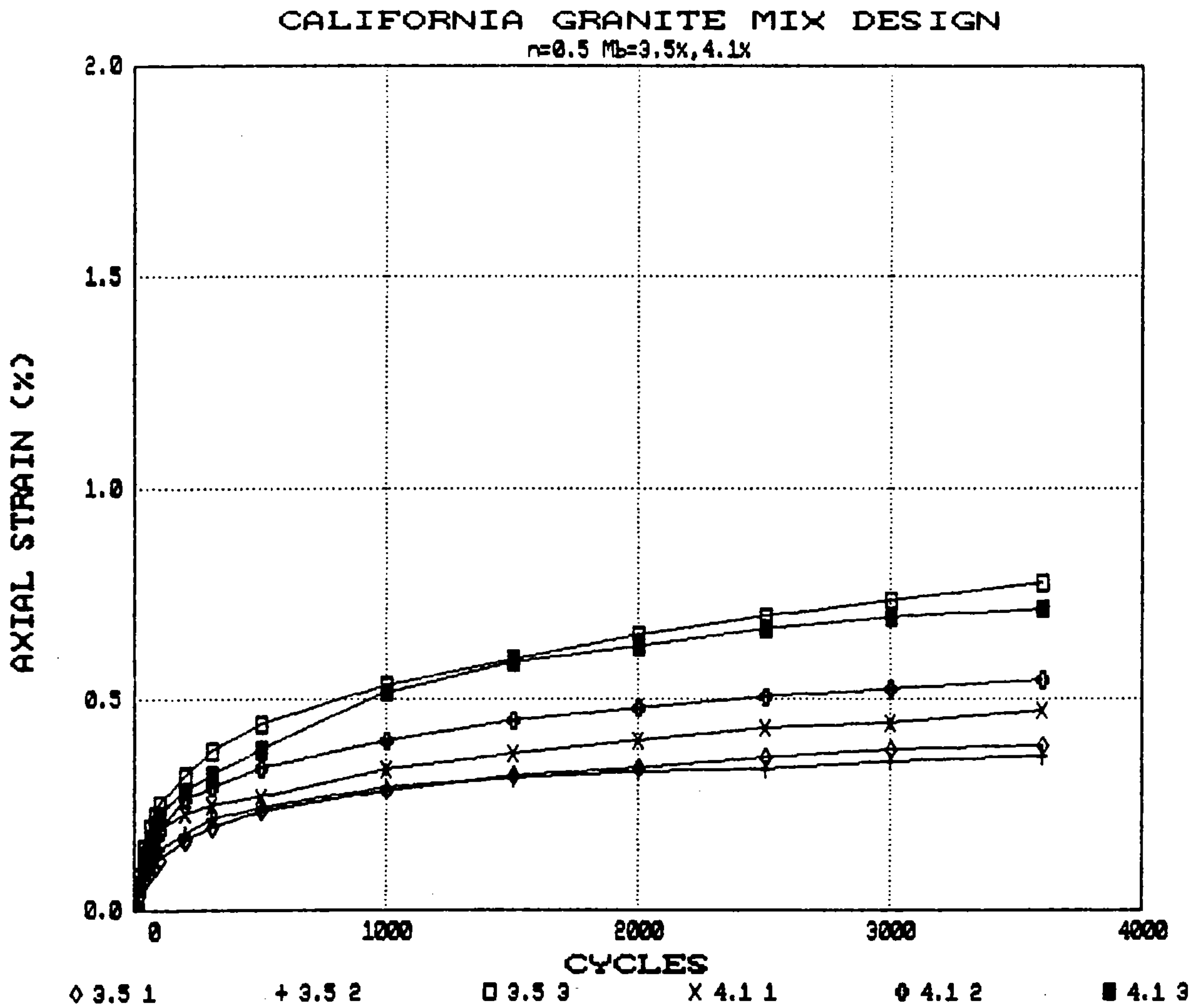

Figure 10.5 Deformation curves plotted from the results of RLA tests. Granite aggregate, mix formulations corresponding to n=0.5, M_B=3.5% and n=0.5, M_B 4.1%.

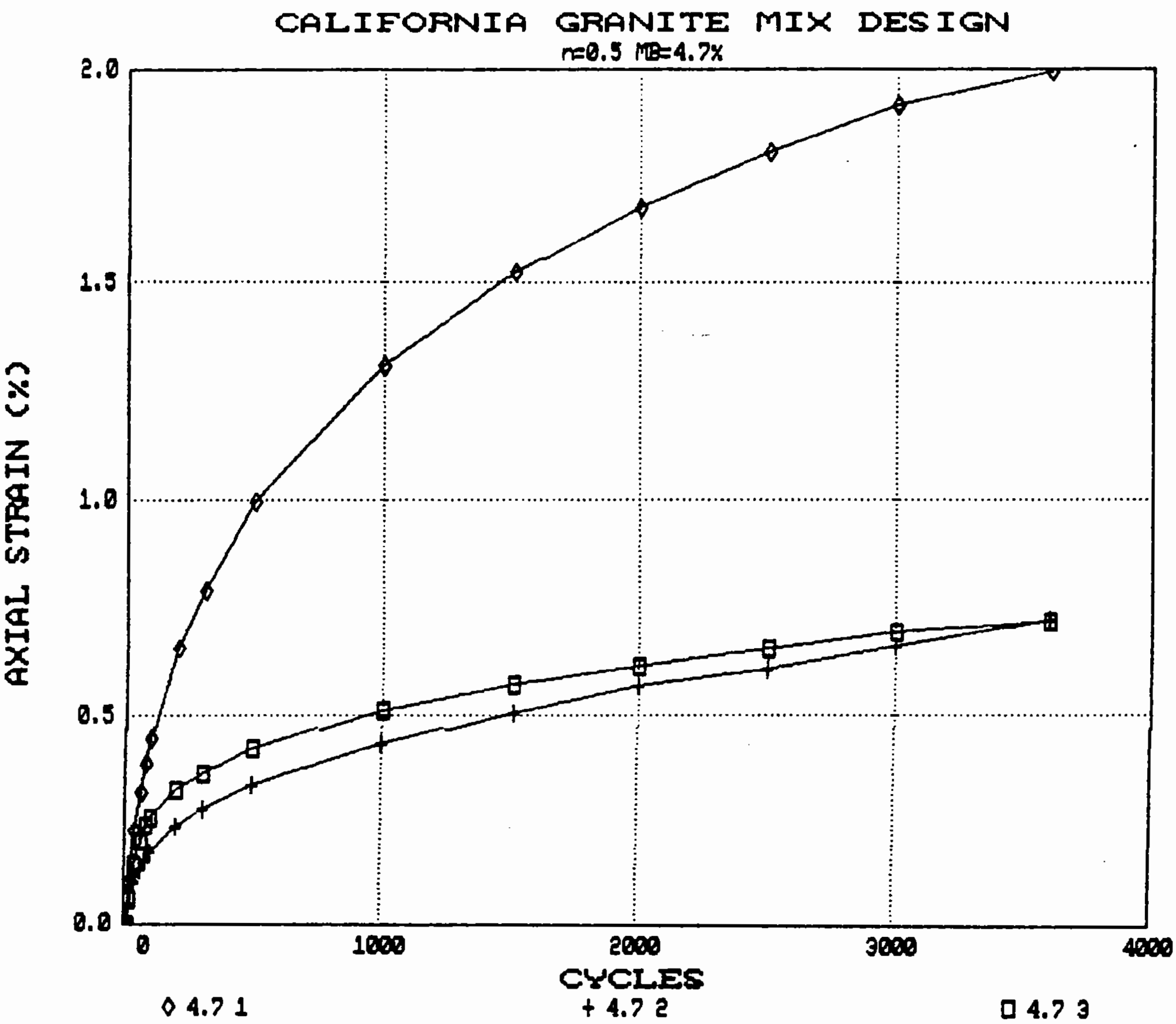

Figure 10.6 Deformation curves plotted from the results of RLA tests. Granite aggregate, mix formulations corresponding to n=0.5, M_B=4.7%.

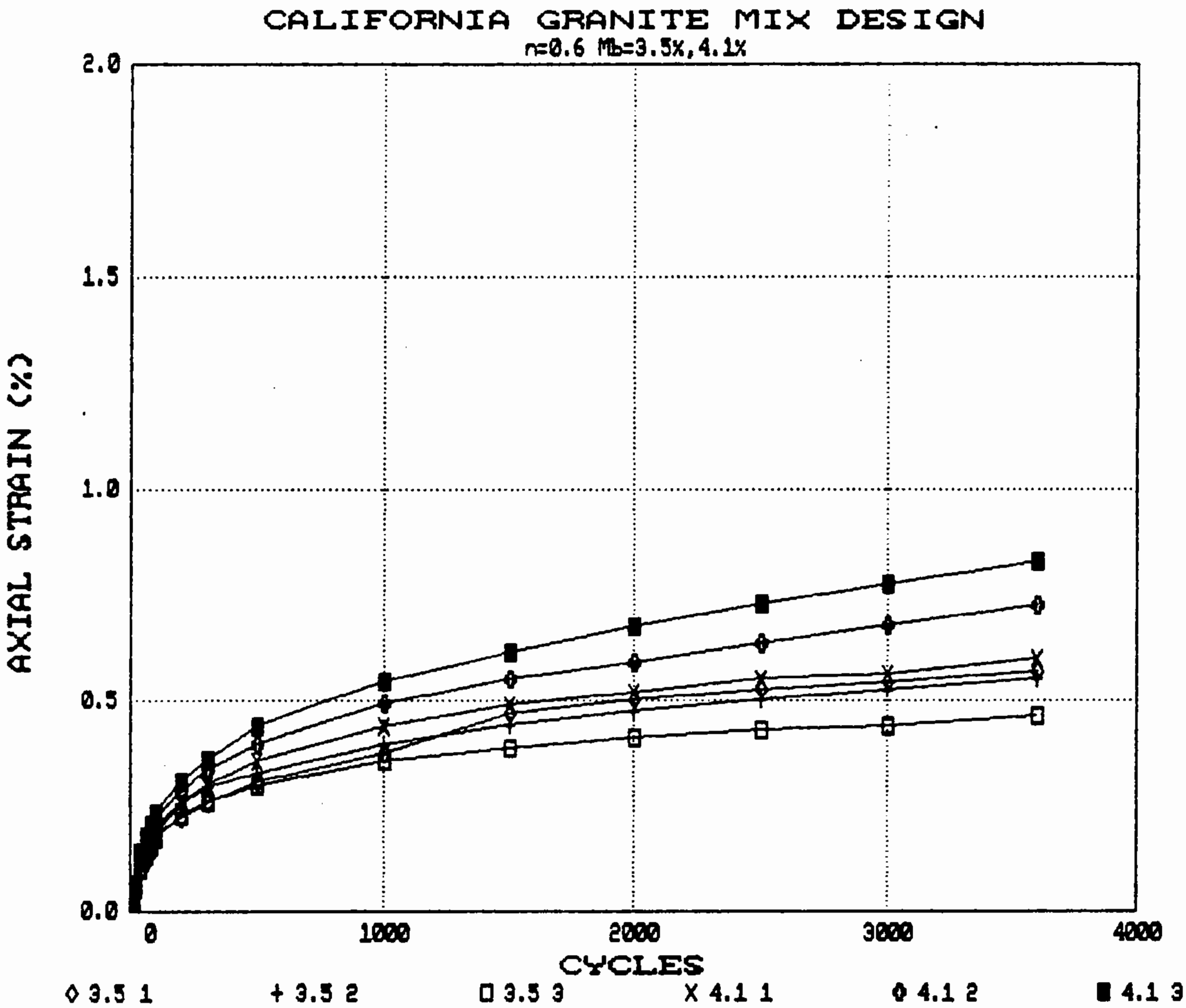

Figure 10.7 Deformation curves plotted from the results of RLA tests. Granite aggregate, mix formulations corresponding to n=0.6, M_B=3.5% and n=0.6, M_B=4.1%

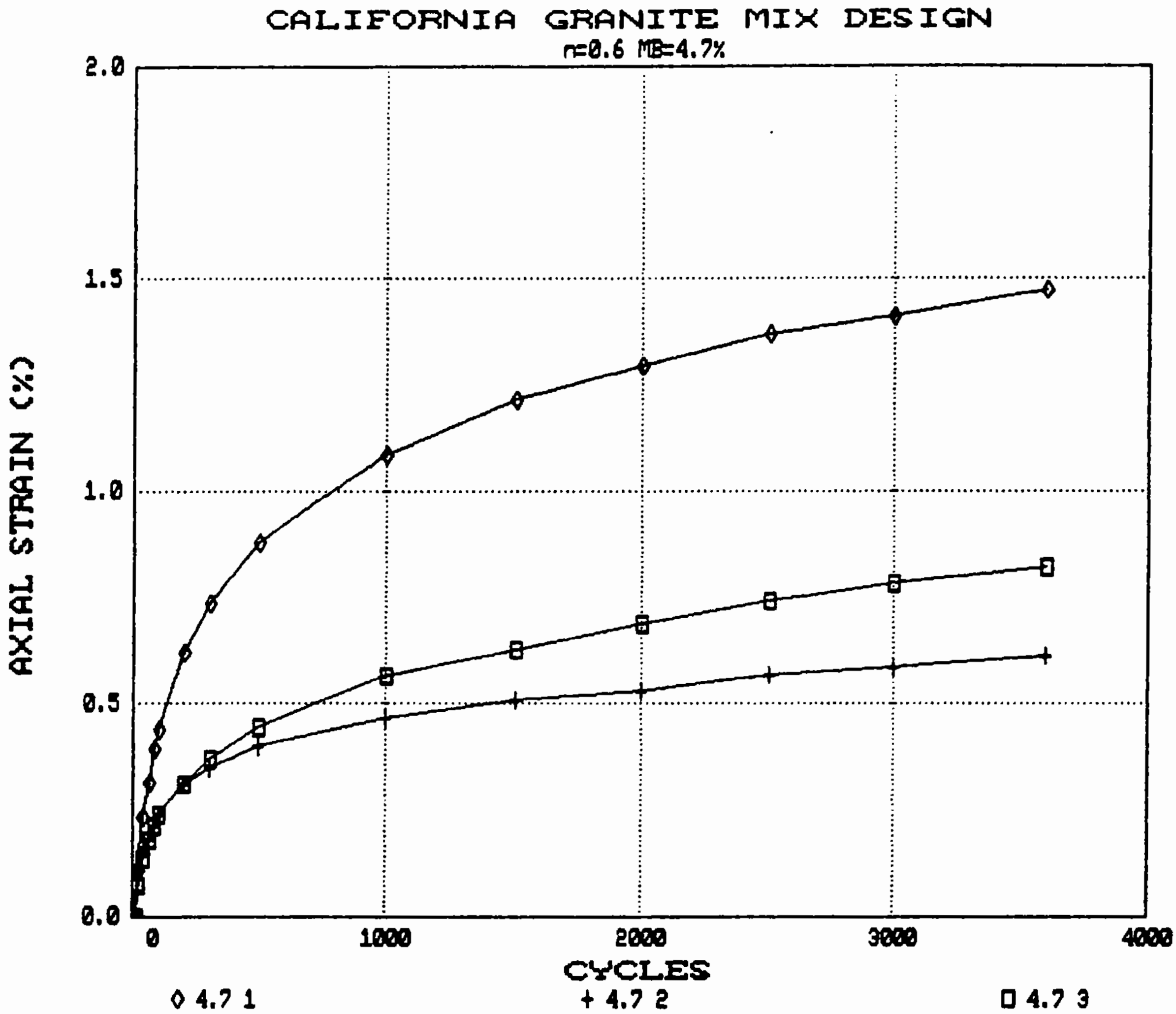

Figure 10.8 Deformation curves plotted from the results of RLA tests Granite aggregate, mix formulations corresponding to n=0.6, M_B=4.7%

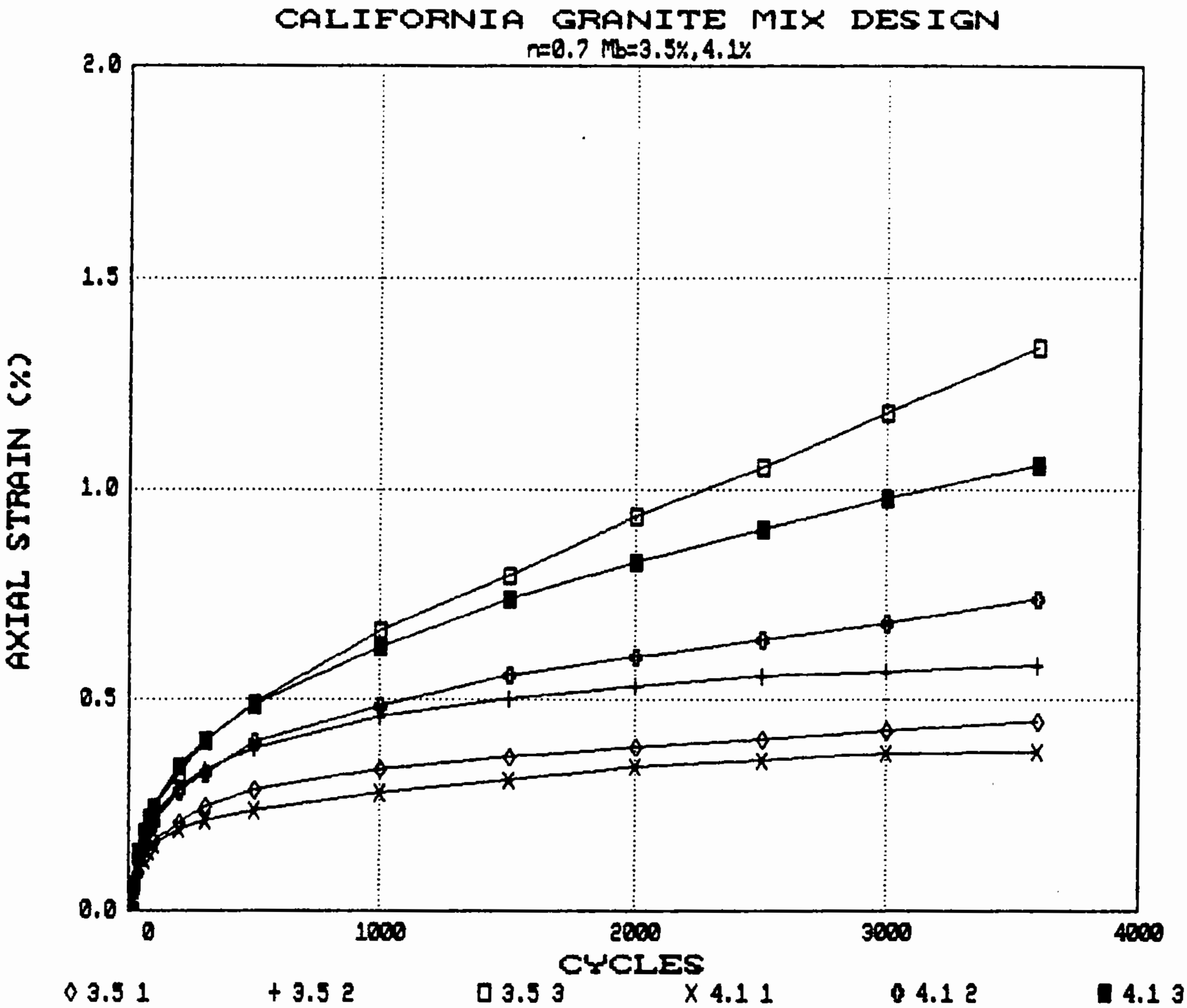

Figure 10.9 Deformation curves plotted from the results of RLA tests. Granite aggregate, mix formulations corresponding to n=0.7, M_B=3.5% and n=0.7 M_B=4.1%

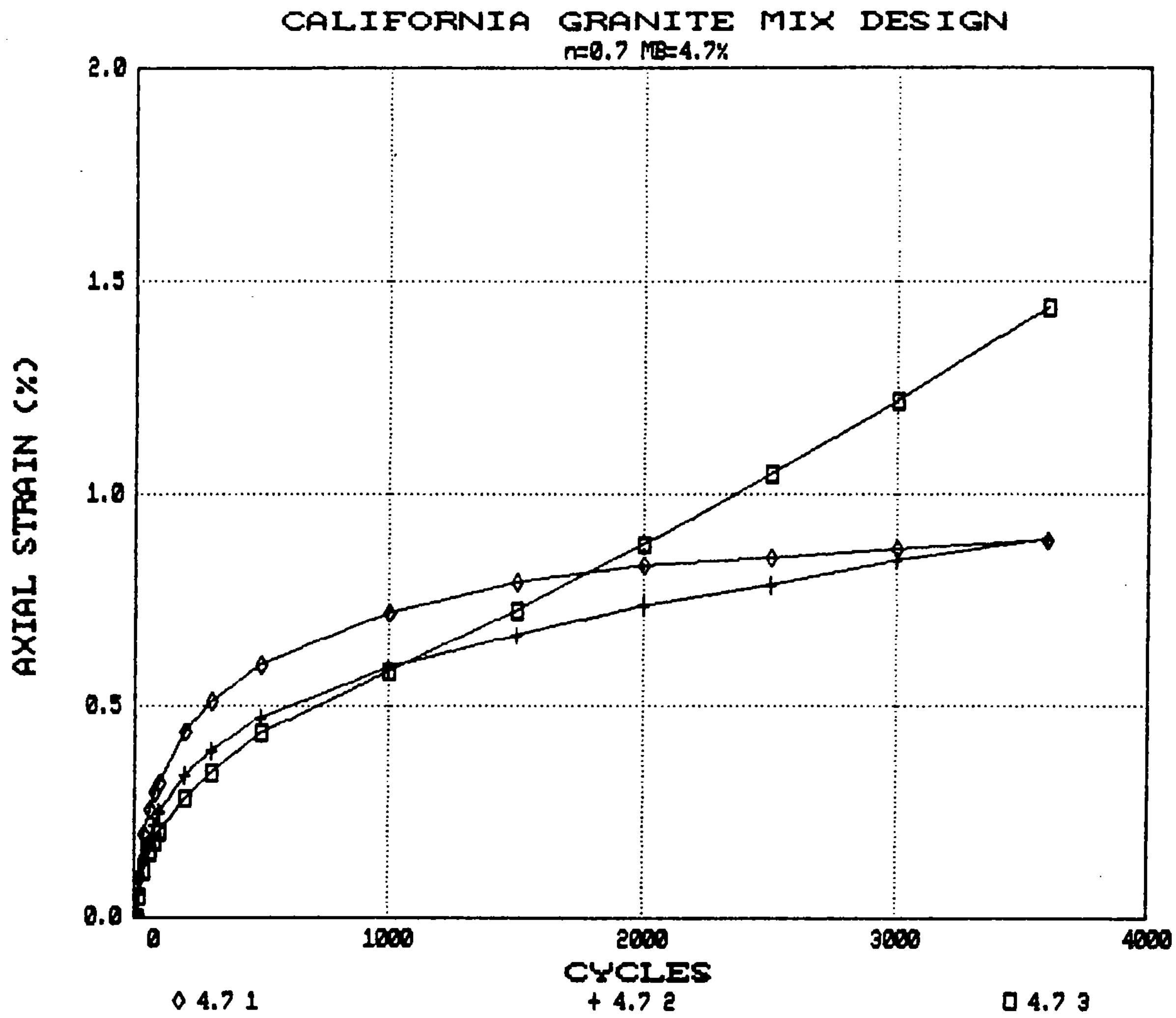

Figure 10.10 Deformation curves plotted from the results of RLA tests. Granite aggregate, mix formulations corresponding to $n=0.7$, $M_B=4.7\%$

the voids and lead to an unstable mix. At lean binder contents, all the gradations exhibit a very high resistance to deformation when compacted to 100 PRD, which is a direct result of how the aggregate packs, creating a stone matrix with a strong interlock, and a high volume of voids. At the coarsest gradation of n = 0.7, the mixtures at refusal density exhibit improved deformation resistance qualities at binder content of 3.5% and 4.1% than the specimens compacted at level 2. Most of the curves on the RLA plots show a good shape characteristic confirming the high quality of the aggregate. Curve assessments considering the strain rate index and ultimate value of strain per specimen are shown in Tables 10.9 to 10.11.

Table 10 . Ultimate strain and strain rate index for specimens at gradation exponent n = 0.5

Specimen No	Mix - formulation	A: Strain at end of test (%)	B: Strain rate index	Product (A x B)
1	0.5 3.5 1	0.39	0.27	0.10
2	0.5 3.5 2	0.37	0.20	0.07
3	0.5 3.5 3	0.78	0.31	0.24
4	0.5 4.1 1	0.47	0.29	0.14
5	0.5 4.1 2	0.55	0.27	0.19
6	0.5 4.1 3	0.72	0.28	0.20
7	0.5 4.7 1	1.99	0.34	0.68
8	0.5 4.7 2	0.72	0.4	0.29
9	0.5 4.7 3	0.72	0.29	0.21

NB Strain rate index $= \dfrac{\varepsilon_{3600} - \varepsilon_{1000}}{\varepsilon_{3600}}$

Table 10.10 Ultimate strain and strain rate index for specimens at gradation exponent n = 0.6

Specimen No	Mix formulation	A: Strain at end of test (%)	B: Strain rate index	Product (A x B)
10	0.6 3.5 1	0.57	0.34	0.19
11	0.6 3.5 2	0.55	0.28	0.15
12	0.6 3.5 3	0.46	0.22	0.10
13	0.6 4.1 1	0.6	0.27	0.16
14	0.6 4.1 2	0.72	0.32	0.23
15	0.6 4.1 3	0.83	0.34	0.28
16	0.6 4.7 1	1.47	0.26	0.38
17	0.6 4.7 2	0.61	0.21	0.13
18	0.6 4.7 3	0.82	0.31	0.25

NB Strain rate index = $\frac{\varepsilon_{3600} - \varepsilon_{1000}}{\varepsilon_{3600}}$

Table 10.11 Ultimate strain and strain rate index for specimens at gradation exponent n = 0.7

Specimen No	Mix formulation	A: Strain at end of test (%)	B: Strain rate index	Product (A x B)
19	0.7 3.5 1	0.45	0.25	0.11
20	0.7 3.5 2	0.58	0.21	0.12
21	0.7 3.5 3	1.34	0.5	0.67
22	0.7 4.1 1	0.38	0.26	0.10
23	0.7 4.1 2	0.74	0.34	0.25
24	0.7 4.1 3	1.06	0.41	0.43
25	0.7 4.7 1	0.89	0.19	0.16
26	0.7 4.7 2	0.89	0.34	0.31
27	0.7 4.7 3	1.44	0.6	0.86

NB Strain rate index = $\frac{\varepsilon_{3600} - \varepsilon_{1000}}{\varepsilon_{3600}}$

The values of strain rate index illustrate the high deformation resistance characteristics of this aggregate. The only specimens which exceed the suggested limit of 0.35 are the mixtures compacted to level 3 at a gradation exponent of n = 0.7, which have been shown to be inferior mixes from previous mix designs. The mix formulations such as n = 0.5, M_B = 4.7%, 100PRD which have exhibited large strains, failing to satisfy the criterion of a maximum axial strain of 1% at the end of the test, have shown 'locking up' characteristics, and a decreasing strain rate as the test progressed. It would appear, therefore, that a bituminous mixture comprising a continuous gradation of the California granite, requires a poorly compacted open grading in order to achieve failure characteristics in terms of strain rate index in a deformation test.

The purpose of the mix design procedure is to make best use of available resources and to quantify material properties against performance based criteria. The Californian granite showed very good mechanical properties in terms of elastic stiffness and deformation resistance, over a range of mix formulations and compaction levels, but the analysis of the results must select a mixture which optimises the material characteristics.

The results of the mix design procedure suggest that for this particular aggregate and binder, a gradation based on a curve corresponding to an exponent of n = 0.5, and a binder content targeting on an average value between 3.5% and 4.1% would give satisfactory end product properties if adequately compacted. This recommendation is very similar to those applied to the limestone granite aggregates which were evaluated in Chapter 9. It is likely that such a mix formulation will typically exhibit optimum qualities, but the allowable tolerances on the grading and binder content will vary according to aggregate type.

10.3.2 Volumetric proportions and mechanical properties

By testing all the specimens in the design procedure, it is possible to assess the accuracy of the initial screening procedure based on the volumetric proportions. All specimens manufactured at the refusal density would be eliminated at the first phase as this does not represent a realistic level of compaction which could be achieved on site. In the case of this aggregate, the 100PRD specimens have exhibited very good mechanical properties at binder contents of 3.5% and 4.1%, comparable with the same mix formulations compacted at level 2. This suggests that over-compaction of these mix formulations is not necessarily undesirable as the volumetric composition of the mixes facilitates structural stability to be maintained. An interesting comparison can be drawn here with the limestone aggregate, (Figures 9.7 to 9.12) where over compaction increases the deformation susceptibility of the mixtures because of the different aggregate packing characteristics. When the considered gradations are mixed with a binder content of 4.7%, over compaction substantially reduces the resistance to permanent deformation.

These effects may be estimated from an evaluation of the volumetric proportions of the full array of specimens, for example the datum specimen of the California granite series with a void content of 3.4% would not be expected to show a significantly reduced resistance to deformation to the corresponding mix formulation compacted to 96 PRD. However, if the same gradation is mixed with a binder content of 4.7%, and exhibits a void content of 1.6% at 100PRD, the structural stability of the specimen may be questioned.

Specimens 3, 24 and 27 which would have been rejected at the first screening, do show inferior mechanical properties compared with associated mix formulations, both in elastic stiffness and deformation resistance. This suggests that the approach to the assessment of volumetric proportions is valid, and the elimination of mix formulations through such a screening process is beneficial to the procedural format of the design method.

10.4 COMPARISON OF NOTTINGHAM DESIGN WITH MARSHALL & HVEEM

In order to make a genuine evaluation of the mix procedure which has been developed through this project, the mix formulation exhibiting optimum characteristics was compared with the mix formulations shown in Table 10.3, resulting from the Marshall and Hveem design procedures. The asphalt contents were taken as 5.1% for both medium and coarse gradations. Specimens were manufactured according to the Nottingham method, the volumetric proportions of which are displayed in Table 10.12.

Elastic stiffness values are also included in the Table where it can be seen that the medium gradation has superior qualities. Inspection of Figure 10.1 shows that the medium gradation represents a similar curve to a grading specified by an exponent of $n = 0.4$. The volumetric proportions of the three specimens manufactured with the medium gradation are not dissimilar to those exhibited by the $n = 0.5$, $M_B = 4.7\%$ mix formulations used in the Nottingham design method, but as indicated by earlier work with limestone aggregate, the n = 0.4 gradation exhibits a slightly higher VMA for a given level of compaction. The medium gradation exhibits the greatest resistance to permanent deformation, Figure 10.11, with optimum resistance occurring at compaction level 2. Values of ultimate strain and strain rate indices for the deformation curves of the SHRP mix formulations are shown in Table 10.13.

Table 10.12 Volumetric proportions and elastic stiffness results of medium and coarse gradations

Grad-	M_B (%)	Compaction level	Density (g/ml)	V_V (%)	V_B (%)	VMA (%)	PRD (%)	Elastic stiffness (MPa)
MG	5.1	1	2.525	1.8	12.6	14.4	100	4800
MG	5.1	2	2.422	5.8	12.1	17.9	95.9	5000
MG	5.1	3	2.399	6.7	12.0	18.7	95	4850
CG	5.1	1	2.516	2.1	12.6	14.7	100	4700
CG	5.1	2	2.501	6.6	12.0	18.6	95.4	3100
CG	5.1	3	2.395	6.8	12.0	18.8	95.2	2400

MG : Medium Gradation, CG : Coarse Gradation

The 100 PRD and CG, M_B = 5.1 level 3 specimens exhibit a strain in excess of 1% at the termination of the test, and a product value greater than 0.35. It is apparent from the elastic stiffness values and the RLA results, that the coarse gradation, which does not approximate to any curve specified by the grading equation, is an inappropriate grading for this aggregate type.

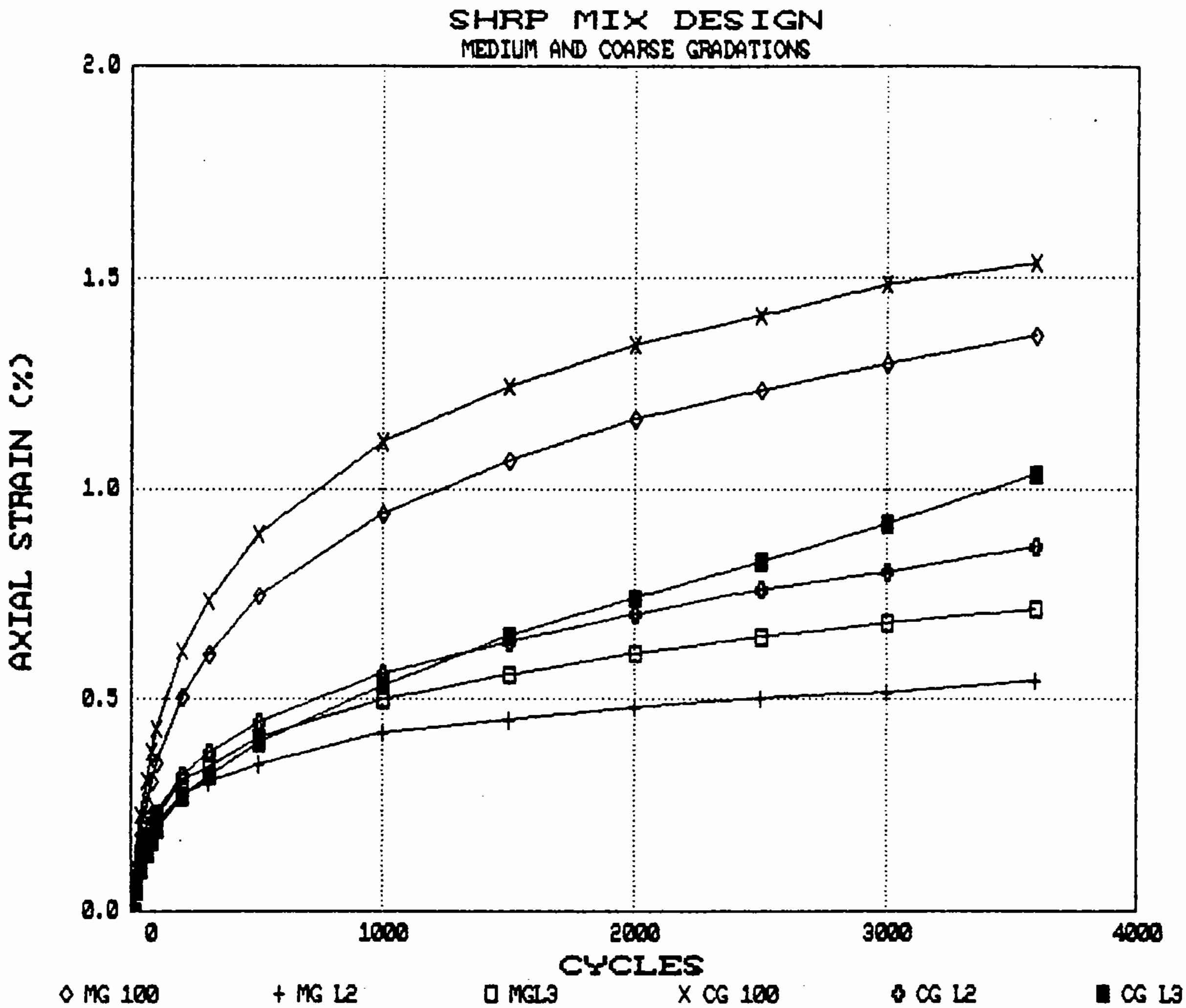

Figure 10.11 Deformation curves plotted from the results of RLA tests. Granite aggregate, mix formulations corresponding to the SHRP recommendations of the medium and coarse gradations.

Table 10.13 Ultimate strain and strain rate index for SHRP mix formulations

Mix formulation	A: Strain at end of test (%)	B: Strain rate index	Product (A X B)
MG 5.1 1	1.37	0.31	0.43
MG 5.1 2	0.55	0.23	0.17
MG 5.1 3	0.71	0.30	0.21
CG 5.1 1	1.54	0.28	0.43
CG 5.1 2	0.86	0.35	0.30
CG 5.1 3	1.04	0.48	0.50

10.4.1 Comparison of optimum designs

The mix formulations exhibiting the preferential mechanical properties were the n = 0.5, M_B = 3.1% level 2 specimen from the Nottingham design, and the Medium Gradation, M_B = 5.1% level 2 specimen from the SHRP mixtures representing Marshall and Hveem designs. Direct comparisons between the mechanical properties are shown in Table 10.14 and Figure 10.12 from which it may be concluded that the Nottingham designed formulation shows improved characteristics over the specimen recommended from 'traditional' design methods.

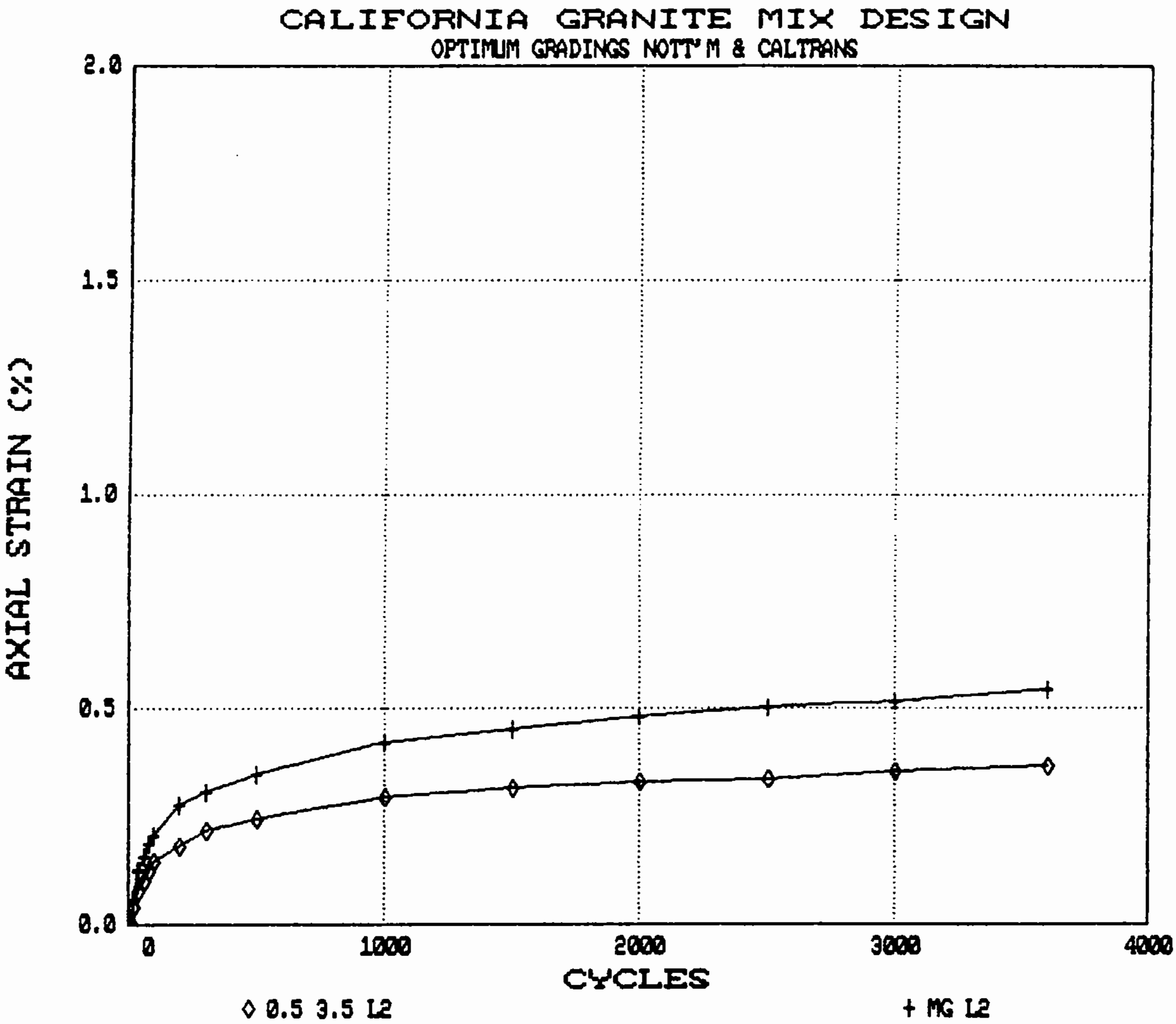

Figure 10.12 Comparison of the deformation resistance characteristics of the mix formulations: n=0.5, M_B=3.5% level 2 and the Medium Gradation M_B=5.1% Level 2

Table 10.14 Mechanical properties of optimum mix formulations resulting from each design method

Mix formulation	Elastic stiffness (MPa)	Strain at end of test (%)	Strain rate index
n = 0.5, M_B = 3.5% level 2	8800	0.37	0.20
M.G, M_B = 5.1% level 2	5000	0.55	0.23

10.4.1.1 Fatigue Resistance

The substantial difference in binder contents between the two mix formulations indicates there will be a corresponding difference in the predicted fatigue lives of the two mixtures. Figure 10.13 illustrates the fatigue relationships based on the prediction model discussed in Chapter 4, which utilises input parameters of volume of binder in the mix, and the initial softening point of the binder to construct fatigue lines. For the inducement of a given tensile strain, the SHRP mix formulation exhibits a longer fatigue life than the n = 0.5, M_B = 3.5% level 2 mixture, but for a fixed layer thickness, the tensile strains generated at the underside of the layer will differ for the two mix types because of their different values of elastic stiffness. The Nottingham mix formulation exhibits the greatest elastic stiffness and therefore a reduced tensile strain for a given applied load. The evaluation of the mix formulations in terms of the fundamental parameters, with particular reference to the assessment of fatigue properties, should be performed in parallel with pavement design, which will enable consideration to be given to the effect of layer thickness and the associated economic implications. This is illustrated through an example shown in Appendix D.

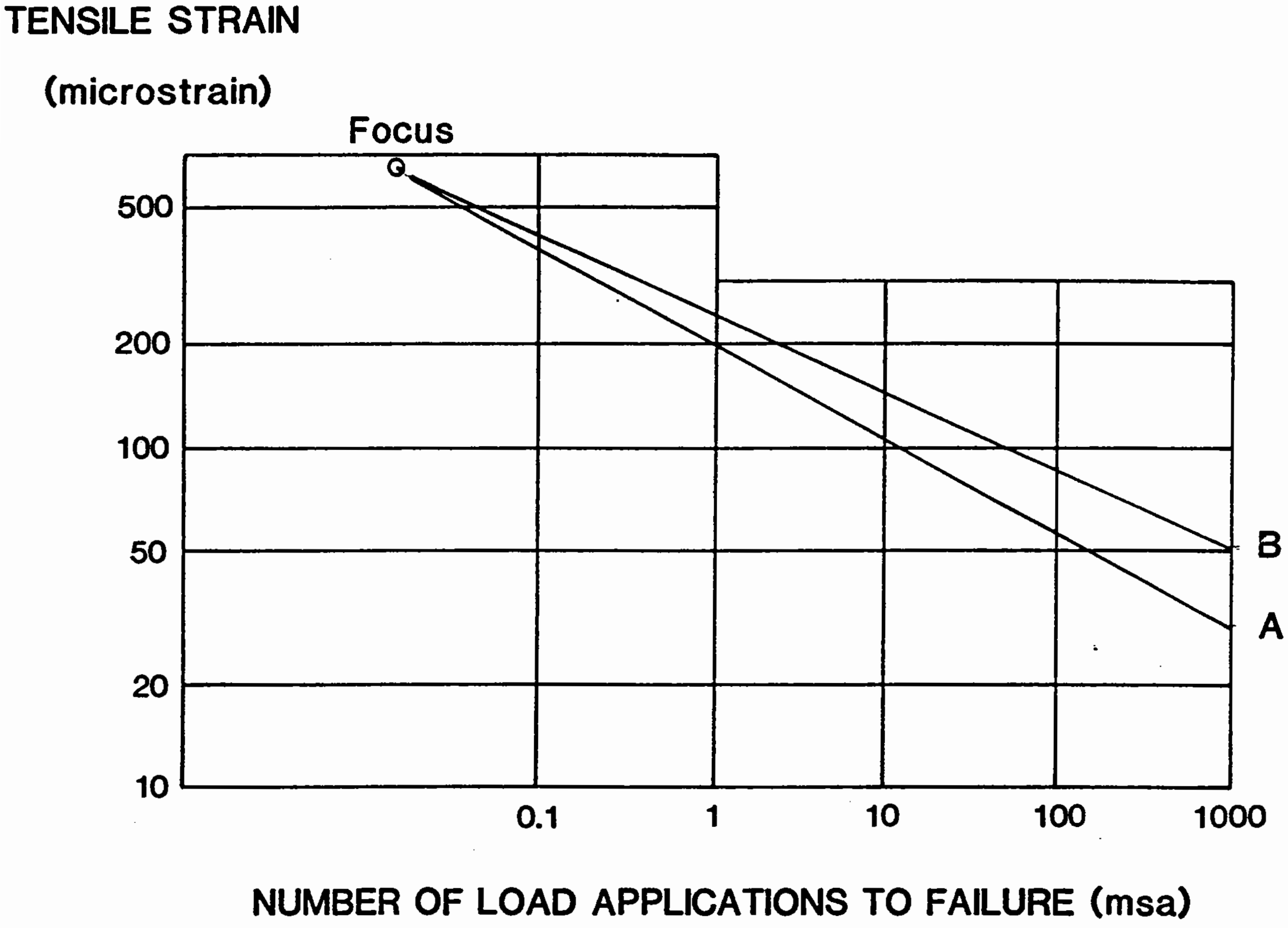

Figure 10.13 Fatigue relationships of mix A (n=0.5, M_B=3.5% level 2) and mix B (Medium Gradation, M_B=5.1% level 2)

10.4.2. Discussion

The new design procedure has demonstrated that, for a given source of aggregate, mix formulations with good mechanical properties can be identified for roadbase and basecourse layers. The investigation performed with the Californian granite showed that the Nottingham approach resulted in a mix formulation with a superior elastic stiffness, and improved deformation resistance to the mix formulations resulting from the Marshall and Hveem designs. The exercise was carried out using individual fraction sizes of aggregate, sieved out from the quarry bins, facilitating absolute accuracy to target gradations forming an idealised investigation. In practice, the blending of quarry bin sizes will cause deviations from the target gradings, which is likely to exaggerate the differences in mixture properties.

The assessment of the SHRP mix formulation comprising an aggregate gradation which approximates to curve represented by an exponent of n = 0.4, allows a further evaluation of the general influence of the role of the aggregate grading on the mechanical properties of the mixture. In order to fully explore the effects, a series of mixtures were manufactured using the SHRP gradation with binder contents of 3.5%, 4.1% and 4.7%, to correspond with the Nottingham formulations. The volumetric proportions of the specimens are shown in Table 10.15.

Table 10.15 Volumetric proportions of mix formulations fabricated at the SHRP 'medium' gradations

n	M_B	Compaction level	Density	V_V	V_B	VMA	PRD
0.4 (MG)	3.5	1	2.455	7.0	8.4	15.4	100
0.4 (MG)	4.1	2	2.360	10.6	8.1	18.7	96.1
0.4 (MG)	4.7	3	2.275	13.8	7.8	21.6	92.7
0.4 (MG)	3.5	1	2.503	4.2	10.1	14.3	100
0.4 (MG	4.1	2	2.386	8.7	9.6	18.3	95.3
0.4 (MG)	4.7	3	2.323	11.1	9.3	20.4	92.8
0.4 (MG)	3.5	1	2.524	2.5	11.6	14.1	100
0.4 (MG)	4.1	2	2.409	6.9	11.1	18.0	95.4
0.4 (MG)	4.7	3	2.369	8.5	10.9	19.4	93.9

The n = 0.4, M_B = 3.5% level 1 specimen exhibits a very high void content, commensurate with the previous findings that a gradation fine of the maximum density curve, mixed with a lean binder content has a lower compactibility than a corresponding formulation course of the fuller curve. The relationships between the grading exponent, binder content and compaction level are illustrated in Figures 10.14 to 10.16. In each case, maximum density appears to occur at a grading exponent of n = 0.5/n = 0.6, with a minimum VMA becoming less pronounced as the binder content increases.

The influence of the grading exponent on the mechanical properties of the mixtures is shown on Figures 10.17 to 10.22. Maximum elastic stiffness occurs at the refusal density compaction level, with a peak value typically coinciding with a grading exponent of n = 0.5, although the optima migrates to n = 0.4 at a binder content of 4.7%. Compaction level 2

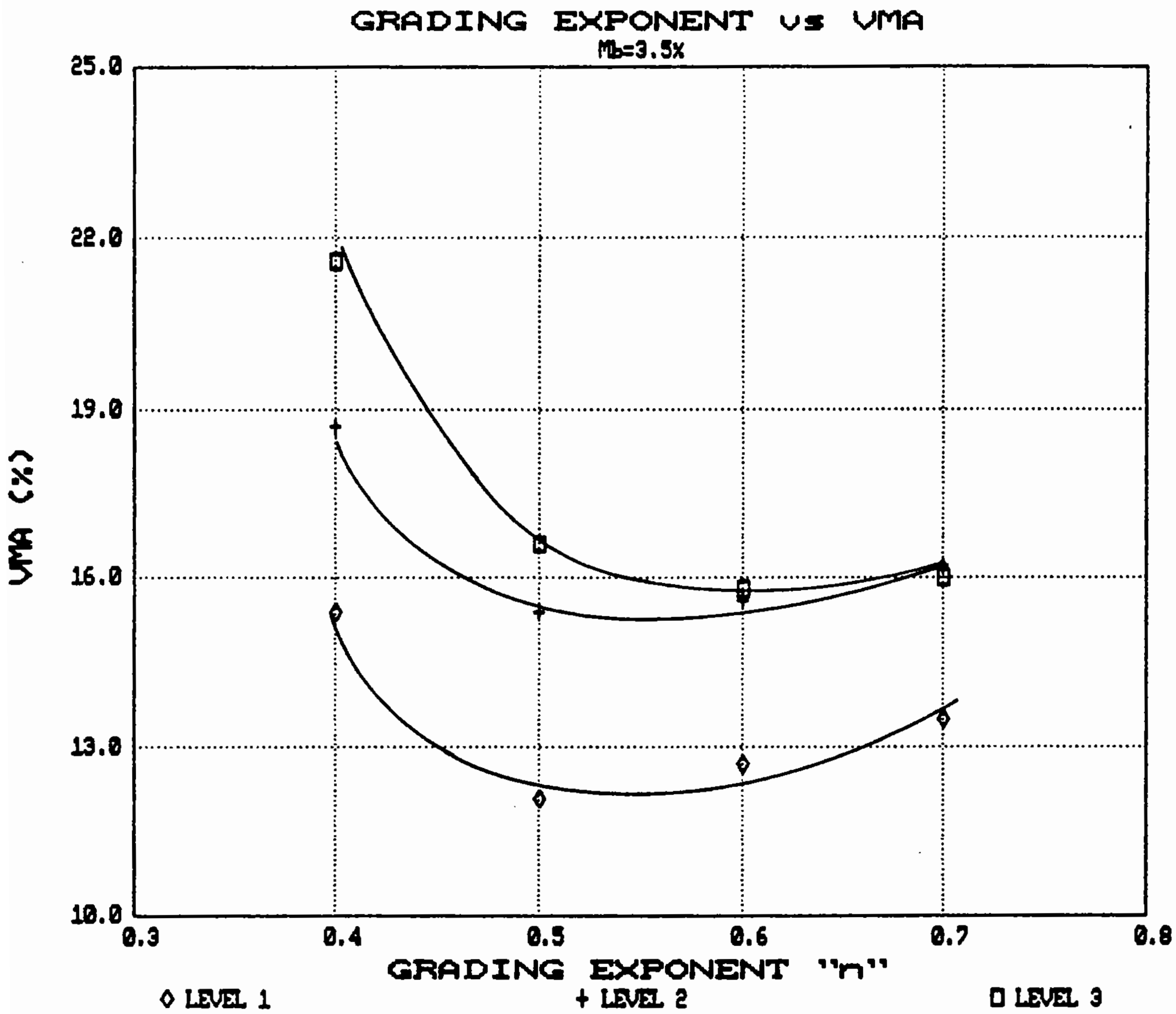

Figure 10.14 Influence of grading exponent on VMA (M_B =3.5%)

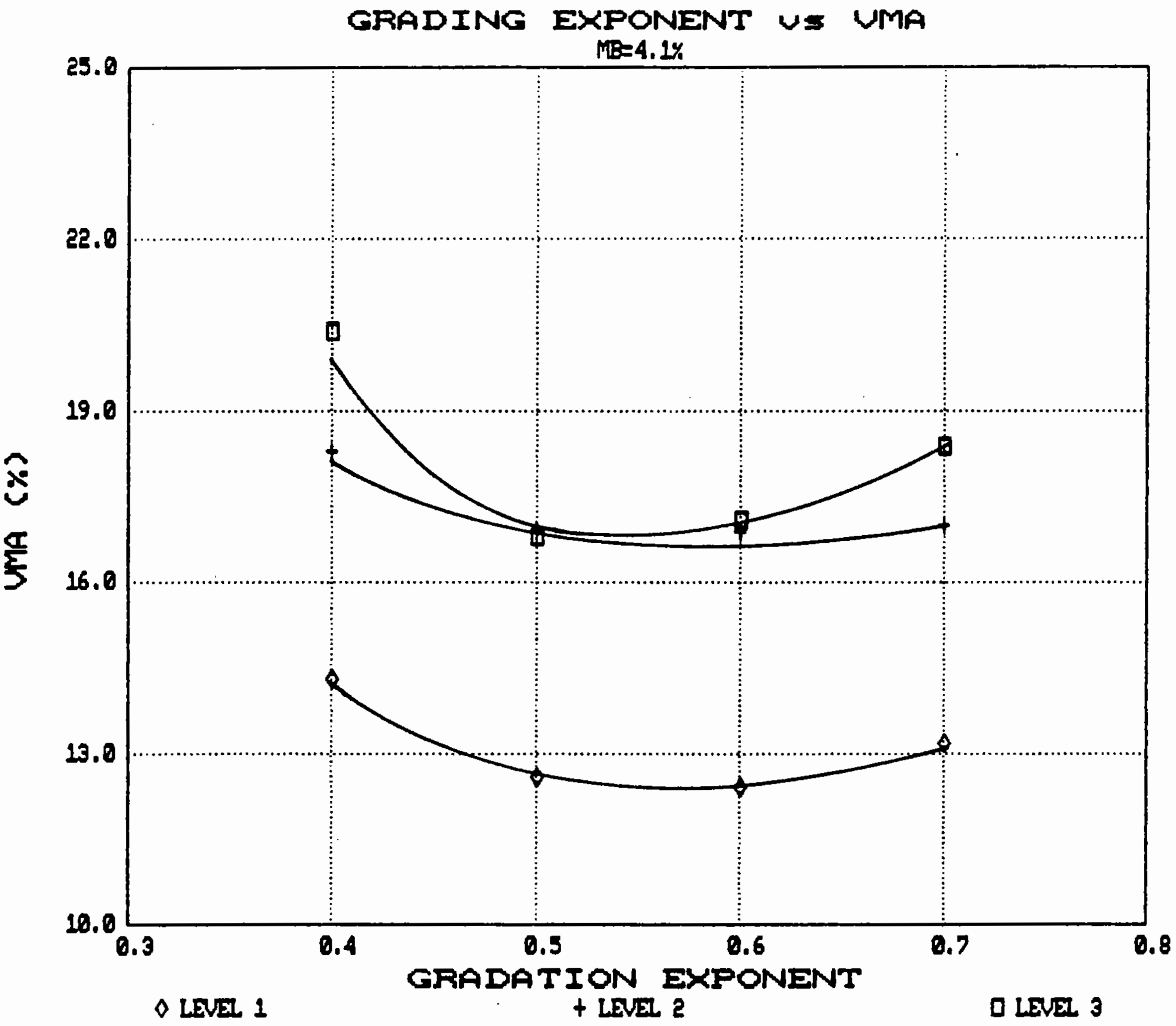

Figure 10.15 Influence of grading exponent on VMA (M_B=4.1%)

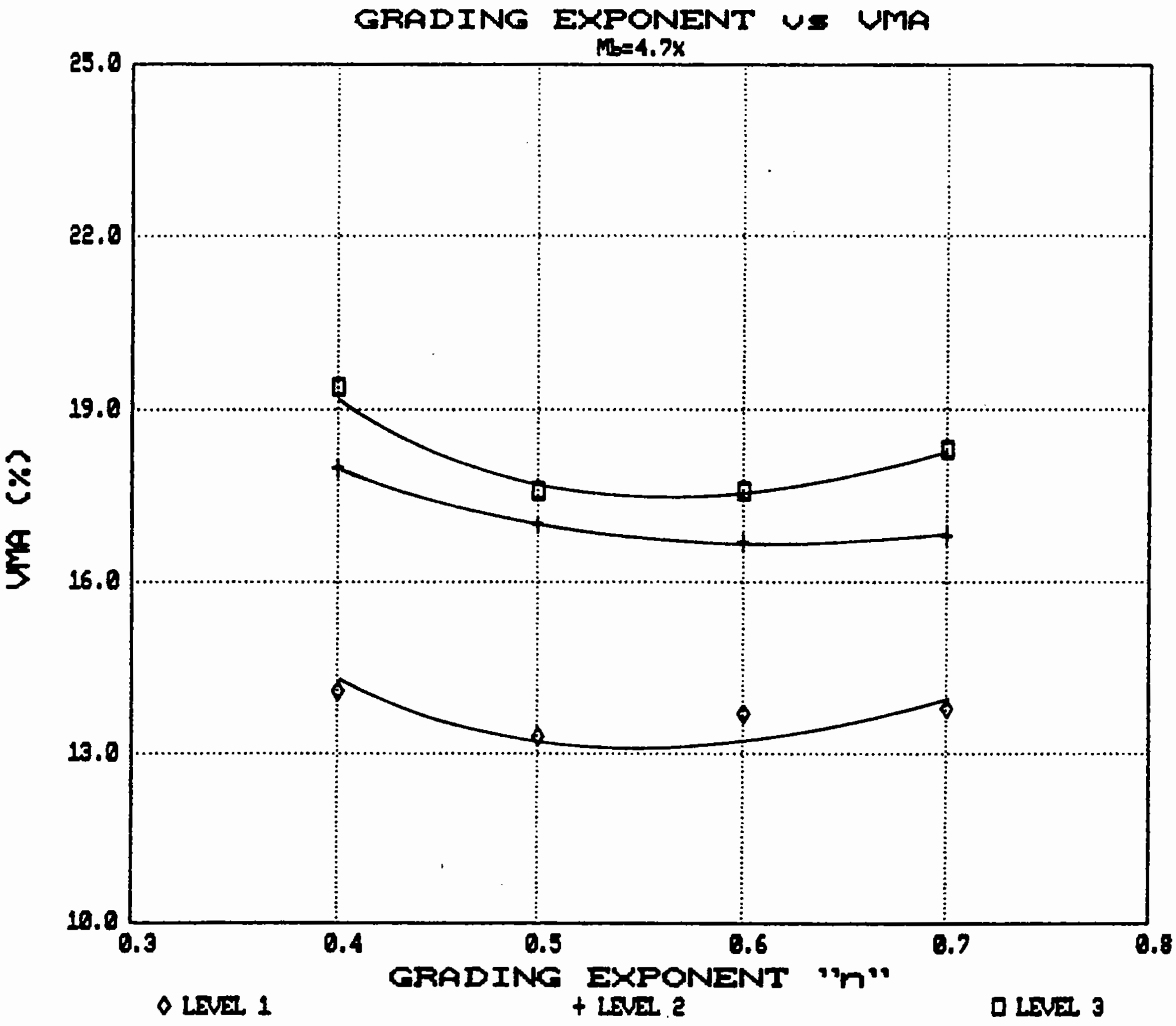

Figure 10.16 Influence of grading exponent on VMA (M_B=4.7%)

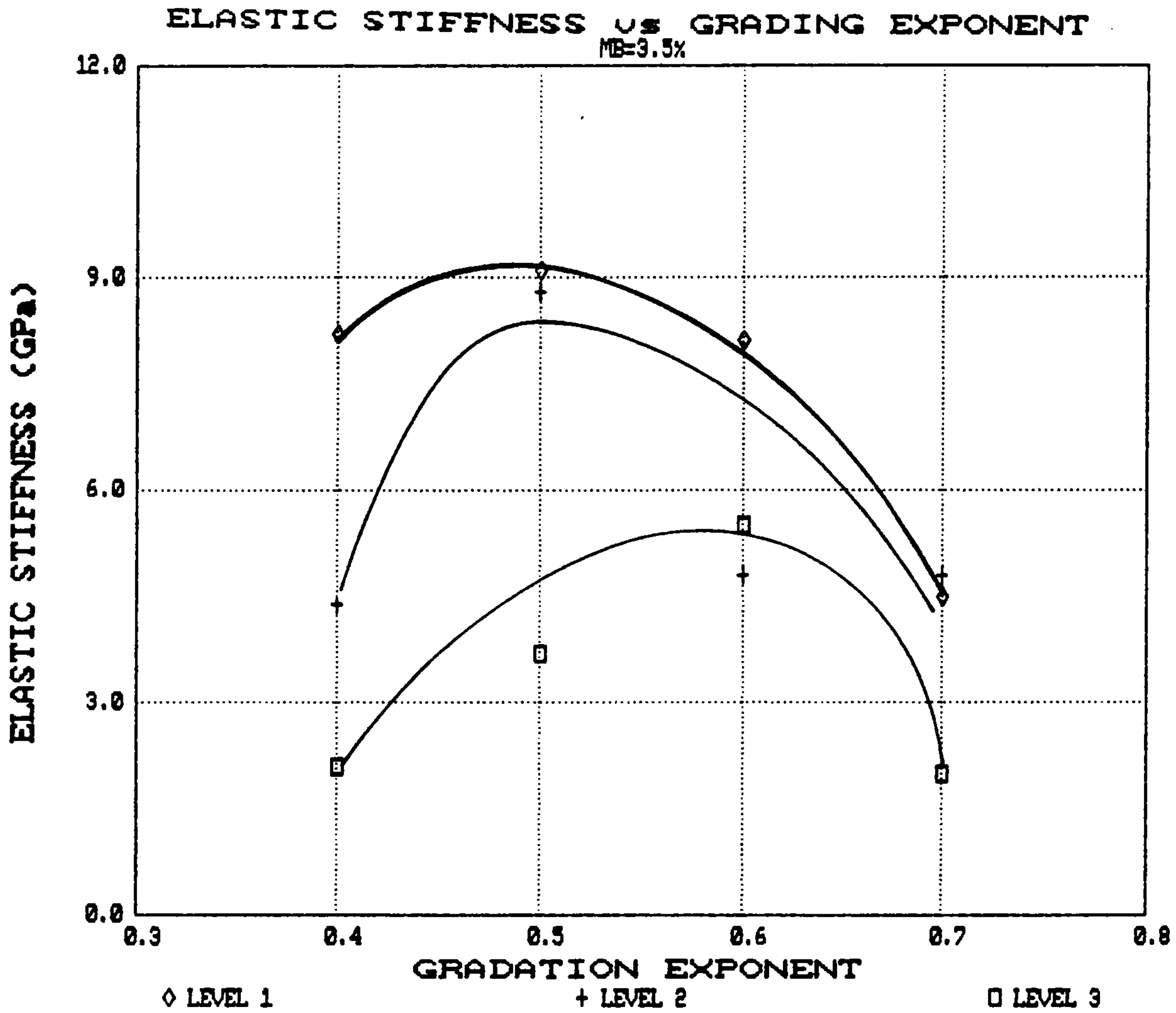

Figure 10.17 Influence of grading exponent on elastic stiffness (M_B=3.5%)

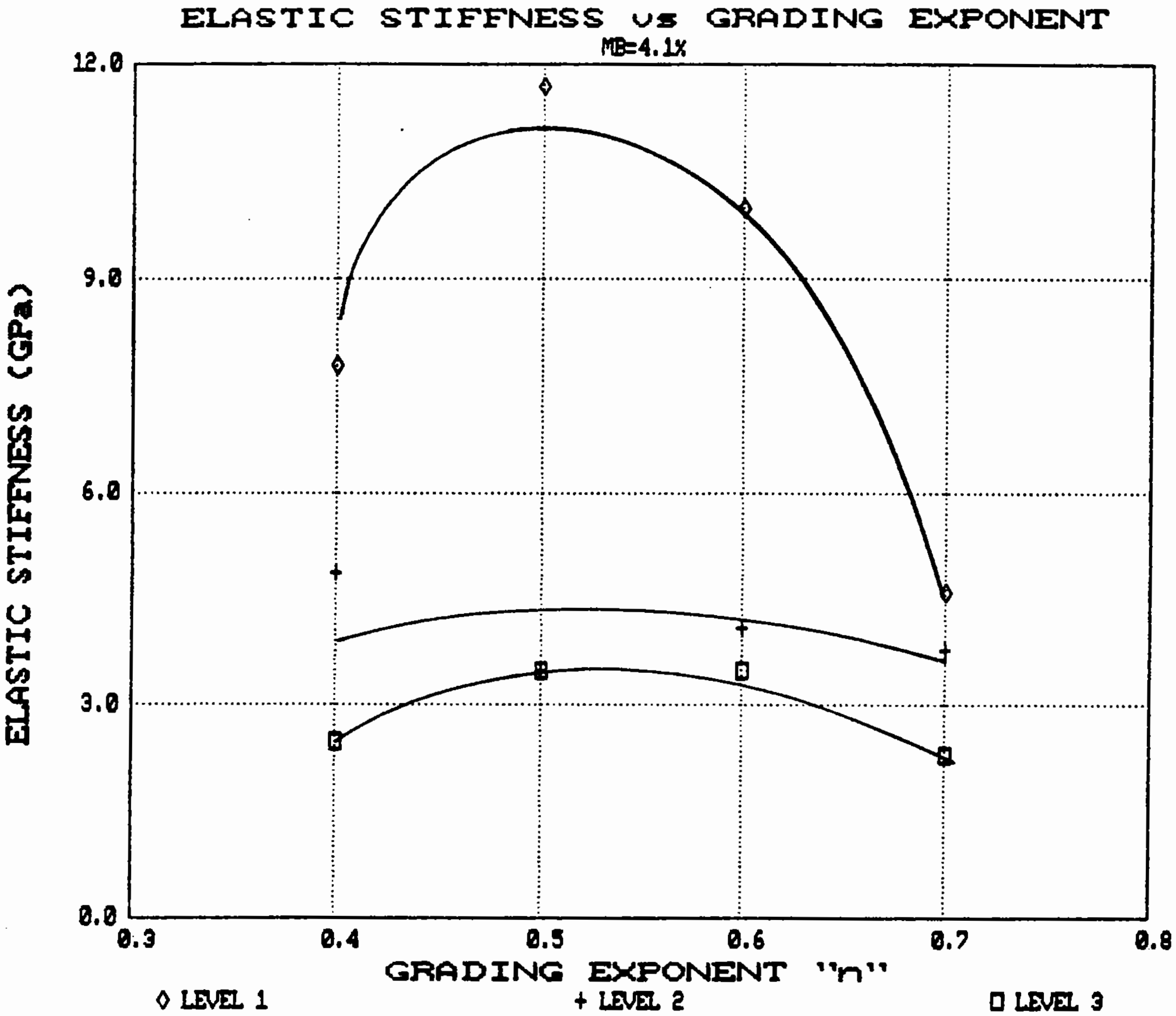

Figure 10.18 Influence of grading exponent on elastic stiffness (M_B=4.1%)

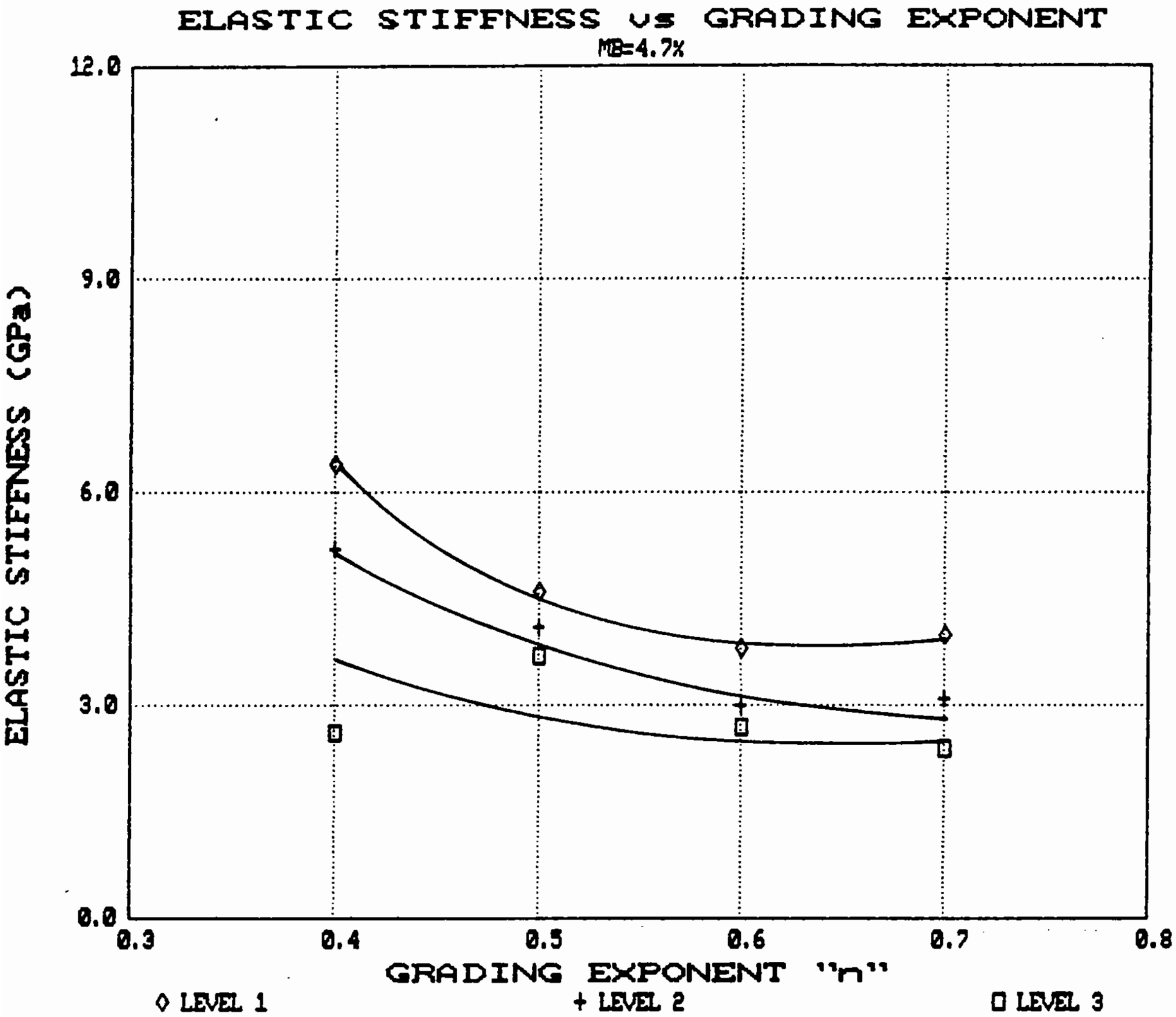

Figure 10.19 Influence of grading exponent on elastic stiffness (M_{B}=4.7%)

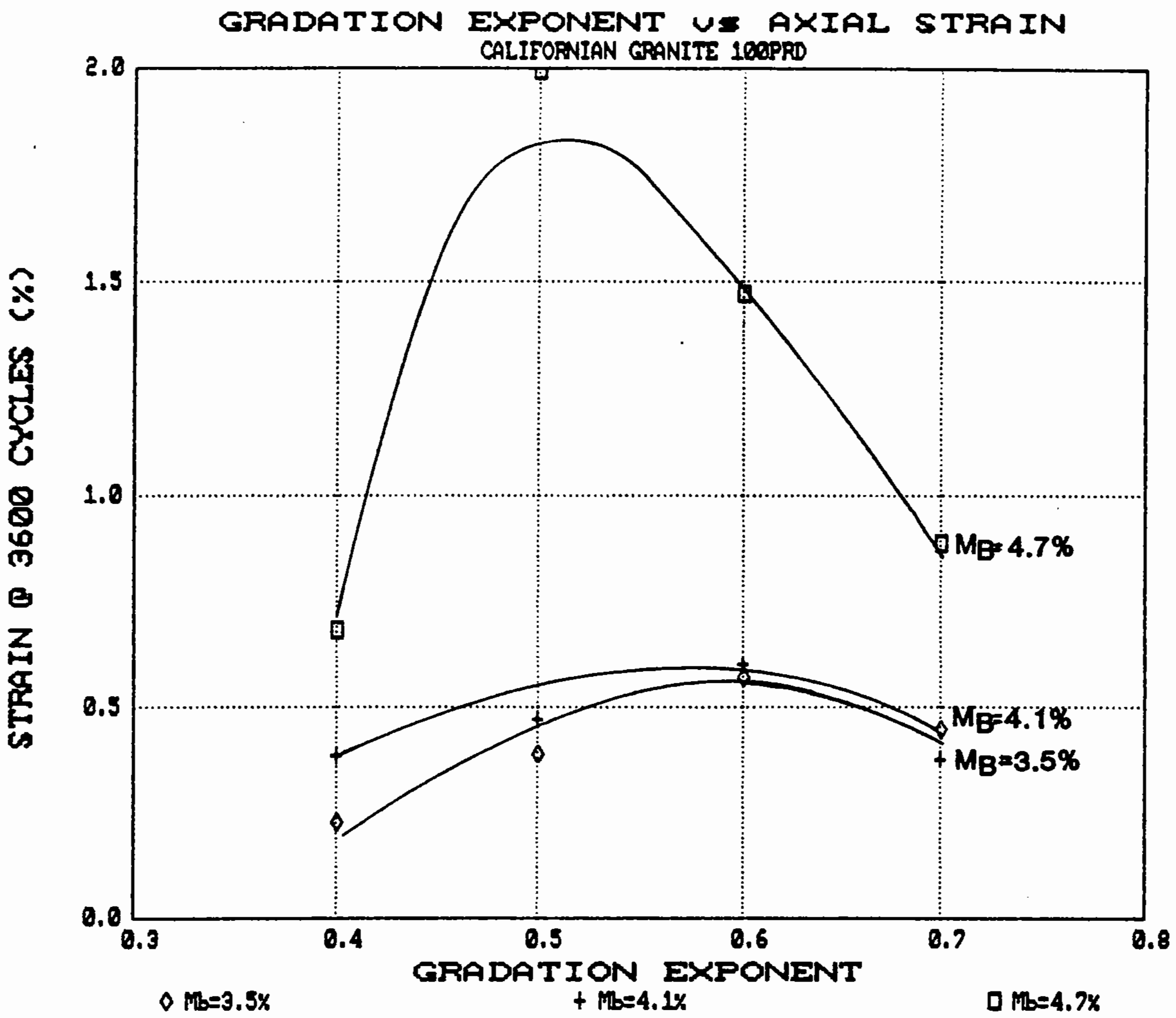

Figure 10.20 Influence of grading exponent on deformation resistance (specimens compacted to refusal)

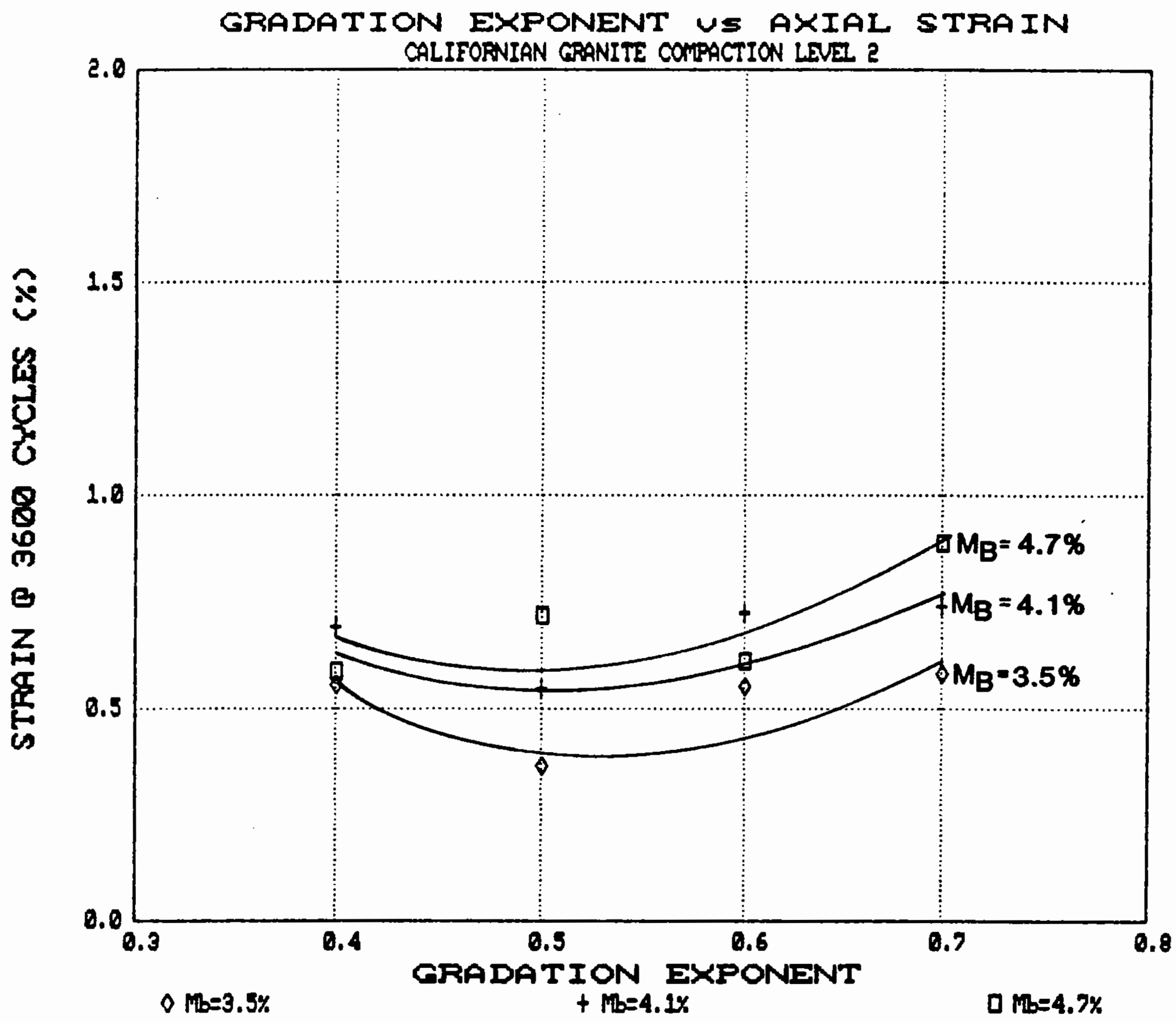

Figure 10.21 Influence of grading exponent on deformation resistance (specimens compacted at level 2)

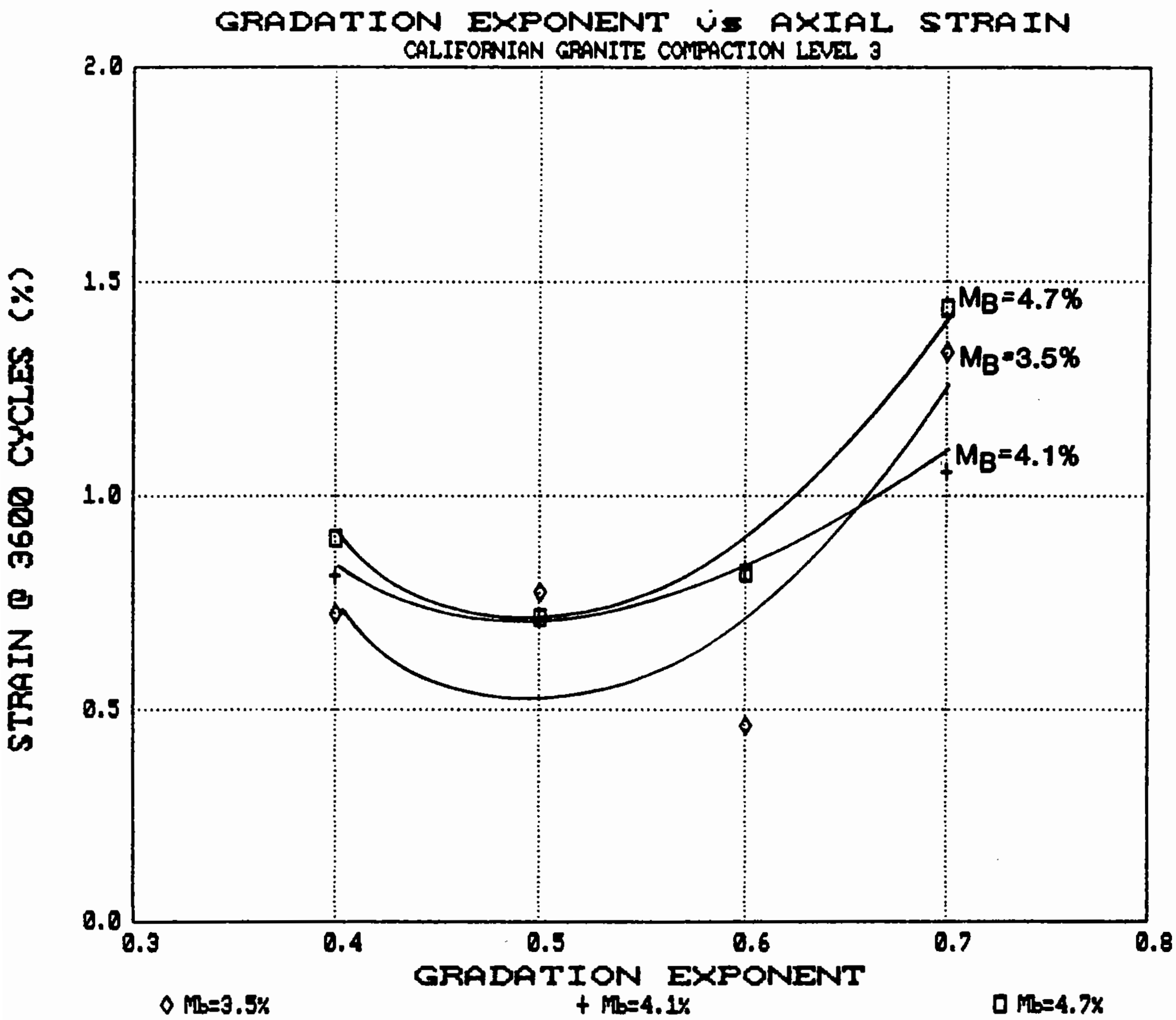

Figure 10.22 Influence of grading exponent on deformation resistance (specimens compacted at level 3)

represents satisfactorily compacted site material, the results of which indicate that preferential elastic stiffness was achieved at an aggregate grading of n = 0.5 mixed with a binder content of 3.5%. The Figures show that elastic stiffness generally decreases as the binder content increases, although the n = 0.4 gradation exhibits stable values throughout binder variations. The resistance to permanent deformation may be quantified by the measured value of strain at the end of the RLA test. At compaction levels 2 and 3, optimum resistance was achieved with a grading exponent of 0.5 and a binder content of 3.5% (Figures 10.21 and 10.22), with an increasing binder content coinciding with a reduction in deformation resistance. The n = 0.5 and n = 0.6 gradations maintain a high resistance to deformation at very high states of compaction at the lean and medium binder contents, but lose structural stability when a high binder content (4.7%) is introduced to the gradation, (Figure 10.20).

From an overall assessment of the Figures it may be concluded that the optimum mix performance in terms of elastic stiffness and deformation resistance occurs at a gradation which approximates to n = 0.5, a lean binder content of 3.5% and a compaction level in excess of 95 PRD. This conclusion is commensurate with previous mix designs following the Nottingham procedure. Both the n = 0.4 and n = 0.6 gradations exhibit good properties indicating the tolerances which could be allowed on a target gradation using this aggregate type. The n = 0.7 grading and the high binder content of 4.7% are synonymous with mixtures of inferior properties.

Chapter 3 introduced the concept of characterising aggregate gradings by considering the percentage of material passing a sieve of size 0.03 times the maximum particle size. Work performed by Hudson and Davies (36) and more recently Cooper and Brown (35) suggested a link between this parameter, and the mechanical properties of associated bituminous mixtures. It was discovered that an exclusive value for this parameter does not exist, as factors such as compaction influence the equivalent fines content which relates to optimum mix performance. The work of Hudson and Davies demonstrated that the minimum VMA of the gradations which were assessed corresponded to 12%, at equivalent fines contents of between 18% and 22% (Figure 3.11). The relationship between the grading exponent and the equivalent fines content is shown on Figure 10.23, for a maximum particle size of 28mm and a filler content of 6%. The equivalent fines contents of 18% and 22% from Hudson and Davies correspond to grading exponents of between 0.4 and 0.5, corroborating the work of Fuller. Figures 10.24 and 10.26 illustrate the effect of the equivalent fines contents of the gradations considered in the Nottingham design method, on the VMA's of the resultant specimens using the Californian granite aggregate. The minimum VMA typically occurs at an 'EFC' of approximately 17%, which equates to a grading exponent of around 0.53. This is a significant conclusion, as it indicates that for this aggregate type and range of binder contents, the densest mixtures are achieved using an aggregate grading which is quantifiably coarser than the Fuller curve. This deduction remains unchanged for variations in compactive effort and binder content. Therefore it would appear that the classification of a gradation using the concept of an equivalent fines content, incorporates aggregate packing characteristics and mixture compactability, and could be used as a guideline for specifying maximum density gradings for different aggregate types.

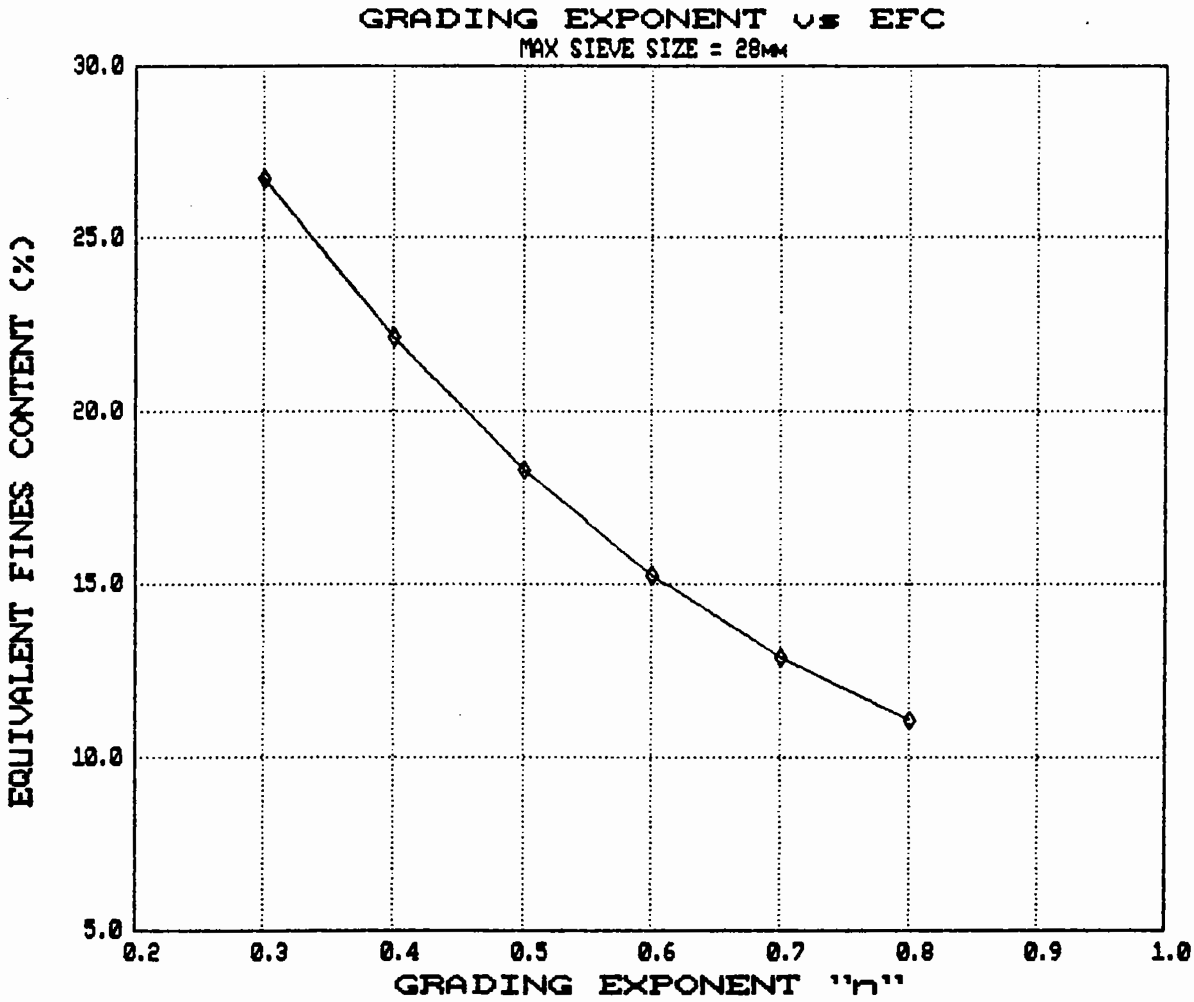

Figure 10.23 The relationship between grading exponent and equivalent fines content

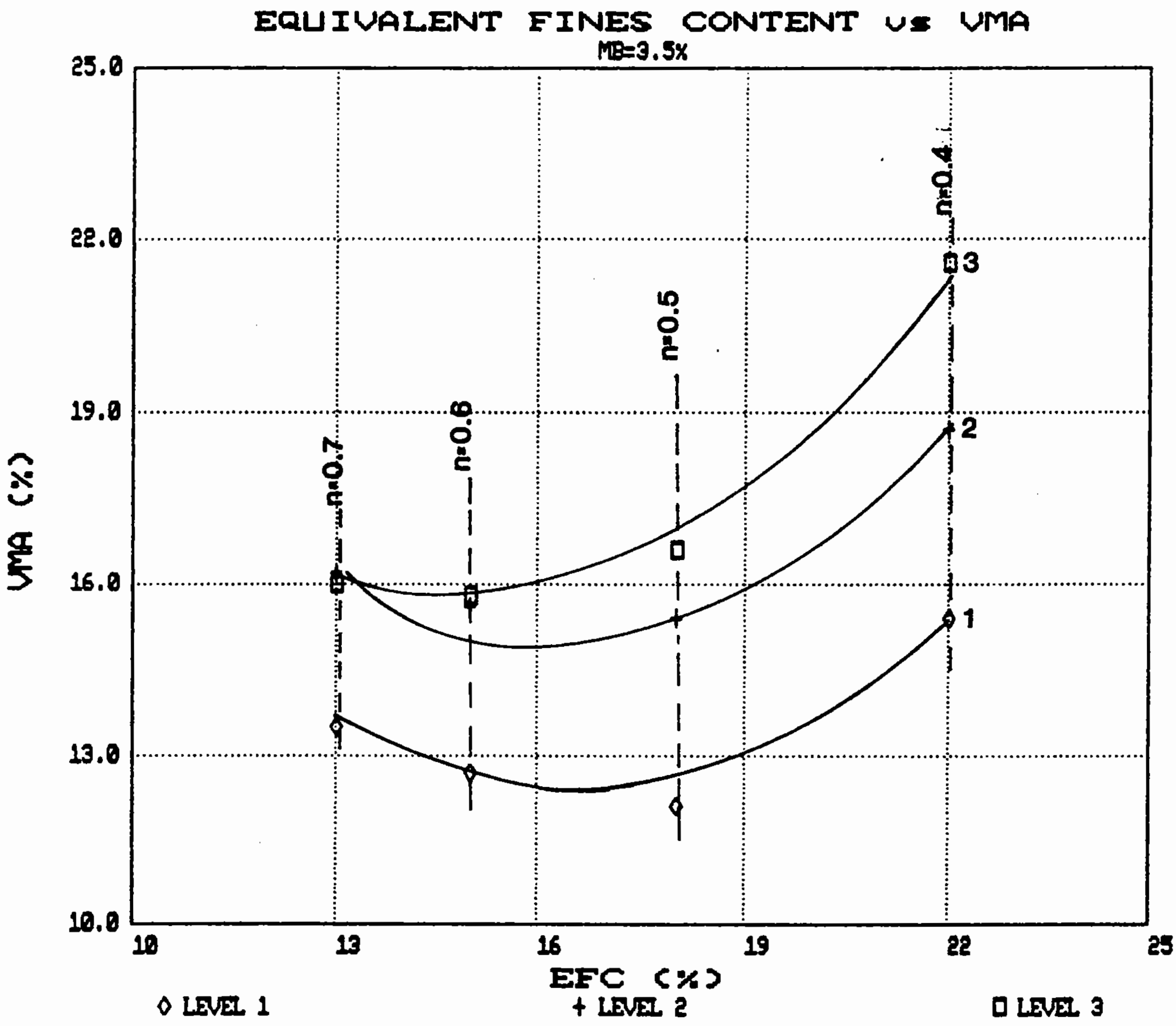

Figure 10.24 The effect of variations in EFC on the VMA of granite specimens (M_B=3.5%)

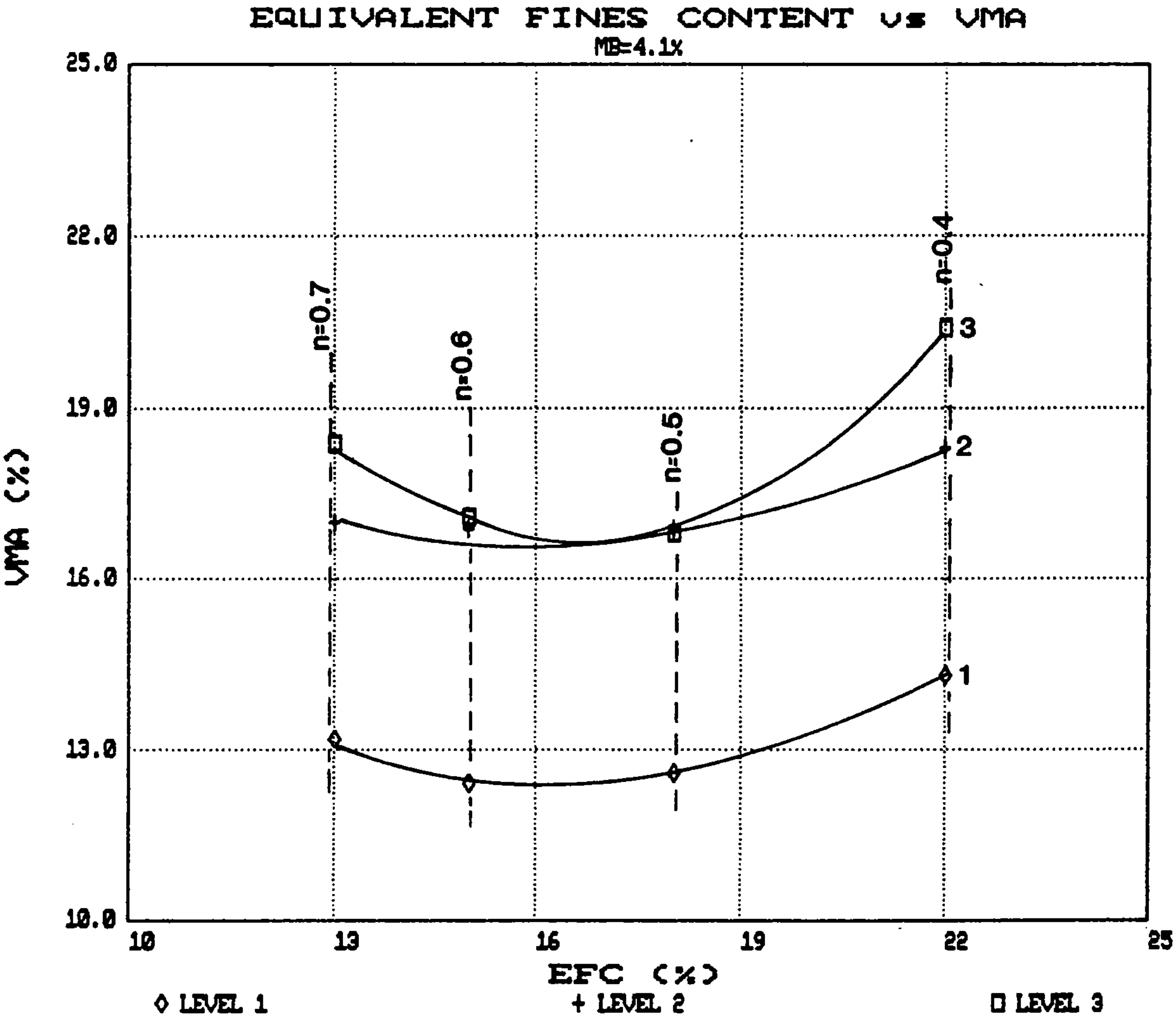

Figure 10.25 The effect of variations in EFC on the VMA of granite specimens (M_B=4.1%)

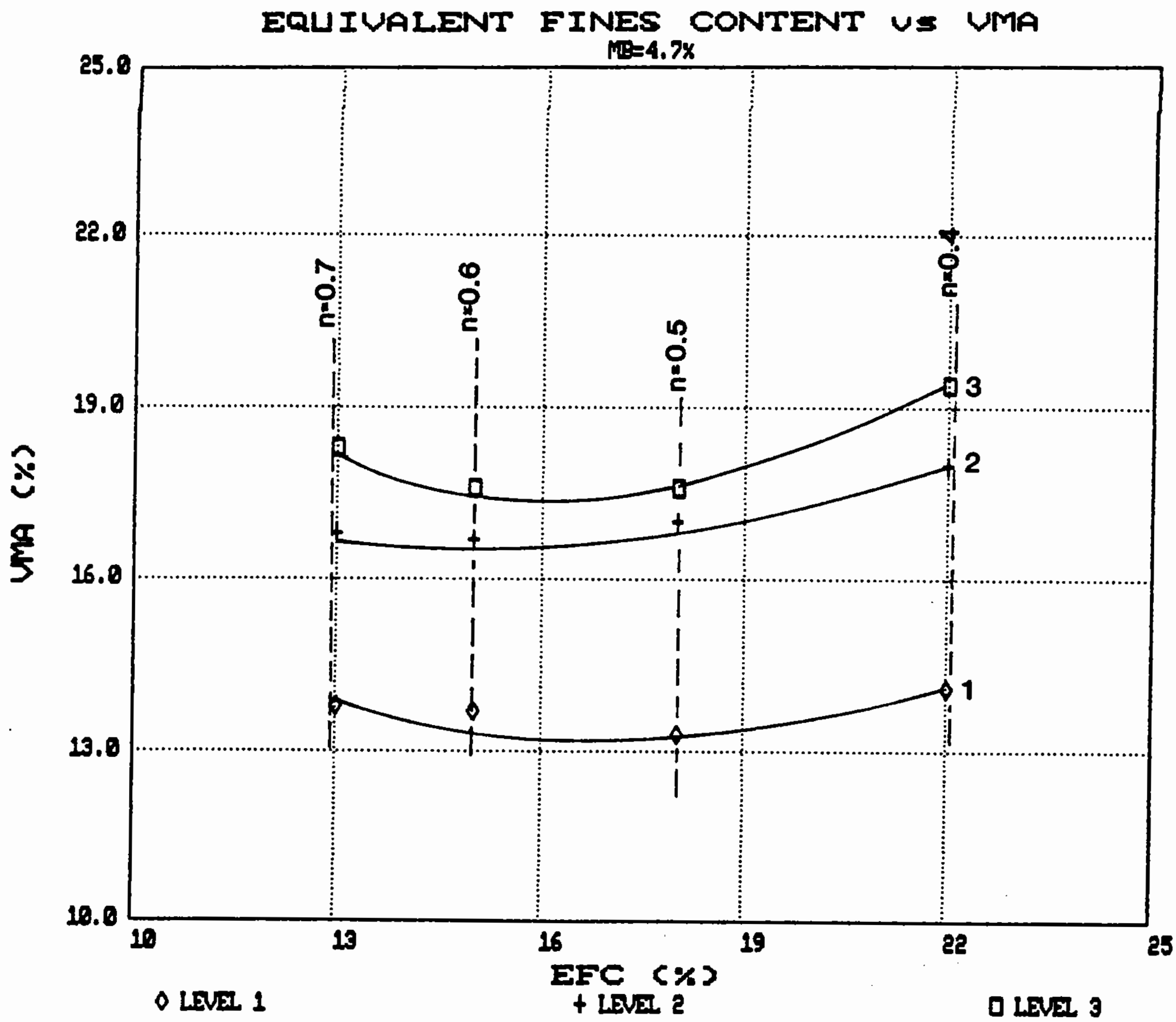

Figure 10.26 The effect of variations in EFC on the VMA of granite specimens (M_B=4.7%)

10.5 WHEEL TRACKING INVESTIGATION

A brief investigation was performed using the wheel tracking apparatus, described in section 6.2.3.2, to compare the findings of the design procedure using an alternative test method. Although the wheel tracking test is not a fundamental test, it has gained popularity amongst highway engineers due to the physical, and visual resemblance to the action of a vehicle tyre traversing a section of pavement.

The exercise carried out was intended to obtain the rutting characteristics of the $n = 0.5$, $M_B = 3.5\%$ mix formulation from the Nottingham design, and the $n = 0.4$ (medium gradation) $M_B = 5.1\%$ mixture advocated from the SHRP investigation. Specimens for the wheel tracking test were manufactured using Nottingham University's rolling compaction facility, (section 5.4), and wheel tracking tests were performed at 40°C.

The results of the tests are shown in Figures 10.27 and 10.28, where the progression of rut development for each specimen is displayed on a deformation/load curve. The results show that the SHRP mix formulation has a substantially greater resistance to permanent deformation than the Nottingham designed material, which contradicts the findings from the mix design procedures. However, the mechanical properties of bituminous coated materials are related to the volumetric composition of the mixture, so for a given aggregate type, similar volumetric proportions must be achieved in order to validate comparisons.

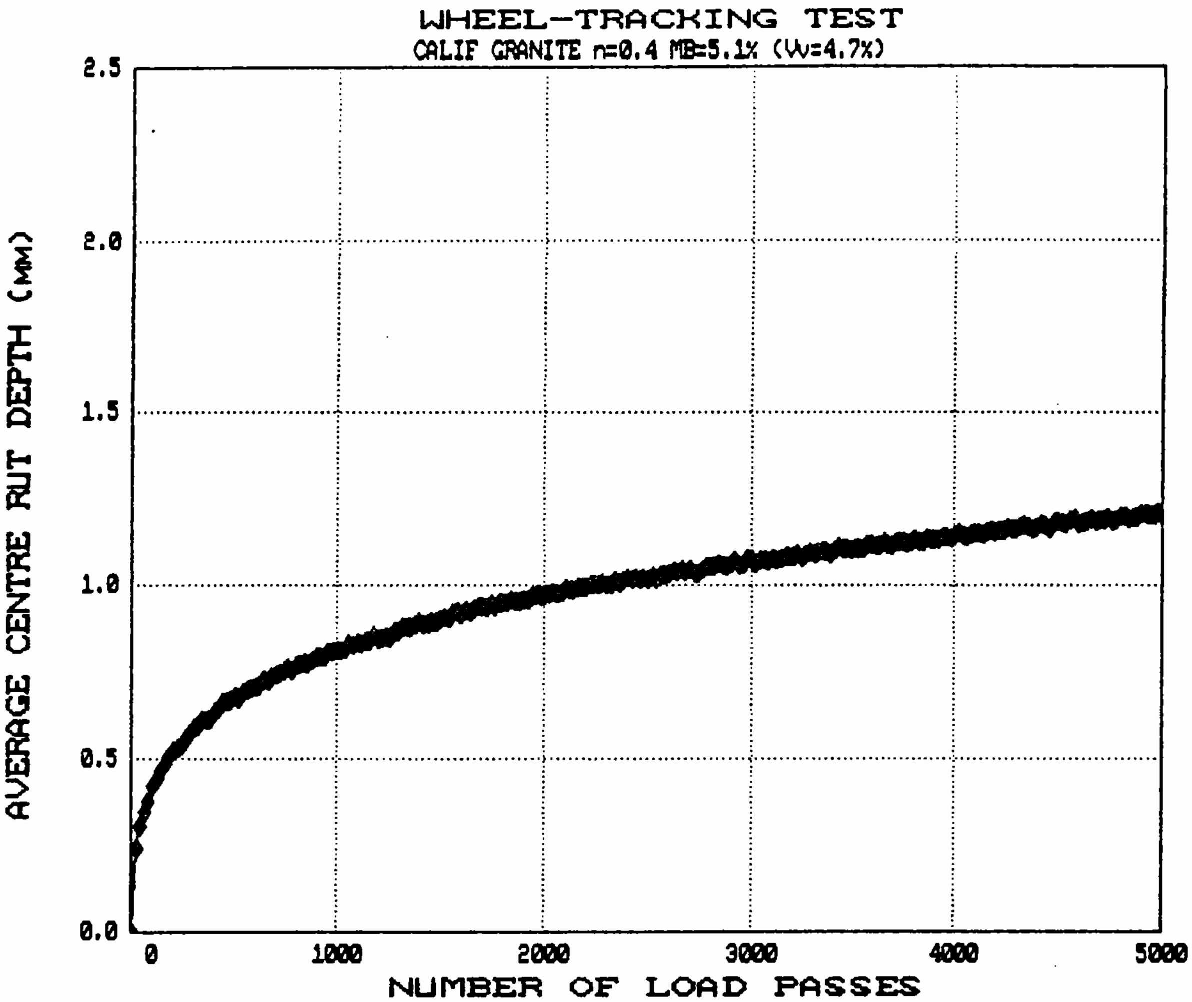

Figure 10.27 Rut depth development during wheel tracking test. Mix formulation corresponds to the SHRP recommendations.

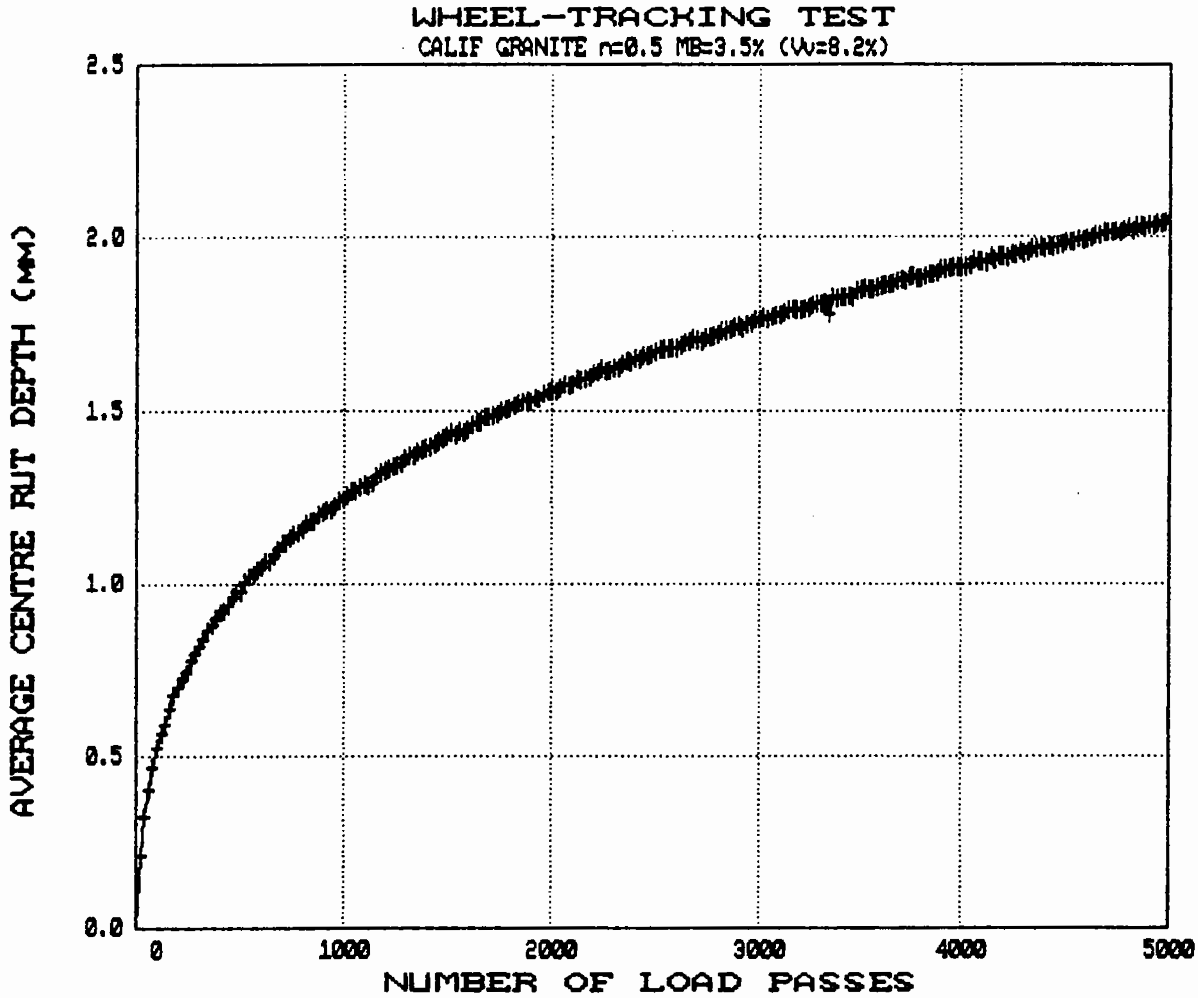

Figure 10.28 Rut depth development during wheel tracking test
Mix formulation corresponds to n=0.5, M_B=3.5%, low compaction level

A series of five cores were taken from each roller compactor slab after the wheel tracking tests had been performed. Densities of the cores were obtained by gravimetric means, and volumetric proportions subsequently calculated, values of which are displayed in Tables 10.16 and 10.17.

Table 10.16 Volumetric proportions of cores taken from n = 0.5 M_B = 3.5% specimen

Specimen	Density (g/ml)	V_V (%)	V_B (%)	VMA (%)	PRD (%)
0.5A	2.433	7.8	8.3	16.2	95.6
0.5B	2.388	9.5	8.2	16.7	93.6
0.5C	2.408	8.7	8.3	17.0	94.4
0.5D (rut)	2.424	8.1	8.3	16.4	95.0
0.5E	2.458	6.9	8.4	15.3	96.3
Average	2.422	8.2	8.3	16.5	94.9

Table 10.17 Volumetric proportions of cores taken from n = 0.4 (Medium Gradation) M_B = 5.1% specimen

Specimen	Density (g/ml)	V_V (%)	V_B (%)	VMA (%)	PRD (%)
0.4A	2.440	5.1	12.2	17.3	96.6
0.4B	2.447	4.8	12.2	17.1	96.9
0.4C	2.470	3.9	12.3	16.3	97.8
0.4D (rut)	2.450	4.7	12.3	17.0	97.0
0.4E	2.443	5.0	12.2	17.2	96.7
Average	2.450	4.7	12.2	16.9	97.0

"rut' denotes a core taken from the wheel track

The average void content of the Nottingham slab is 8.2%, compared with 4.7% for the SHRP specimen, with corresponding percentage refusal density values of 94.9% and 97.0%. Both mix formulations have been shown to exhibit improved mechanical properties with increasing level of compaction, particularly the $n = 0.4$ $M_B = 5.1\%$ mixture, which may be used to justify the difference in the deformation resistance characteristics exhibited in the wheel tracking test.

This observation highlights one of the problems associated with the use of larger specimens and that is the control, and reproduction of volumetric proportions. The slabs manufactured in the roller compactor facility are fabricated to a volume, theoretically controlling the volumetric format of the material. The work has shown that material density and volumetric composition varies within the specimen, which although reflects the situation in the field, causes problems in laboratory analyses and fundamental research work generally. This creates an inconvenient situation when dealing with specimens of material which require 25 kg of aggregate, and considerable laboratory effort for manufacture.

An adjustment to the volume to which the specimens are compacted, allowed the fabrication of another slab at the $n = 0.4$ $M_B = 5.1\%$ designation, which exhibited and average void content of 6.5% and PRD of 95.2%. The wheel tracking test results of this slab are shown in Figure 10.29, where a rut depth of approximately 2.4mm is exhibited, compared with 2.0mm SHRP mix formulation at an average void content of 4.7%, Figure 10.27.

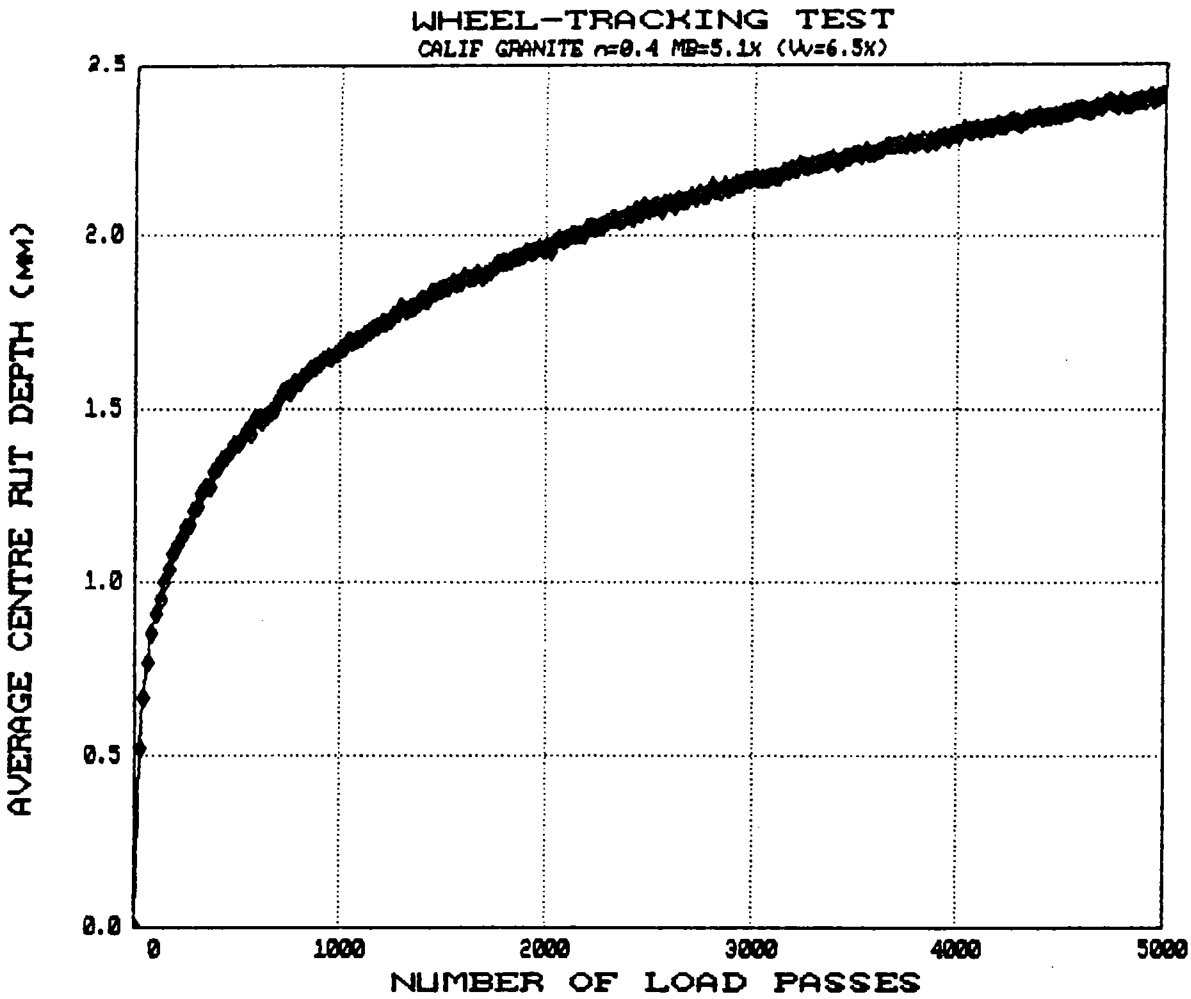

Figure 10.29 Rut depth development during wheel tracking test

Second slab corresponding to the SHRP recommendations.

A comparison between Figures 10.27 and 10.29 appears appropriate, given the relative states of compaction of the two specimens. The results show that specimen n = 0.5, M_B = 3.5% has a slightly improved resistance to permanent deformation compared with the n = 0.4 (medium gradation) M_B = 5.1% mix formulation, when tested under identical conditions. This finding was predicted from the approach used in the mix design procedure.

CHAPTER ELEVEN
DURABILITY TESTING

The previous three chapters have included the results of the new mix design procedure applied to three different types of aggregate, which have led to recommendations for mix formulations based on the fundamental engineering properties of bituminous materials. The mechanical properties associated with bituminous materials were introduced in chapter 4, and were classified as the following parameters:

Elastic Stiffness
Fatigue Strength
Resistance to permanent deformation
Durability
Workability

The mix design procedure evaluates mix formulations through quantification of the elastic stiffness, deformation resistance and fatigue strength, and selects an optimum mixture based on these parameters. This methodology should satisfactorily lead to the optimisation of resources, and a good material performance in the early service life of the pavement. However, it is necessary to introduce some means by which the engineer can assess the ability of the material to retain the design values of the aforementioned parameters throughout its' service life. This means that in addition to the assessment of the fundamental mechanical properties, the durability of the material must also be quantified.

Bituminous materials are exposed to various environmental factors which contribute to a deterioration of the structural integrity of the mixture. Long term ageing, primarily through oxidation of the bitumen, is a phenomenon which causes the binder to harden, increasing the pen value and the penetration index. In an advanced state, this can reduce the cohesive strength of the bitumen and lead to fracturing of the asphalt mortar. The effect of ageing can be minimised by densification of material and thick binder films. This has been addressed within the design procedure through recommendations of aggregate gradations and minimum binder volumes.

Roadbase materials only receive direct exposure to air during the construction phase, before a wearing course is laid and compacted on top of the base layer. A more aggresive and damaging agent leading to durability problems with roadbase mixtures is moisture. Degradation of an asphaltic concrete due to moisture related problems principally occurs through two mechanisms:

a) loss of cohesive strength and stiffness in the bitumen film
b) failure of the adhesive bond between the aggregate and bitumen

The first mechanism can be due to several factors such as hardening of the bitumen and physicochemical effects, and will result in an overall reduction in mixture stiffness, and therefore fatigue strength. The second mechanism occurs when the aggregate has a preference for absorbing water, which can lead to 'stripping' problems and premature pavement distress. The extent of environmental effects on a roadbase material are not as severe as those experienced by wearing course mixtures, but nevertheless must be quantified and evaluated in terms of a mix design procedure. It is not uncommon for roadbases to be drenched with water prior to the laying of base and wearing courses, which effectively locks any residual moisture inside the pavement.

A bituminous mixture is classified as being susceptible to moisture related damage if a sample specimen fails a 'moisture sensitivity' test, which implies that the material would fail prior to reaching its design life due to moisture related mechanisms.

11.1 REVIEW OF TEST METHODS

Methods of evaluating the moisture sensitivity of bituminous mixtures may be divided into two categories

i) Tests where an uncompacted, coated mixture is immersed in water for a given duration at a given temperature, and a visual determination is made of the separation of the binder from the aggregate.

ii) Tests which use compacted specimens, which may be road cores or laboratory prepared samples, and condition the specimens by simulating in-service conditions. Results are usually expressed as ratios of conditioned to unconditioned values of elastic stiffness, or another strength parameter.

The indirect tensile test is a popular method to obtain a ratio of retained strength between a conditioned and unconditioned specimen. The

AASHTO T-283 procedure (102) involves soaking the Marshall briquette sized specimens under a partial vacuum for five minutes prior to sealing the sample in film and leaving in a temperature controlled environment of -18°C for sixteen hours. At the end of this duration, the specimen is then transferred to a water bath at 60°C for 24 hours which completes the conditioning period. The specimen temperature is then reduced to 25°C for RLIT testing. A retained stiffness index in excess of 0.7 is suggested to indicate that a mix formulation is unlikely to exhibit distress through moisture related mechanisms (103). This intensive conditioning of specimens includes the freeze-thaw effect after forcing moisture into the sample using an environmental chamber.

An alternative procedure is the 'Immersion compression test', (104) which is currently used by many agencies in the U.S.A.. This test evaluates the loss of adhesion or cohesion by immersing the specimens in water at an elevated temperature and then measuring the retained compressive strength through a test arrangement as shown in figure 11.1. The Asphalt Institute recommends that the index of retained strength should be greater than 75%.

Other states employ boiling water tests to identify moisture susceptible mixtures, in which an uncompacted mix is placed in a container of boiling water for between 1 and 10 minutes, with the damage analysis evaluating the stripping potential of the mixture relying on a visual assessment. Evidence has shown that this approach provides a poor guideline for the durability of the mix formulation (105), and improvements to the procedural evaluations do not arise from extending the duration of boiling.

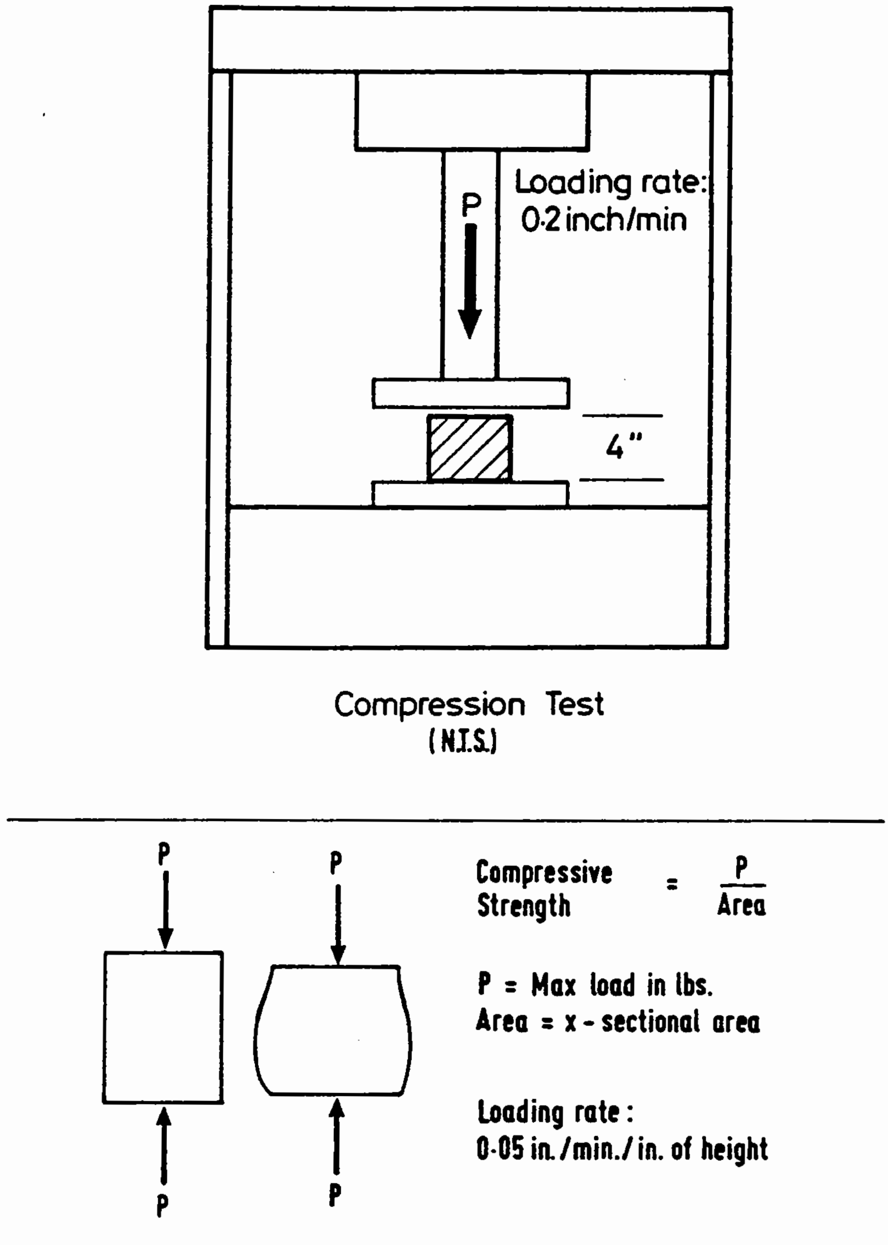

Figure 11.1 Configuration for compression test following immersion in water. After SHRP summary report on water sensitivity A/1R-89.003) (re 107)

Variations to the three test methods exist, including a retained strength index test using the Marshall apparatus, but no test has been shown to be preferable over the other alternatives for accurate prediction of moisture sensitive mixtures.

In the context of a mix design procedure, a check is required on the preliminary design mix to assess the potential of moisture sensitivity. This necessitates a rapid and accurate test, which may indicate amendments to the mix formulation, should failure in such a test occur.

11.1.1 A retained stiffness index

The mix design procedure which has been developed within this project has addressed problems pragmatically, in an effort to produce a rational and implementable design method for dense bituminous concrete. The inclusion of a durability test, therefore, must adhere to the same format. The design procedure has introduced the NAT as a comprehensive test apparatus for bituminous materials, which incorporates a facility for elastic stiffness testing, repeated load axial and static load deformation tests. A durability test should use the NAT in order to maintain a high level of consistency with the procedural format. From a review of the test methods, a measurement of retained strength of a specimen, after a conditioning period, would appear the most appropriate evaluation of mix durability, and the use of the RLIT test would facilitate such an assessment.

The conditioning of test specimens is not specific, but a period of immersion in water at an elevated temperature would appear to identify moisture sensitive mixtures from an analysis of a retained stiffness index.

11.1.2 Application within a mix design procedure

A check on the durability of a mix formulation would constitute the ultimate step in the mix design procedure. From the results of the fundamental mechanical tests of elastic stiffness and resistance to permanent deformation, and the estimate of fatigue lives using a prediction model, a mix formulation optimising the available resources may be identified. It is this mix formulation which must be evaluated in terms of moisture sensitivity. The result of a durability test would establish whether the mixture was satisfactory to recommend as the design mix, or whether adjustments to the mix composition would be required to eliminate the possibilities of mixture deterioration through moisture related mechanisms. In this case any changes to the mix formulation must be performed with a considered evaluation of the implications to the fundamental mechanical properties.

11.2 AN INVESTIGATION INTO MOISTURE SENSITIVITY

An investigation into an index of retained elastic stiffness was performed on bituminous mixtures containing two types of aggregate. Limestone from Dene Quarry, and a chert gravel from Texas, selected from the SHRP library of materials, were assessed. The conditioning period of the specimens comprised immersion in a water bath at 60°C for 24 hours, followed by a further 24 hour submergence at 20°C. RLIT tests were performed at 20°C.

11.2.1. Limestone aggregate

Specimens were manufactured according to the procedures of the design method, at aggregate gradations corresponding to n=0.5 and n=0.6, and a 70 pen bitumen at binder contents of 3.5%, 4.1% and 4.7%. Mixtures were compacted to the mid-range level. Table 11.1 shows the volumetric proportions of the specimens.

Table 11.1 Volumetric proportions of Limestone specimens

Spec No.	Grading Exponent 'n'	M_B (%)	Compaction Level	Density (g/ml)	V_V (%)	V_B (%)	VMA (%)	VFB (%)	PRD (%)
1	0.5	3.5	2	2.405	4.8	8.6	13.4	64	95.7
2	0.5	4.1	2	2.388	4.6	10.00	14.6	68	95.6
3	0.5	4.7	2	2.401	3.2	11.5	14.7	78	96.7
4	0.6	3.5	2	2.400	5.0	8.6	13.6	63	96.0
5	0.6	4.1	2	2.387	4.7	10.00	14.7	68	95.6
6	0.6	4.7	2	2.413	2.7	11.6	14.3	81	96.6

The VFB, (voids filled with binder) parameter, represents the quotient of V_B divided by VMA, and hence gives an indication of the binder film thickness on the aggregate particles which is related to the durability of the mixture. RLIT tests were performed initially on the specimens, then immediately after the conditioning period, and a final series of tests were carried out having allowed the samples to dry after conditioning. Results are displayed in Table 11.2.

Table 11.2 Elastic stiffness values of Limestone specimens before and after moisture conditioning

Spec No	Mix formulation	Initial Elastic Stiffness (MPa)	Post-conditioning Elastic stiffness (MPa)	Final Elastic Stiffness (MPa)
1	0.5, 3.5,2	10000	10 000	8500
2	0.5,4.1,2	6500	7500	6500
3	0.5,4.7,2	3800	4300	4000
4	0.6,3.5,2	7000	8000	7500
5	0.6,4.1,2	5800	6200	6000
6	0.6,4.7,2	5400	6700	5000

Allowing for tolerances on results which occur with the RLIT test, there does not appear to be a discernible reduction in the values of elastic stiffness immediately after the conditioning period, or when the specimens have dried after the soaking process. Therefore it would appear reasonable to conclude that this type of aggregate is not susceptible to distress through moisture related mechanisms, irrespective of the binder content of the mixtures. The results corroborate the hypothesis that limestone offers very good cohesive properties with bitumen, as this type of aggregate is not noted for stripping failures.

11.2.2 Texas aggregate

A second exercise was performed using a Texas chert and a Boscan asphalt, with both materials taken from the SHRP investigations. The aggregate is a rounded river gravel, and was selected to be included within the SHRP effort, as it is known to be a poor aggregate for use in pavement engineering. Specimens were manufactured over the range of binder contents at gradations of n=0.5 and n=0.6. Volumetric proportions of the specimens are shown in Table 11.3.

Table 11.3 Volumetric proportions of chert specimens

Spec No.	Grading Exponent 'n'	MB (%)	Compaction level	Density (g/ml)	V_V %	V_B %	VMA %	VFB %
1	0.5	3.5	1	2.426	1.7	8.9	10.6	84
2	0.5	3.5	2	2.329	5.6	8.5	14.1	60
3	0.5	4.1	2	2.329	4.4	10.0	14.4	69
4	0.5	4.7	2	2.362	2.5	11.6	14.1	82
5	0.6	3.5	2	2.326	5.7	8.5	14.2	59
6	0.6	4.1	2	2.335	4.5	10.0	14.5	69
7	0.6	4.7	2	2.343	3.3	11.5	14.8	77

The moisture conditioning period of the specimens was the same as that applied to the limestone samples. Only one RLIT test was performed immediately after the conditioning sequence on each specimen, the results of which are shown in Table 11.4.

Table 11.4 Elastic stiffness values of chert specimens before and after moisture conditioning

Spec No.	Mix formulation	A: Initial Elastic Stiffness (MPa)	B : Post Conditioning Elastic Stiffness (MPa)	Retained stiffness index (B/A)
1	0.5,3.5,1	14000	11500	0.82
2	0.5,3.5,2	7300	4500	0.62
3	0.5,4.1,2	6800	4700	0.69
4	0.5,4.7,2	5500	5500	1.00
5	0.6,3.5,2	4500	2600	0.57
6	0.6,4.1,2	6000	4500	0.75
7	0.6,4.7,2	3400	3300	0.97

With the chert aggregate, the influence of moisture conditioning can be seen to adversely effect the elastic stiffness of the specimens. Two conclusions may be drawn from the results.

1. Increasing the binder content increases the retained stiffness index.
2. Increasing the compaction of the specimen increases the retained stiffness index.

The mixtures with a binder content of 3.5% compacted at the intermediate level, exhibit the greatest reduction in elastic stiffness, with a negligible depreciation occurring when the binder content is increased to 4.7%
The VFB parameter appears to be a relevant indicator of moisture sensitivity for the chert mixes. A VFB over 75% corresponds to a high value of a retained stiffness index, which suggests that distress through

moisture related mechanisms is unlikely in such mixtures. However, comparison with the results obtained from the limestone mix formulations shows that durability problems are associated with the interaction between binder and aggregate type, and not simply the volumetric proportions of the mix, and must be evaluated, therefore in terms of a form of mechanical test.

From the brief investigation performed, a tentative suggestion of a retained stiffness index in excess of 80% could be used to identify mixtures which are unlikely to exhibit durability problems.

11.3 MECHANICAL PROPERTIES OF MIXTURES

The effect of using a hard grade of binder in a bituminous mixture is to increase the elastic stiffness of the mix. A comparison of table 11.2, with tables 9.4 and 9.5 demonstrates the change in the measured elastic stiffness values of the same aggregate type and gradation when mixed with different grades of binder.

Figure 11.2 shows the resistance to deformation characteristics of the limestone mix formulations compacted at the intermediate level. Taking account of the volumetric proportions of the mixes, there does not appear to be a significant improvement in the resistance to permanent deformation of the specimens manufactured using a 70pen binder, compared with mixes investigated in Chapter 9 which comprised a 100 pen binder.

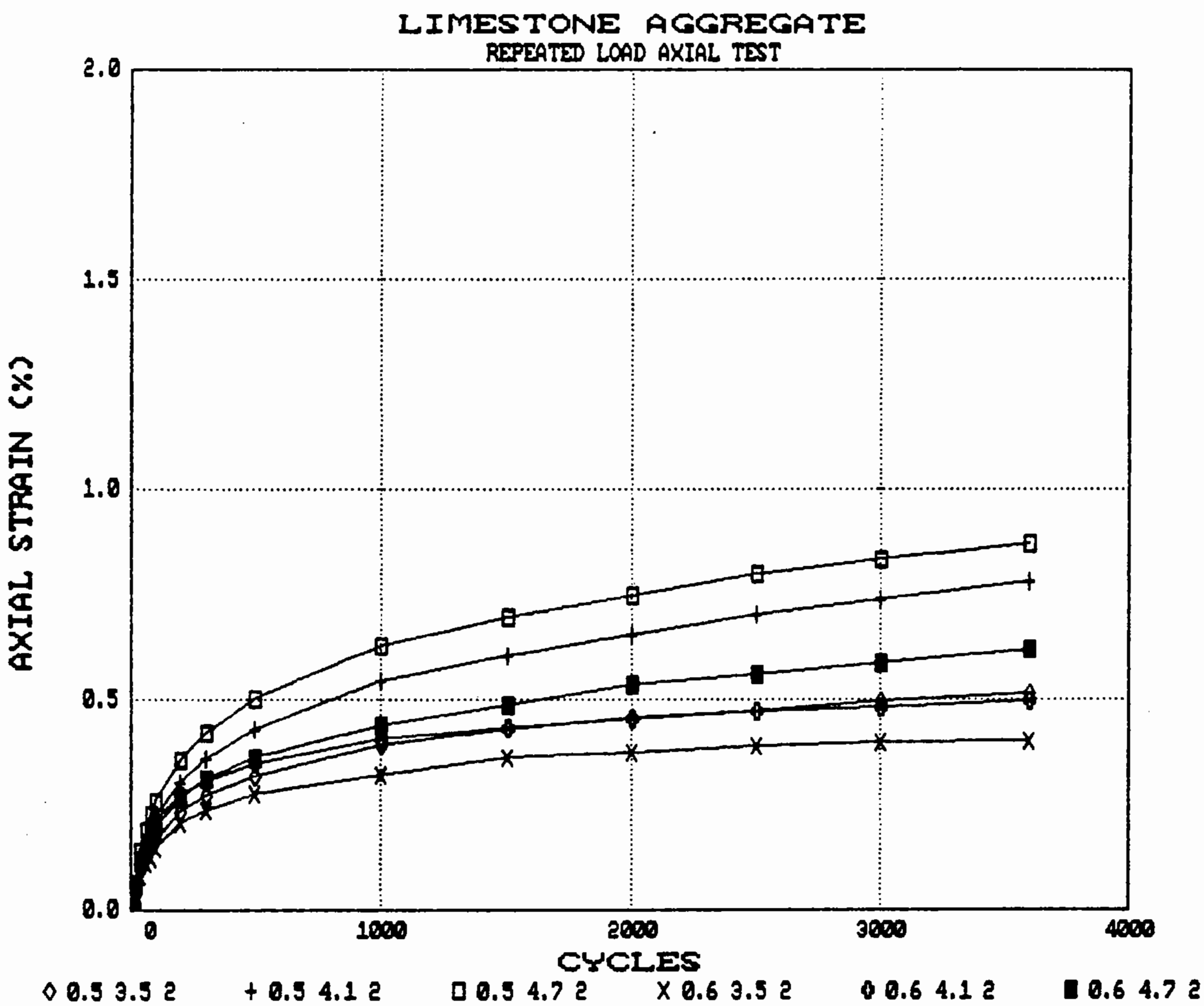

Figure 11.2 Deformation curves plotted from the results of RLA tests. Limestone aggregate, selected mix formulations from the mix design recommendations.

The results of the RLA tests on the chert specimens are illustrated in figure 11.3. The effect of a rounded, smooth textured aggregate on the deformation resistance qualities of a mixture are clearly indicated, with only one mix type at the intermediate compaction level exhibiting satisfactory properties. The change of grading exponent from n=0.5 to n=0.6, increasing the void content of the mixtures, appears to have a considerably greater influence on the resistance to permanent deformation than with mixes comprising angular aggregates.

11.4 DISCUSSION

The durability of a bituminous mix formulation is an important parameter which must be quantified in order to assess the long term performance of the mix. Quantification must be performed by applying a mechanical test prior, and subsequent to a conditioning phase which can simulate and accelerate the environmental distress modes exhibited in the field. A criterion must then be applied to the value of a retained property index in order that mixtures which may exhibit durability problems can be identified. The exercise carried out on the different aggregate types indicated that durability problems are a function of the aggregate-asphalt interaction, and certain aggregate types will be susceptible to distress modes associated with environmental factors. The volumetric proportions of such mixtures influence the magnitude of the reduction in a property such as the elastic stiffness, with minimal effect occurring with binder rich, well compacted mixtures, and a potentially substantial reduction in material performance of lean mixtures with high void contents.

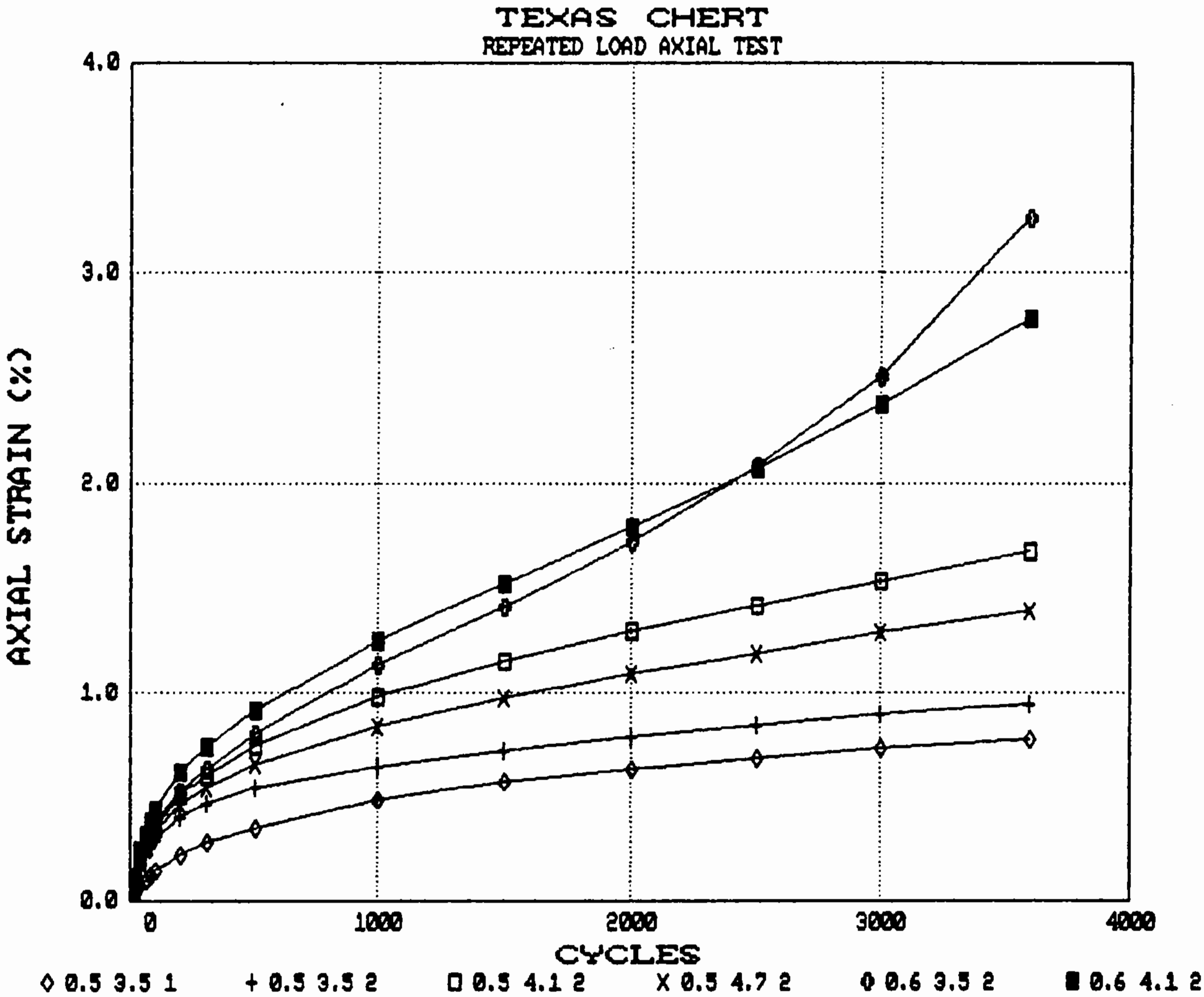

Figure 11.3 Deformation curves plotted from the results of RLA tests. Chert aggregate, selected mix formulations from the mix design recommendations.

Should durability be identified as a potential problem with the mix formulation which results from the design procedure, it may necessitate changes to the composition of the mixture, in order to eliminate or minimise such problems. Changes to mix composition may include increasing the binder content, adjusting the aggregate grading, or a combination of both. Whatever changes are implemented, the mechanical properties of the amended mixture must comply with the criteria applied to the values of elastic stiffness and resistance to permanent deformation, as recommended by the design procedure. An extensive understanding of the influences on mix properties for the full range of mix formulations is required in order to assess the implications of minor alterations to mixture composition. The rational format which has been adopted for the mix design procedure should facilitate this understanding, and enable engineering-based decisions to be made with respect to amending the 'design mix' should durability problems be manifested.

CHAPTER TWELVE

CONCLUSIONS

12.1 INTRODUCTION

The aim of this project was to develop a rational procedure for the design of dense bituminous concrete mixtures. The method which has evolved, addresses the problem in a systematic way, and quantifies material performance through a series of mechanical tests. The results of the investigations which have been performed indicate that the mix formulations which exhibit optimum mixture performance for roadbase materials, satisfy current U.K. specification. However, the existing British Standard for coated macadams, BS 4987, also incorporates substantially inferior mixtures within the wide tolerances allowed on binder contents and aggregate gradings. Therefore, a design procedure is required to ensure that available resources can realise their maximum potential, and resulting materials may be evaluated in terms of engineering properties.

12.2 PRINCIPAL CONCLUSIONS

1. A mix design procedure for roadbase and basecourse layers of flexible pavements has been developed. A flow chart for the procedure is shown in figure 12.1.

2. The performance characteristics of bituminous materials should be quantified according to the properties of:
   elastic stiffness

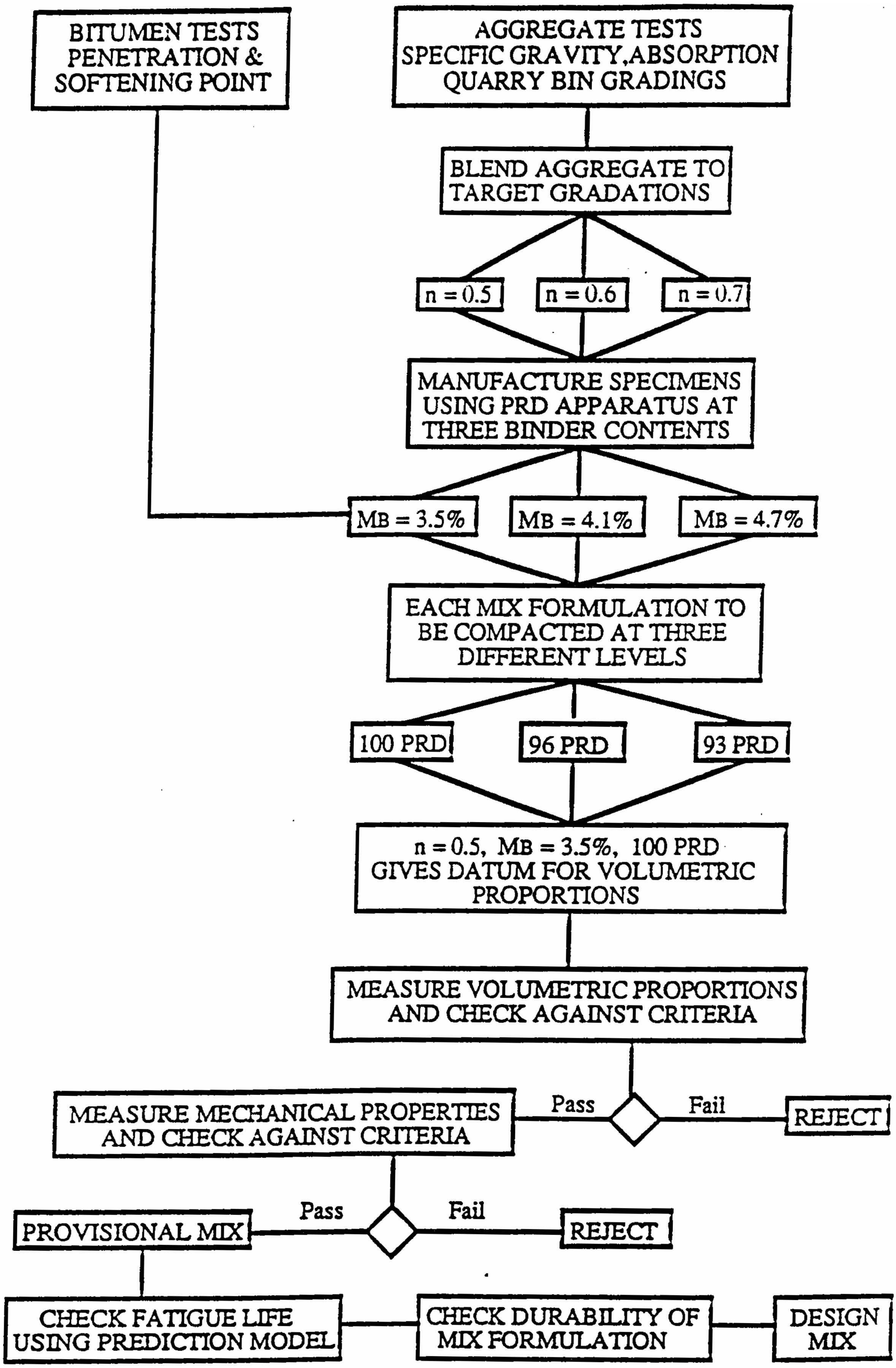

Figure 12.1 The proposed mix design procedure.

resistance to permanent deformation

resistance to fatigue cracking

The properties of elastic stiffness and deformation resistance should be evaluated through the testing of samples of representative material.

3. The properties of bituminous concrete, as used in the main structural layer of flexible pavements can be improved through rational mix design.

4. A method of specimen manufacture which produces material representative of that laid on site is fundamental to a successful mixture design procedure.

5. The Nottingham Asphalt Mix Tester provides a satisfactory facility for the measurement of the mechanical properties of elastic stiffness and deformation resistance.

12.3 GENERAL CONCLUSIONS

1. The use of dry aggregate compaction tests, as performed within this investigation, suggested that this type of approach is an inappropriate means by which to design aggregate gradings. The orientation and packing of the stone particles in a bituminous mixture could not be reproduced using these tests.

2. The gradations of the quarry bin stockpiles must be carefully monitored, in order that the closest approximation to target

gradations can be achieved. Particular attention should be given to the finest, or dust material, as this size fraction constitutes approximately 30% of the grading.

3. The Percentage Refusal Density test apparatus can be used as a facility to manufacture specimens of bituminous concrete. Use of the small diameter compaction foot introduces a kneading action during the compaction operation, which is fundamental to the laboratory reproduction of site material.

4. PRD values should always be quoted with associated values of volumetric proportions.

5. The manual operation of compaction does not appear to significantly effect the 100 PRD level of compaction. It would be possible to standardize compactive efforts by weighting the hammer and automating the compaction sequence.

6. A new approach to the assessment of mix proportions has been developed, which takes account of the packing characteristics of different aggregate types.

7. The maximum specific gravity of a paving mixture should be calculated using the 'Rice method' in order to eliminate the effects of variations in aggregate densities.

8. The calculated values of the volumetric proportions are influenced by the adherence to procedures when using gravimetric means to

evaluate material densities. The use of a sealing agent on moulded surfaces of bituminous specimens may result in distorted values.

9. The recommended criteria applied to the volumetric proportions of roadbase mixtures should be considered as guidelines and not absolute values.

10. Allowable tolerances on the level of compaction of bituminous concrete should be specified. The compactive effort required to achieve a desirable level of compaction can vary according to the aggregate type used within the mix.

11. Inter-particle friction effects the compactibility of a mixture for a given compactive effort.

12. The elastic stiffness of a bituminous material is a major influence on the in-situ fatigue performance of the mixture, as the magnitude of tensile strain on the underside of the layer is dictated by this parameter.

13. An increase in the filler content of the mixture corresponds to an increase in the elastic stiffness, as the filler material combines with the binder to produce a stiff mortar. This also has the effect of reducing the compactability of the material.

14. The resistance to permanent deformation is primarily influenced by aggregate gradation and level of compaction. A satisfactory binder content must be identified in order to mobilise the gradation to

maximum potential. Small variations in binder grade do not appear to effect deformation resistance if the other parameters are optimised.

15. The evaluation of results from a deformation test, (static creep or R.L.A), should include an assessment of the shape of the deformation curve, and the ultimate value of strain induced at the termination of the test.

16. The durability of a bituminous mixture appears to be related to asphalt-aggregate interactions. Volumetric proportions have a secondary influence which may accelerate or retard durability associated problems.

CHAPTER THIRTEEN

PRACTICAL IMPLICATIONS OF THE RESEARCH

Implementation of the findings of research into the 'real world', regardless of discipline, has been shown to be one of the greatest hurdles associated with innovative developments. Research often fails to change or modify current practice because of limited understanding, and general inertia within the market to which the new ideas pertain. The highway industry within the United Kingdom exemplifies this reluctance to accept change, and despite the trend of increasing wheel loads and traffic densities, the approach to road construction at the beginning of the 1990s is not dissimilar to that of the late 1950s, when the motorway programme commenced.

However, the empirical approach does not facilitate any engineering analysis of pavement design or material performance, and there now exists a general acceptance that current specifications are inadequate for addressing the demands to which modern highways are subject.

13.1 END PRODUCT SPECIFICATIONS

For bituminous materials which are used in flexible highway construction, compliance with a recipe specification does not give an indication of material performance, and is therefore an inadequate means of controlling these materials in terms of their engineering properties. It is important that all bituminous materials in highway engineering are quantified according to the salient properties which dictate field performance. The development of the NAT has provided an operator-friendly, comprehensive test apparatus, which facilitates rapid testing of site cores and laboratory prepared specimens, to obtain values of the fundamental properties. Criteria must be applied to these values to identify satisfactory, and potentially unsatisfactory mix formulations in terms of field performance. In order to finalise the acceptance criteria for the different asphaltic materials in a pavement, a data bank must be generated from information taken from existing materials which have exhibited both good and moderate performance. Such a collation of information will require extensive cooperation from local highway authorities and the Department of Transport to provide cores of material for this purpose. Furthermore, a general investment in testing apparatus such as the NAT, is required amongst material suppliers and local authorities, in order to facilitate the necessary testing.

13.2 MIX DESIGN

Consistent production of material to meet an end product specification throws greater responsibility onto the material supplier and the surfacing contractor. This requires an optimisation of resources, and a high level of workmanship on site. In order to ensure that material production can be maintained to satisfy performance based criteria, mix design procedures should be employed to optimise the use of available material. The design method which has evolved from this project for roadbase materials is practically based, and should be capable of implementation within U.K. and overseas practices. The PRD apparatus comprises the equipment required for specimen manufacture, which was selected as it is a familiar tool in highway laboratories and therefore does not need an introduction phase. As previously stated, test equipment, such as the NAT, must be purchased by agents in order to perform material evaluations. It must be stressed that this statement does not represent a chimaeric dream, as there now exists sufficient awareness of the need to quantify bituminous materials according to their engineering properties, and several agencies have acquired NATs in preparation of introducing performance based specifications.

The design procedure recommends a particular mix formulation based on a specific aggregate gradation and binder content. Plant production of such a mix will introduce variations to both of these parameters, which in turn will change the mechanical properties of the compacted mixture. Selection of the design mix must be made with consideration of tolerances to the mechanical properties, and the effect of changes within the gradation and binder content. The information obtained from the mix design procedure should facilitate the making of such judgements which result in practical tolerances for the material supplied without threatening specification compliance.

The mix formulations resulting from the research work which have exhibited the optimum mechanical properties have comprised lower binder contents than those typically used in current practice. This would have the effect of reducing the cost of the material, which is a positive development when combined with enhanced mechanical properties.

The design procedure deals with aggregate gradations on the coarse side of the maximum density curve, which may lead to practical problems. Some material suppliers claim that any continuously graded mixture with a coarse aggregate bias will segregate, invalidating the potential benefits. Segregation is likely to occur when the mixed material is dropped from the hopper into the lorry, the degree of which is probably related to the height of the drop. This type of practical problem cannot easily be addressed, but if it proves to be a genuine source of difficulty then it must be overcome by some means.

It is anticipated that during 1991, two full-scale trials of the mix design procedure will be performed, which should highlight any practical problems which hitherto have not been encountered.

CHAPTER FOURTEEN

RECOMMENDATIONS FOR FUTURE WORK

14.1 INTRODUCTION

A format for the rational design of dense bituminous concrete mixtures has been established from the work undertaken during this investigation. The work has been performed with the background of current U.K. practice, but the principles of the method are applicable to bituminous bound roadbase layers throughout the world. The findings of the research have highlighted areas of work which require further investigation in order that improvements or amendments may be incorporated into the procedure.

Practical difficulties with the mix formulations emerging from the design procedure, which to date have not been identified from laboratory oriented work, should become apparent through the performance of full-scale trials. It is anticipated that two such trials should take place in conjunction with Northamptonshire County Council, and Staffordshire County Council during the summer of 1991. The performance of these trials should identify practical problems with the existing design method, indicating possible areas where modifications are required.

14.2 SPECIFIC AREAS OF FUTURE RESEARCH

14.2.1 Material properties

The reproduction of site compacted material under laboratory conditions is a key factor in a mix design procedure. The relevance of methods of specimen manufacture, compaction procedures and resultant material properties, is of paramount importance, and effort must be concentrated in establishing the optimum approach. A specific investigation into the properties of laboratory prepared specimens, compared with site cores of the same generic material must be undertaken to establish the validity of fabrication techniques. Such an investigation requires the cooperation of local Highway Authorities, and a data bank of information concerning the volumetric proportions, mechanical properties and compaction levels of different mix formulations must be generated. This will permit realistic criteria for the mechanical properties to be established, and provide information concerning the performance of materials if such criteria are not met.

14.2.2 Wearing courses

This investigation has been concerned with the development of a mix design procedure for the main structural layer of flexible pavements. The next phase of the work is the modification of the procedure to wearing course mixtures, which in the U.K. include hot rolled asphalts and pervious macadams. Hot rolled asphalt is currently designed by a British Standard version of the Marshall test, which establishes an optimum binder content for a recipe based gap grading. It is very important to introduce performance based specifications to all bituminous highway materials in order to evaluate the merits of different mix formulations in a quantitative manner.

The application of the mix design method to wearing course mixtures will necessitate changes to the criteria applied to both volumetric proportions and mechanical properties, but the procedural format should remain relatively unchanged.

14.2.3 Filler content

The quantity of material passing the 0.075 mm sieve has been shown to influence the elastic stiffness of bituminous mixtures, and also the compactability of these materials. Further work is required in order that a full understanding of the role of filler material can be established, in terms of the quantity used, and the type of filler incorporated into the mixture. This has particular relevance to wearing course mixtures, and should be included within that phase of continuing research, but as discussed in chapter 9, the filler content effects the mechanical properties of continuously graded roadbase materials. The design procedure has been based on a filler content of 6%, but a more comprehensive investigation is required to understand the effects of changes to this value.

14.2.4 Binder modifications

There are a number of additives available to the highway engineer for enhancing the properties of bitumen and bituminous mixtures. These include rubbers, thermoplastic polymers, thermoplastic rubbers and thermosetting compounds, all of which can improve the mechanical properties of mixes by varying degrees. Modified bitumens have specific applications, in mixtures such as pervious macadams, where large areas of bitumen films are exposed to the air and environment, and areas of pavement subject to high stresses where exceptional resistance to permanent deformation or fatigue is required. Climates where thermal cracking of pavements is a major cause of distress, or where extremely high ambient temperatures are prevalent, may benefit from binder modified mixes, but in each case, the merits of using modified bitumens must be assessed on a cost benefit basis.

In order to fully evaluate the benefits of mixtures containing modified binders, a qualitative comparison must be made with conventional material. The testing procedure involved in the mix design method would provide a facility for drawing such a comparison, making an engineering based quantification of the relative merits of each mix type. This would allow a total life costing to be made which would justify or deem uneconomical the use of binder modifications for a given situation.

14.2.5 Recycling of material

As environmental issues gain greater political credence, there is increased pressure to cease the plundering of the earth's resources, and to recycle existing materials. Recyling of highway pavements has seen limited use, mainly through trials, and the long term performance of recycled materials has yet to be established. However, it would be foolhardy to ignore this aspect of highway construction, and the mix design testing procedures could be used to evaluate the quality of material resulting from a recyling process. This work could also encompass an assessment of materials made with cold bitumen emulsions, which again are gaining popularity through environmental pressure.

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APPENDIX A
CONSTRUCTION OF THE SPECIFIC VOID RATIO DIAGRAM

The void ratio is an expression of the ratio of voids to solid volume, and is characterized by the equation:

$$V_r = \frac{VMA}{1-VMA}$$

Consider the coarse component of binary mixture. The solid volume can be depicted as varying linearly from zero to unity, as shown in figure A1a, with the void ratio of the component represented by Vr_c. Similarly, the fine component may be represented by the diagram A1b in which Vr_f = void ratio of the fine aggregate. In the theoretical case where the coarse aggregate is infinitely large, or the fine aggregate is infinitely small, then the size ratio between the two components is zero, and the fine aggregate will not dilate the coarse aggregate until the bulk volume of fines exceeds the voids within the coarse component. This situation is shown by figure A1c. Real combinations of two aggregate sizes exhibit size ratios between zero and 1 and lie in the region of AED in figure A1c. This can be illustrated more clearly by figure A2 where a typical relationship is shown.

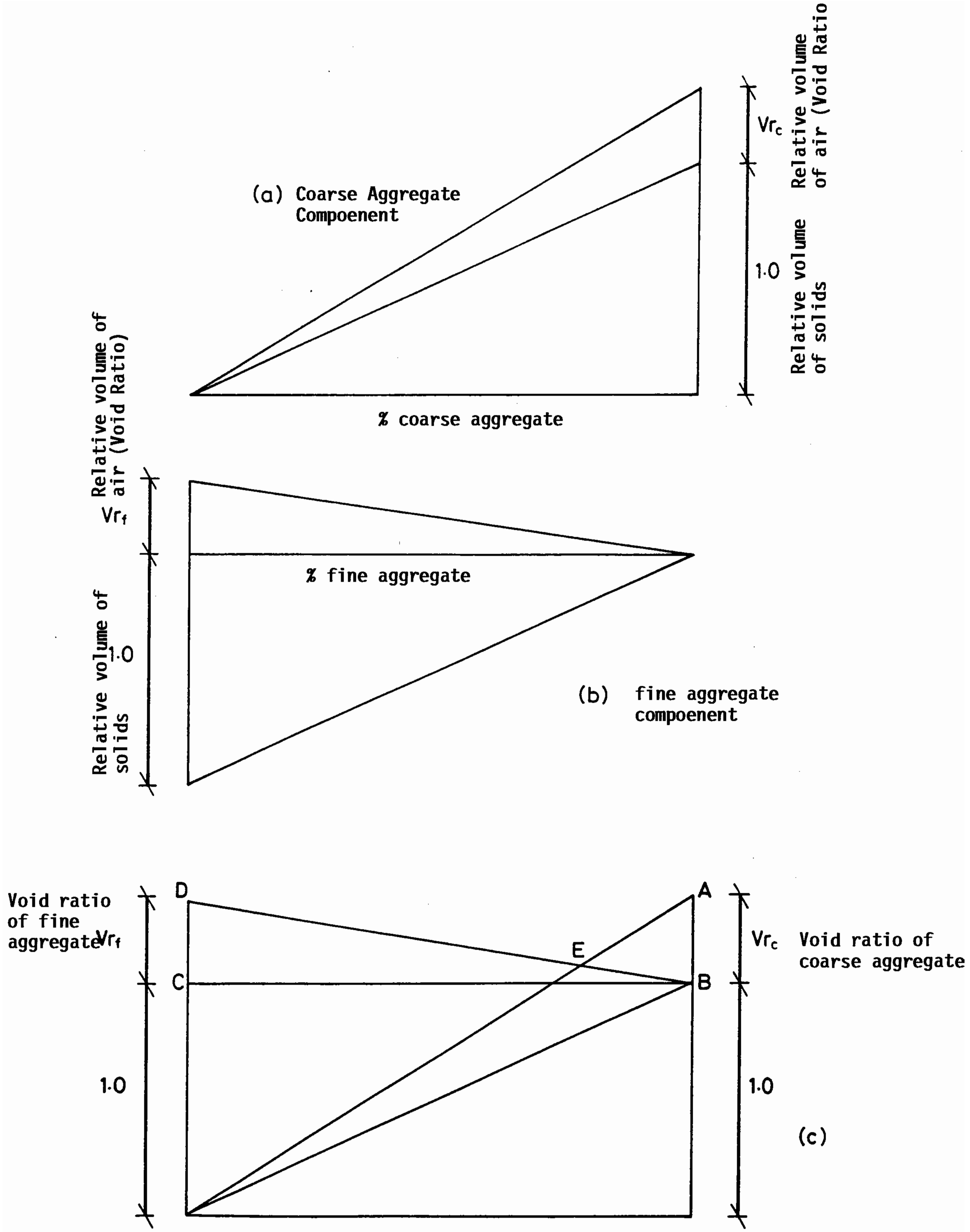

Figure A1 Construction of the void ratio diagram

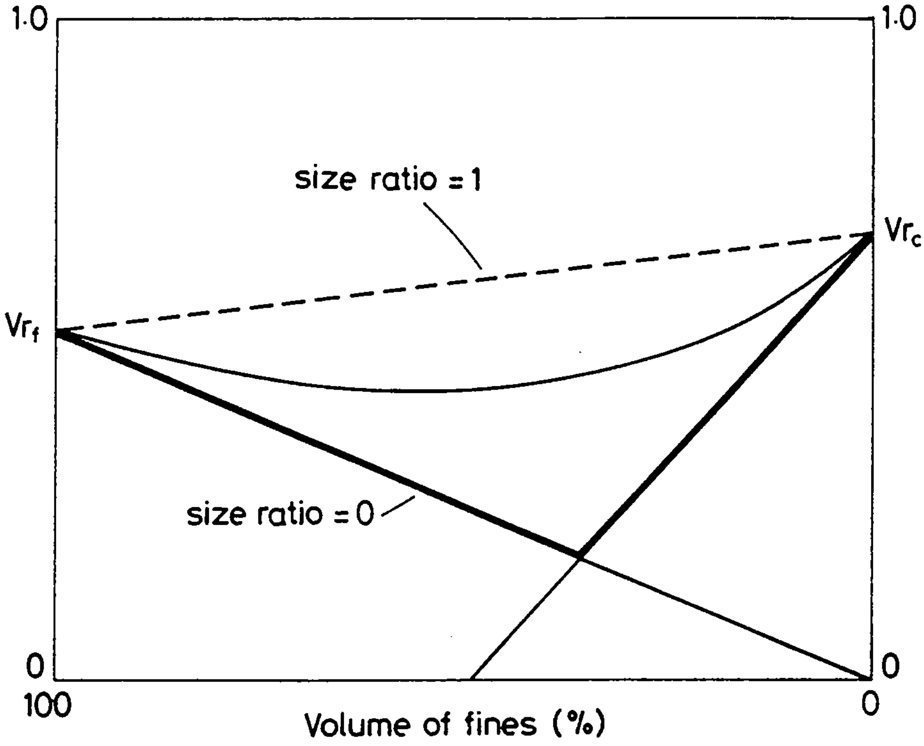

Figure A2 The void ratio diagram

APPENDIX B

RESULTS FROM THE MIX DESIGN PROCEDURE USING GRANITE AGGREGATE

TABLE B.1 Volumetric proportions and elastic stiffness values of specimens (n=0.5)

Grading	M_B (%)	Compac-tion level	Density (g/ml)	V_V (%)	V_B (%)	VMA (%)	PRD (%)	Elastic Stiffness (MP_a)
0.5	3.5	1	2.602	1.7	8.9	10.6	100.0	4600
0.5	3.5	2	2.449	7.5	8.4	15.9	94.1	3600
0.5	3.5	3	2.421	8.6	8.3	16.9	93.0	2500
0.5	4.1	1	2.588	1.3	10.4	11.7	100.0	4000
0.5	4.1	2	2.466	5.9	9.9	15.8	95.3	3800
0.5	4.1	3	2.452	6.5	9.8	16.3	94.7	3200
0.5	4.7	1	2.565	1.2	11.8	13.0	100.0	3000
0.5	4.7	2	2.509	3.3	11.5	14.8	97.8	2500
0.5	4.7	3	2.483	4.3	11.4	15.7	96.8	2200

TABLE B.2

Volumetric proportions and elastic stiffness values of specimens (n=0.6)

Grading	M_B (%)	Compac-tion level	Density (g/ml)	V_V (%)	V_B (%)	VMA (%)	PRD (%)	Elastic Stiffness (MP_a)
0.5	3.5	1	2.594	2.0	8.9	10.9	100.0	4400
0.5	3.5	2	2.492	5.9	8.5	14.4	96.1	4000
0.5	3.5	3	2.430	8.2	8.3	16.5	93.7	3000
0.5	4.1	1	2.600	0.8	10.4	11.2	100.0	4500
0.5	4.1	2	2.431	7.3	9.7	17.0	92.2	3500
0.5	4.1	3	2.398	8.5	9.6	18.1	94.7	2400
0.5	4.7	1	2.563	1.3	11.8	13.2	100.0	4000
0.5	4.7	2	2.454	5.5	11.3	16.8	95.7	2800
0.5	4.7	3	2.418	6.9	11.1	18.0	94.3	1900

TABLE B.3

Volumetric proportions and elastic stiffness values of specimens (n=0.7)

Grading	M_B (%)	Compac-tion level	Density (g/ml)	V_V (%)	V_B (%)	VMA (%)	PRD (%)	Elastic Stiffness (MP_a)
0.7	3.5	1	2.567	3.0	8.8	11.8	100.0	3600
0.7	3.5	2	2.457	7.2	8.4	15.6	95.7	2300
0.7	3.5	3	2.400	9.3	8.2	17.5	93.5	1900
0.7	4.1	1	2.568	2.0	10.3	12.3	100.0	3500
0.7	4.1	2	2.442	6.8	9.8	16.6	95.1	2200
0.7	4.1	3	2.385	9.0	9.6	18.6	92.9	1700

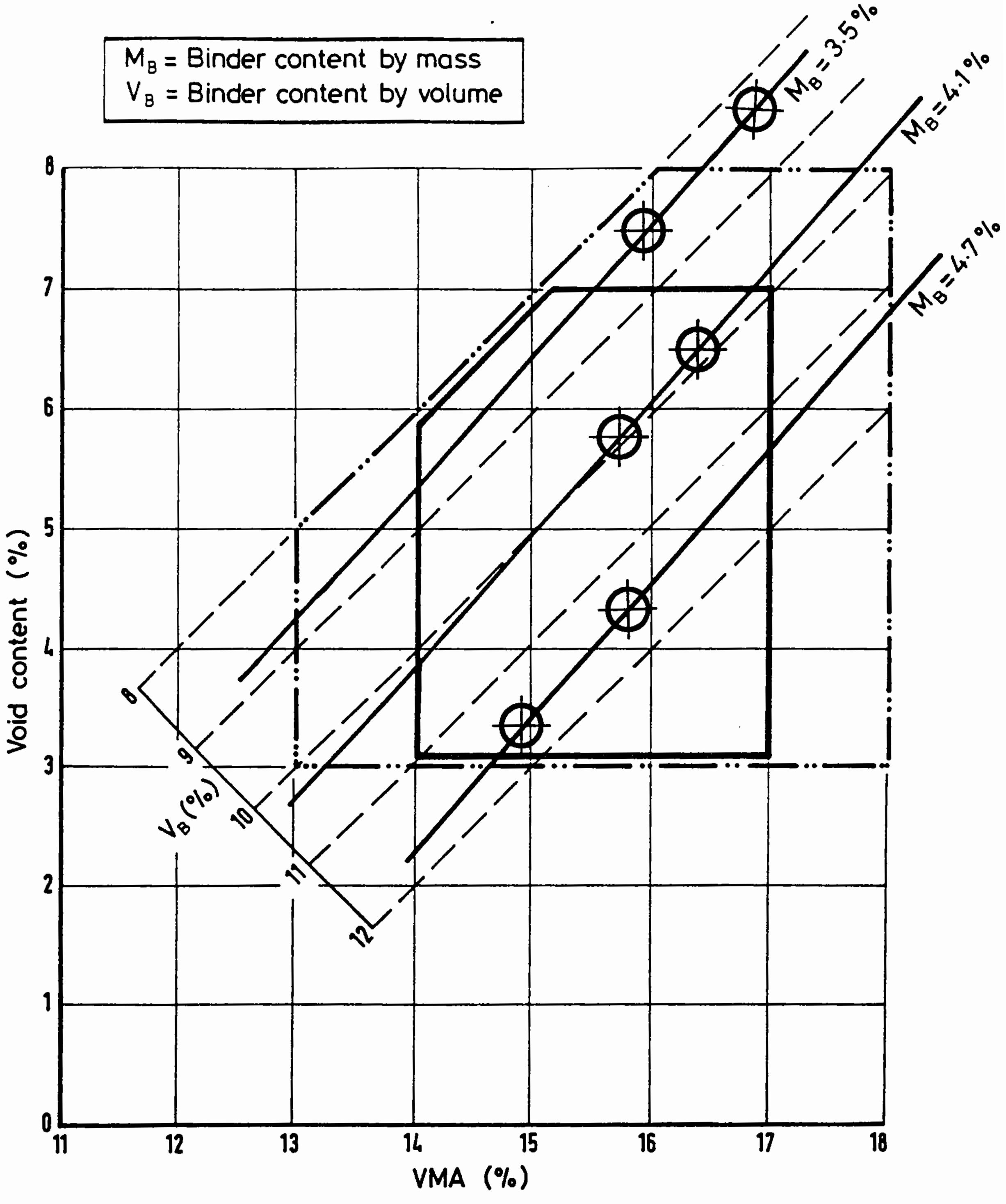

Figure B1 Volumetric proportions of granite specimens n=0.5, compaction levels 2 & 3

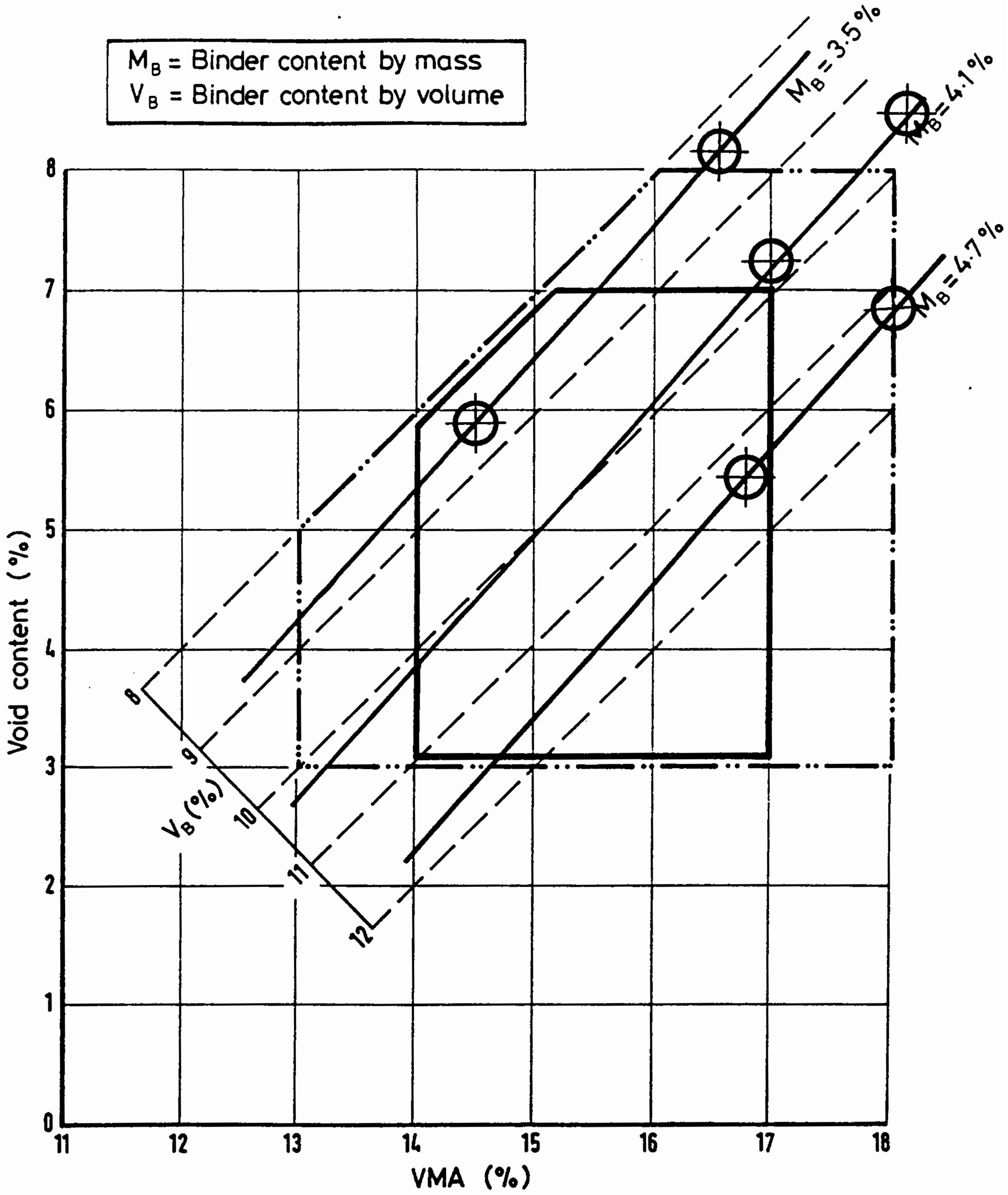

Figure B2 Volumetric proportions of granite specimens n=0.6, compaction levels 2 & 3

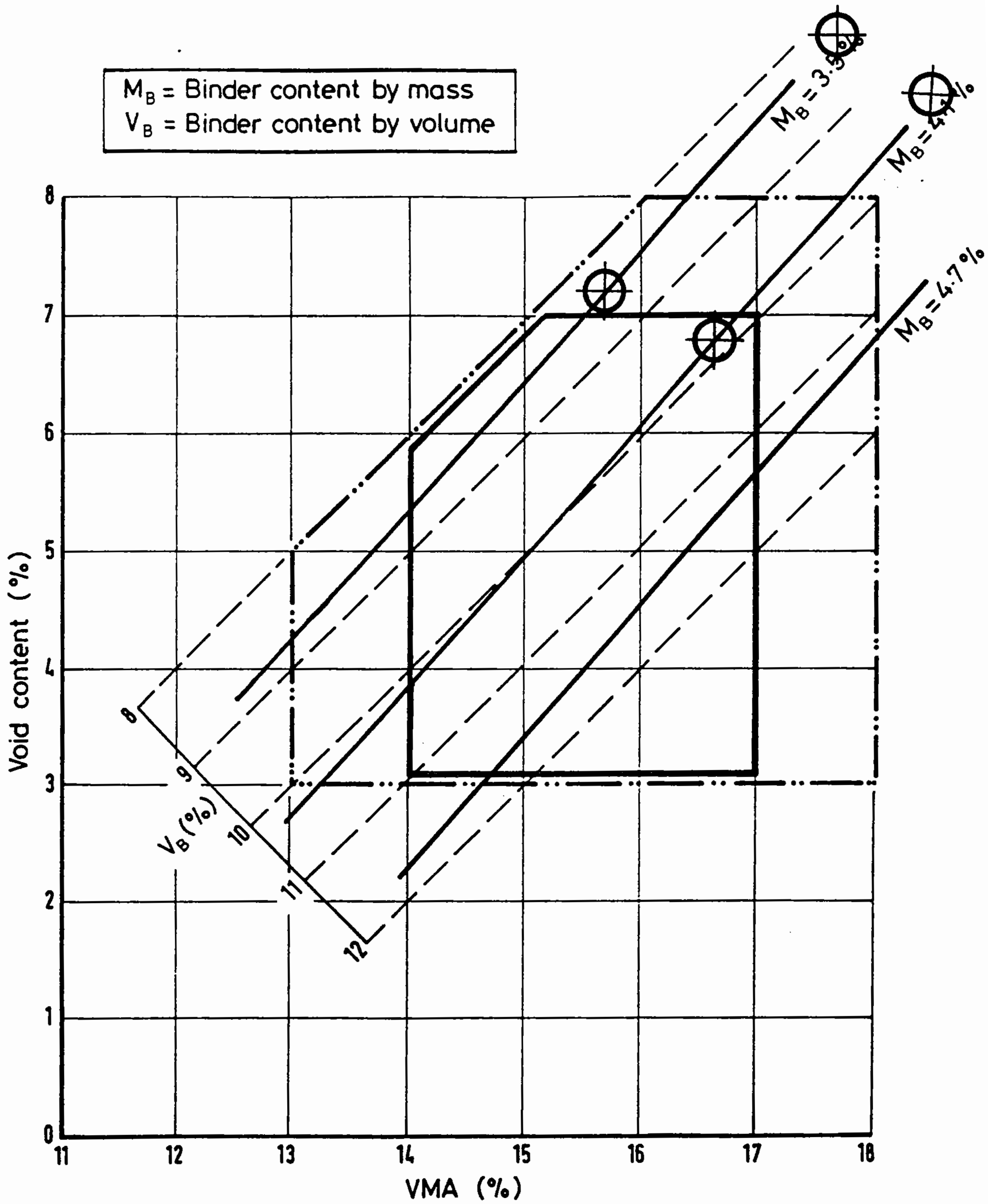

Figure B3 Volumetric proportions of granite specimens n=0.7, compaction levels 2 & 3

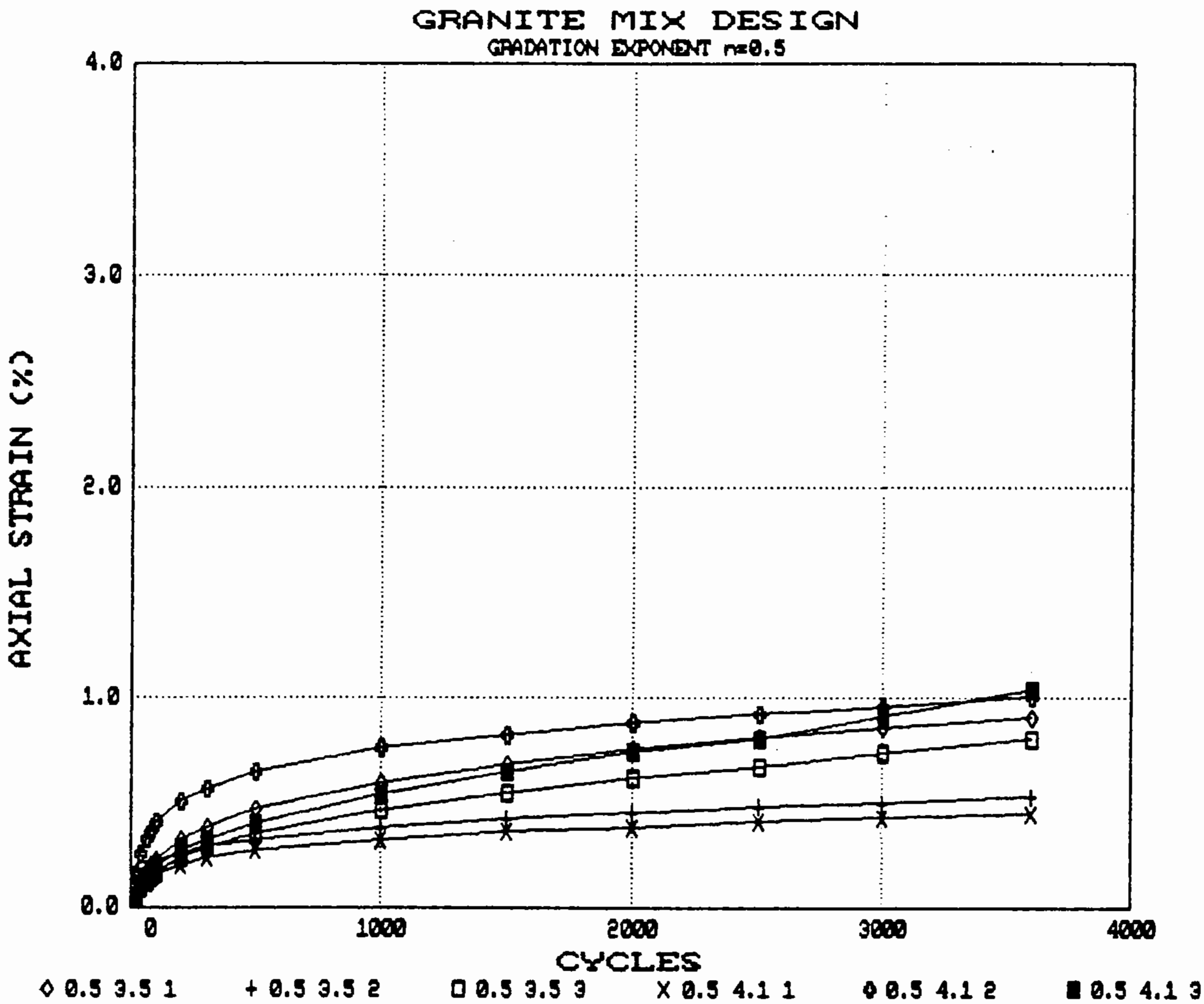

Figure B4 Deformation curves plotted from the results of RLA tests. Granite aggregate, mix formulations corresponding to n=0.5, M_B=3.5% and n=0.5, M_B=4.1%

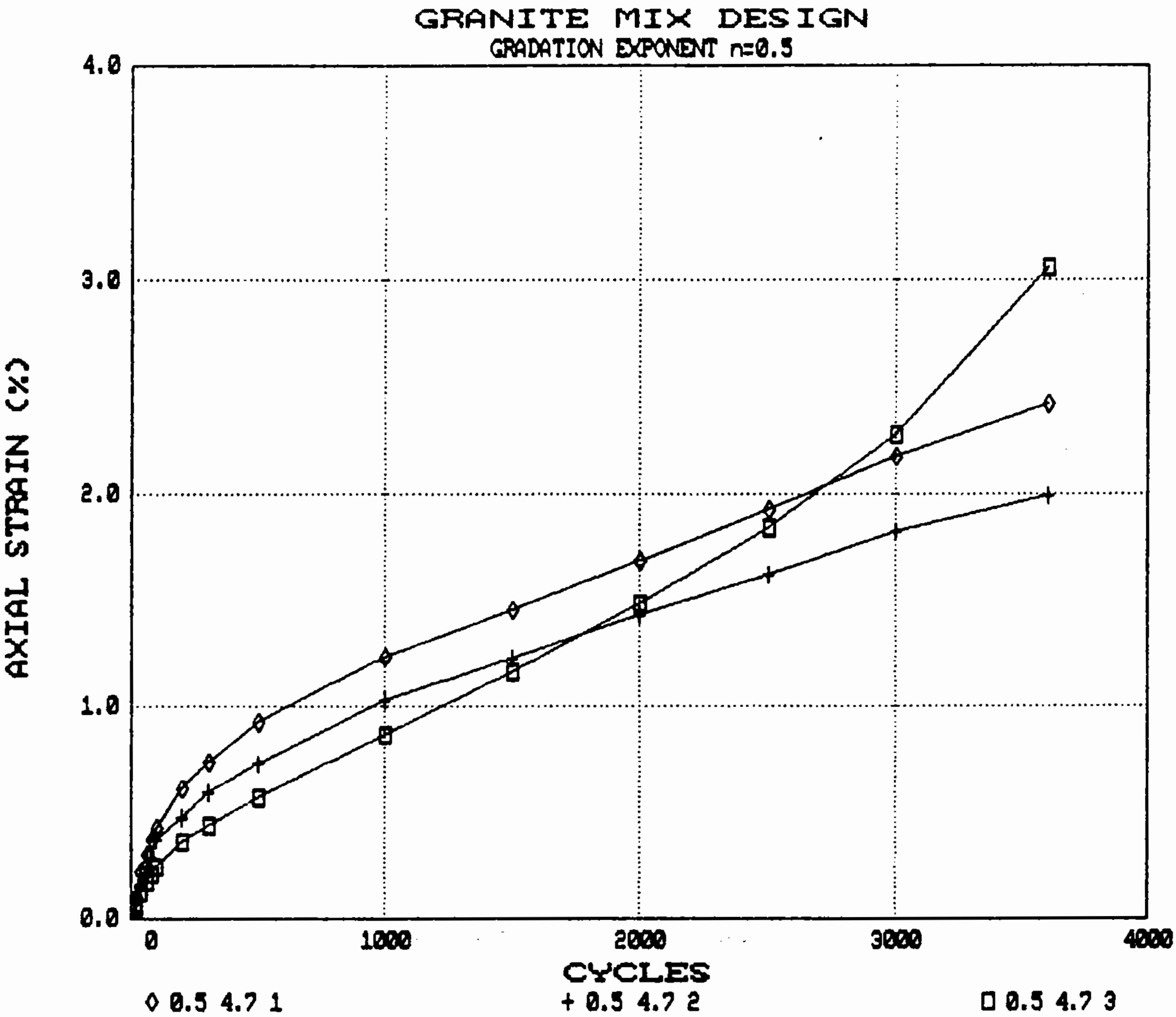

Figure B5 Deformation curves plotted from the results if RLAtests. Granite aggregate, mix formulations corresponding to n=0.5, M_B=4.7%

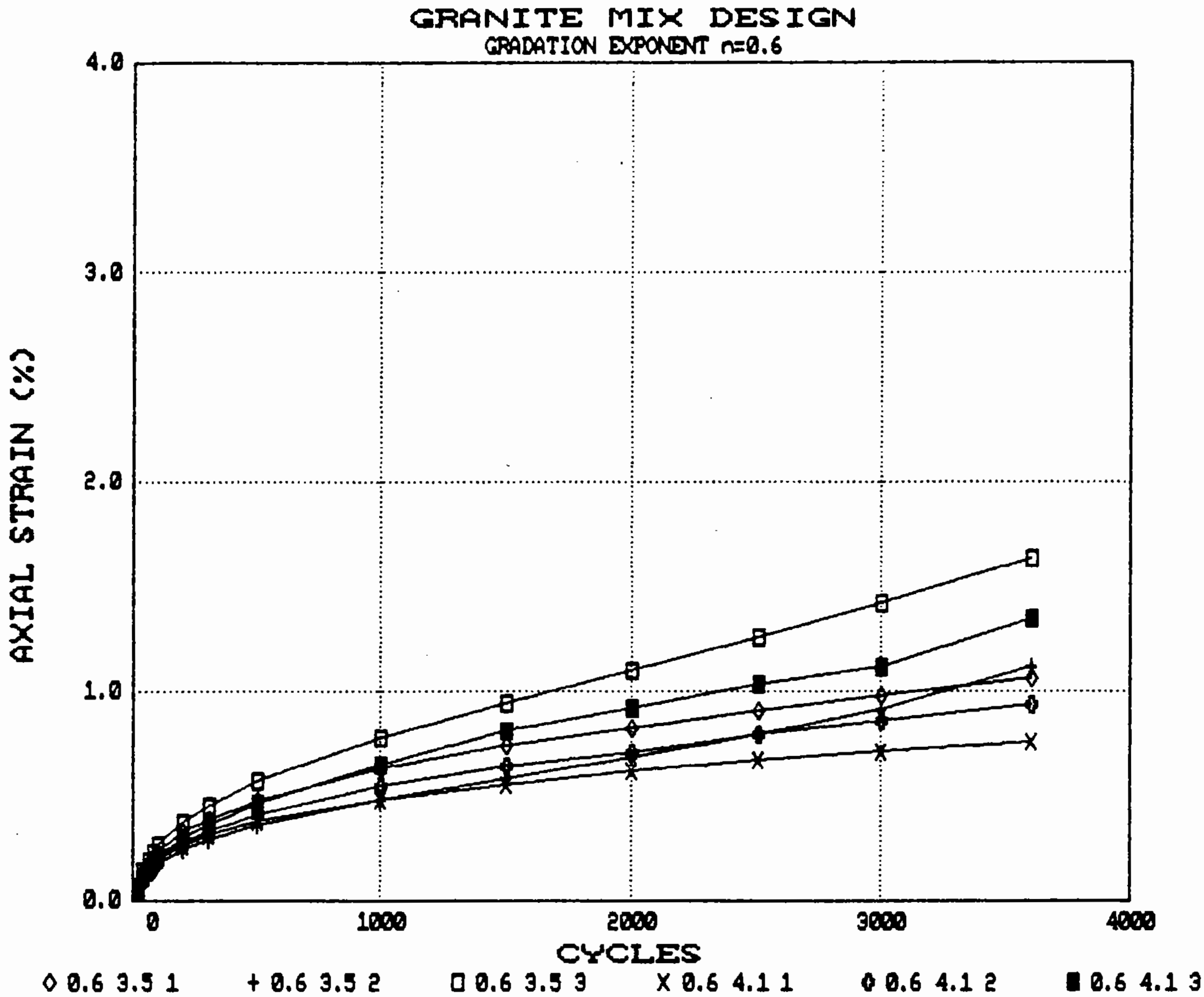

Figure B6 Deformation curves plotted from the results of RLA tests. Granite aggregate, mix formulations corresponding to n=0.6, M_B=3.5%, n=0.6, M_B=4.1%

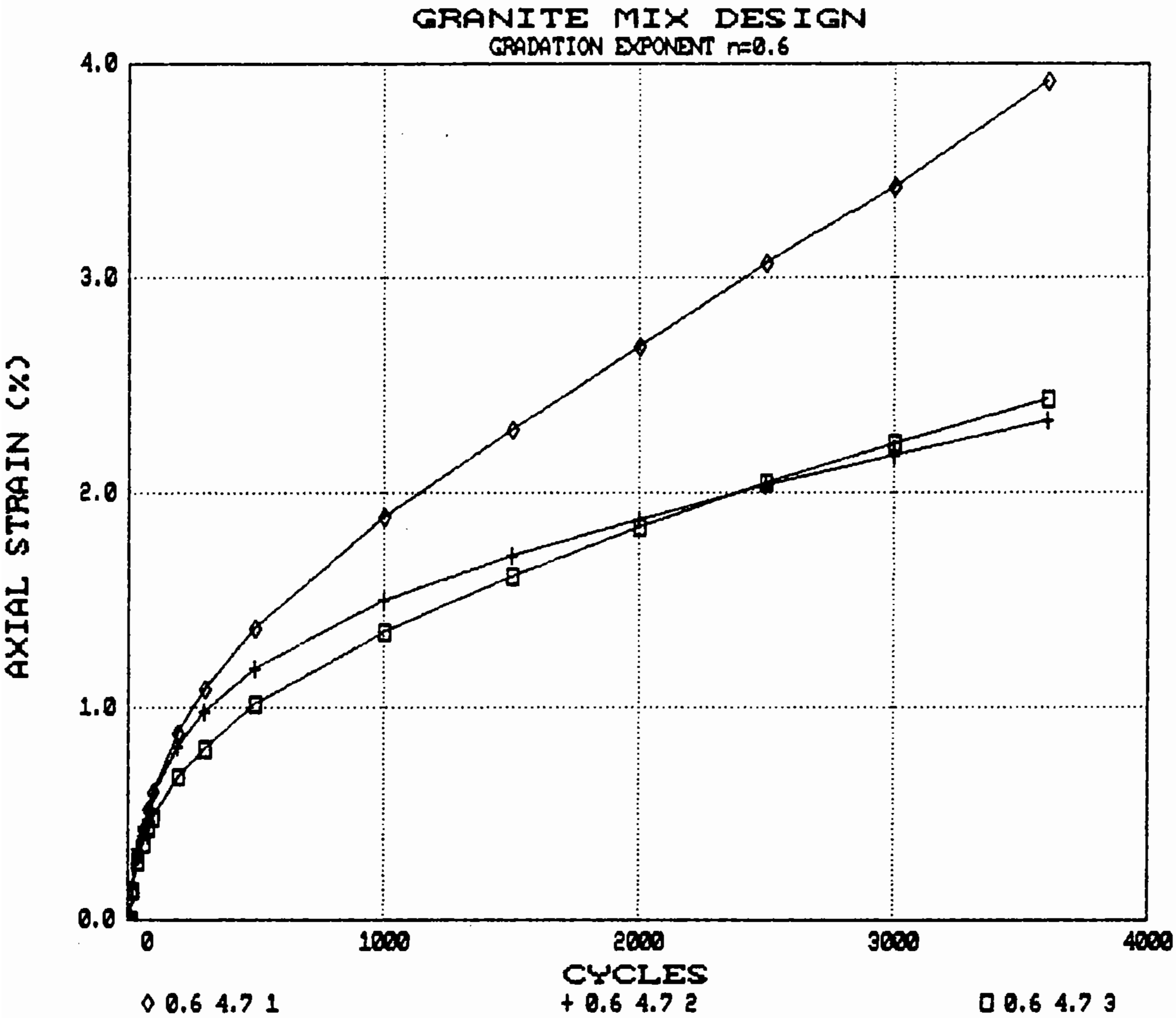

Figure B7 Deformation curves plotted from the results of RLA tests. Granite aggregate, mix formulations corresponding to n=0.6, M_B=4.7%

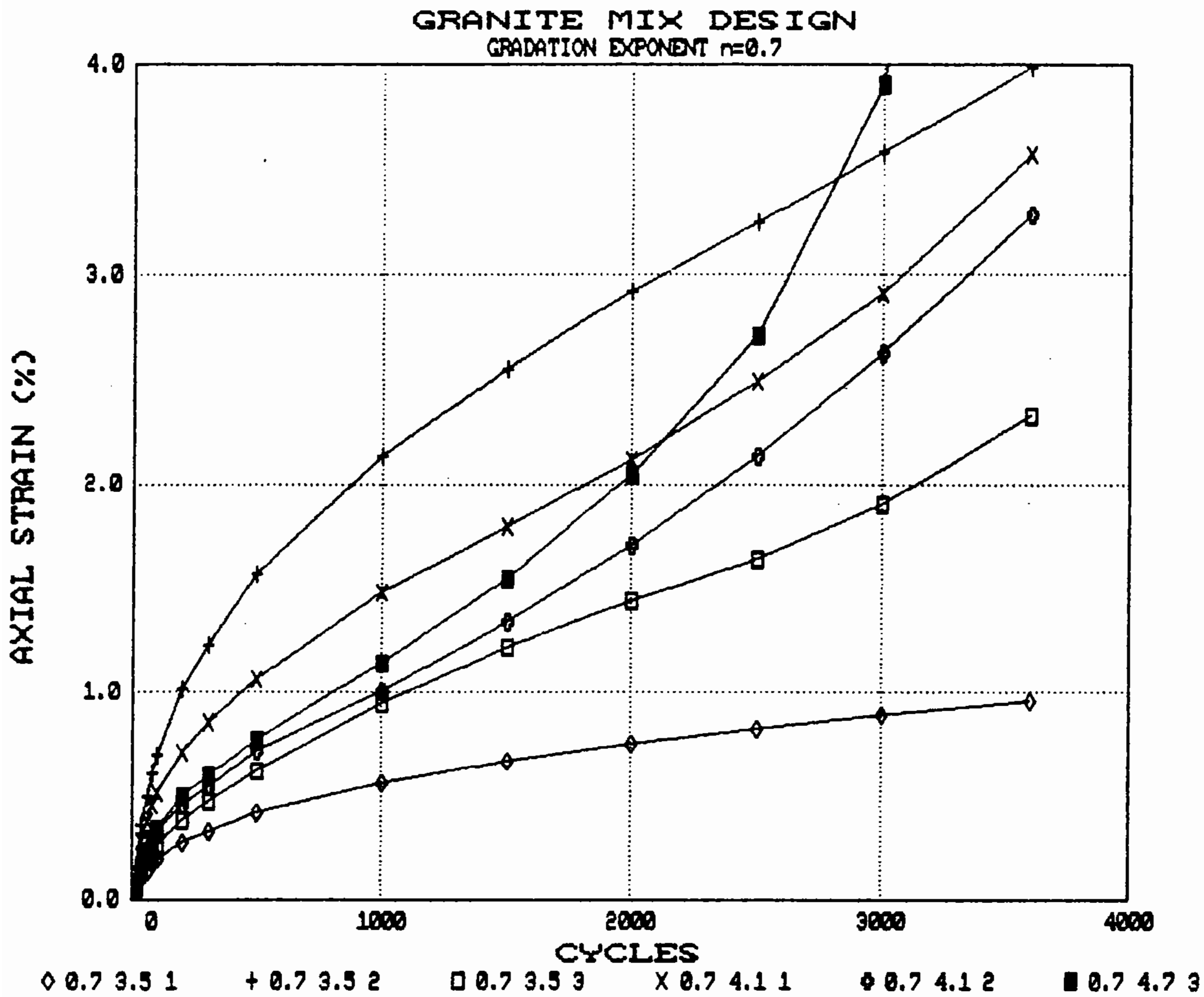

Figure B8 Deformation curves plotted from the results of RLA tests. Granite aggregate, mix formulations corresponding to n=0.7, M_B=3.5%, n=0.7, M_B= 4.1% and n=0.7, M_B=4.7%

APPENDIX C

THE STRATEGIC HIGHWAY RESEARCH PROGRAM MARSHALL MIX DESIGN RESULTS

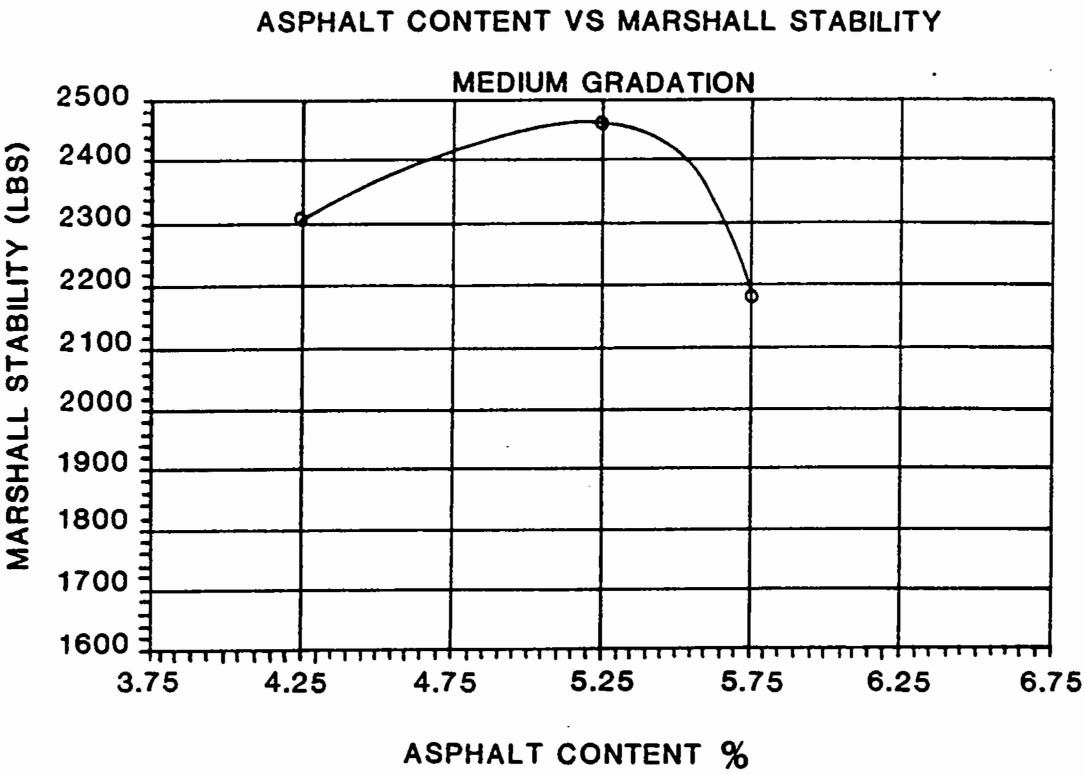

Figure C1 Asphalt content vs Marshall Stability (Medium Gradation)

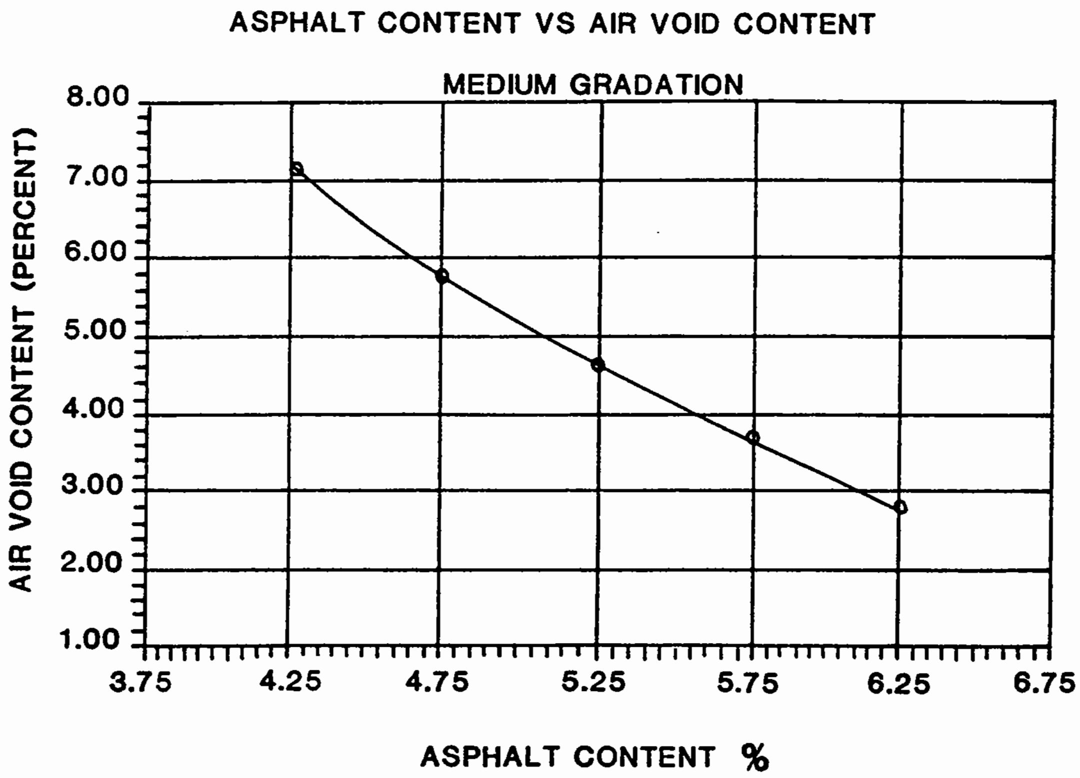

Figure C2 Asphalt content vs air void content (Medium Gradation)

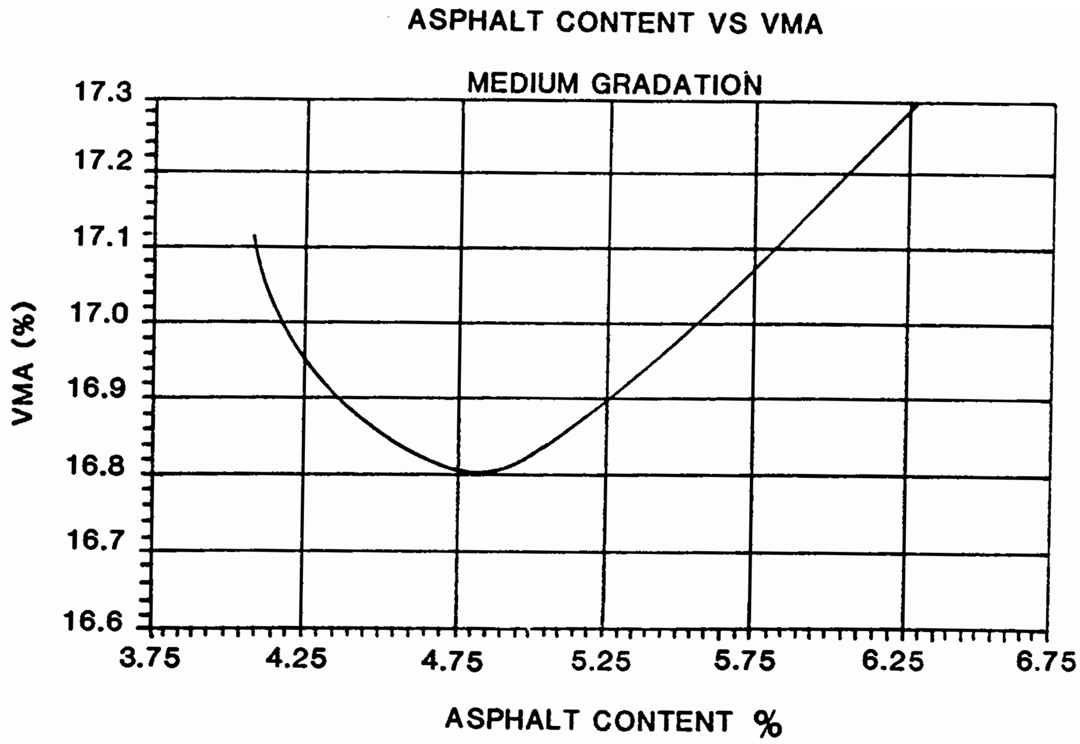

Figure C3 Asphalt content vs VMA (Medium Gradation)

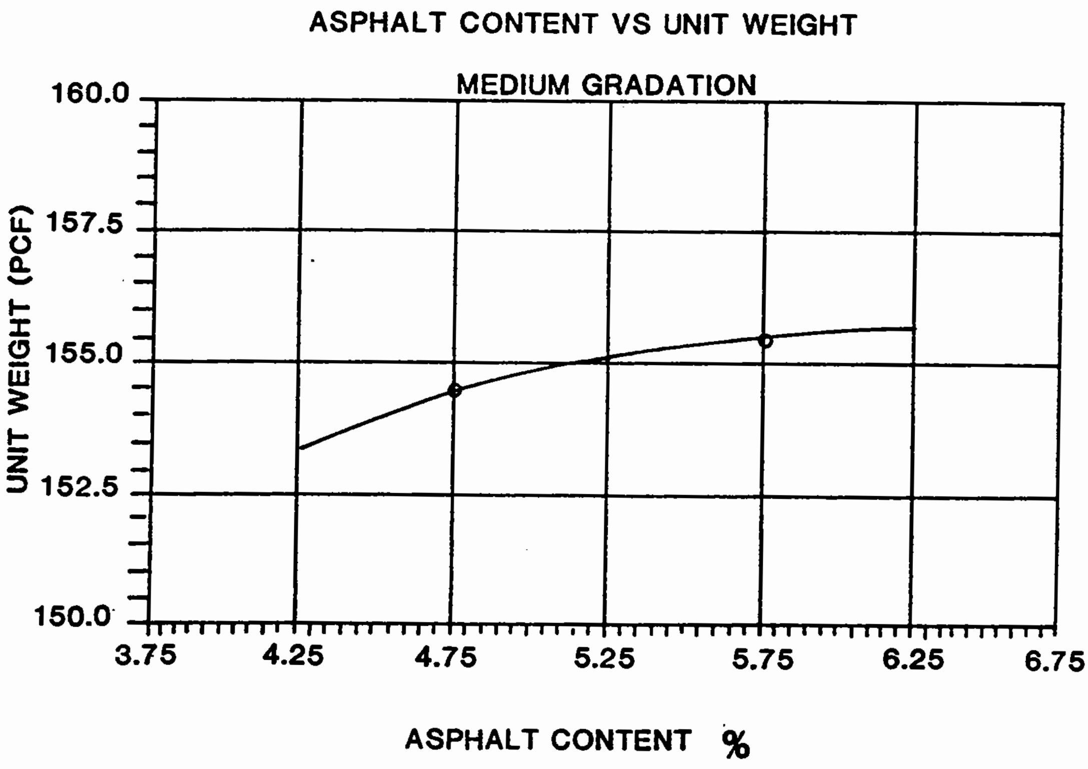

Figure C4 Asphalt content vs unit weight (Medium Gradation)

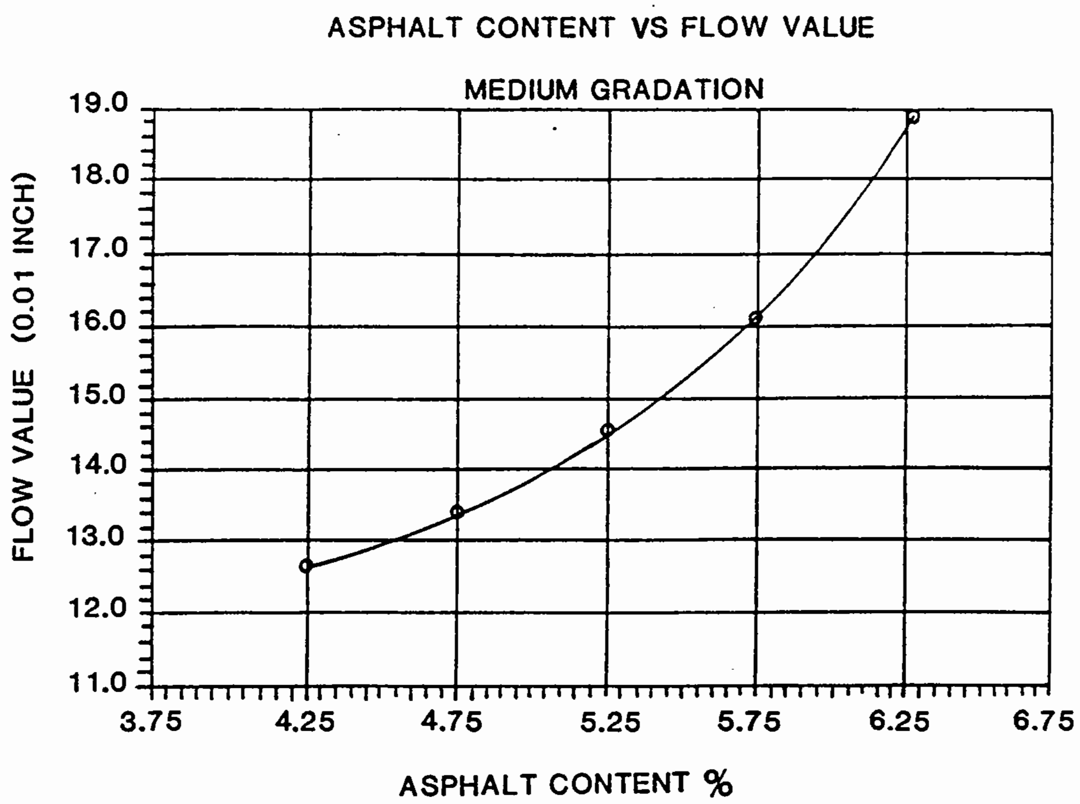

Figure C5 Asphalt content vs flow (Medium Gradation)

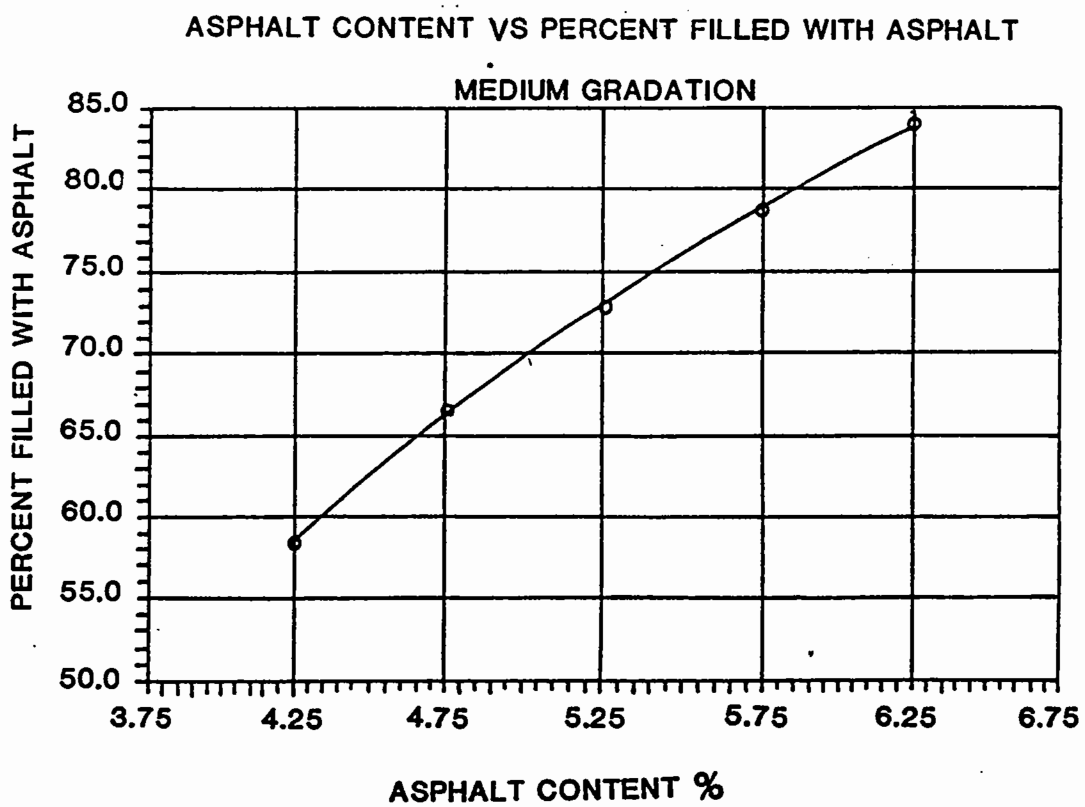

Figure C6 Asphalt content vs voids filled (Medium Gradation)

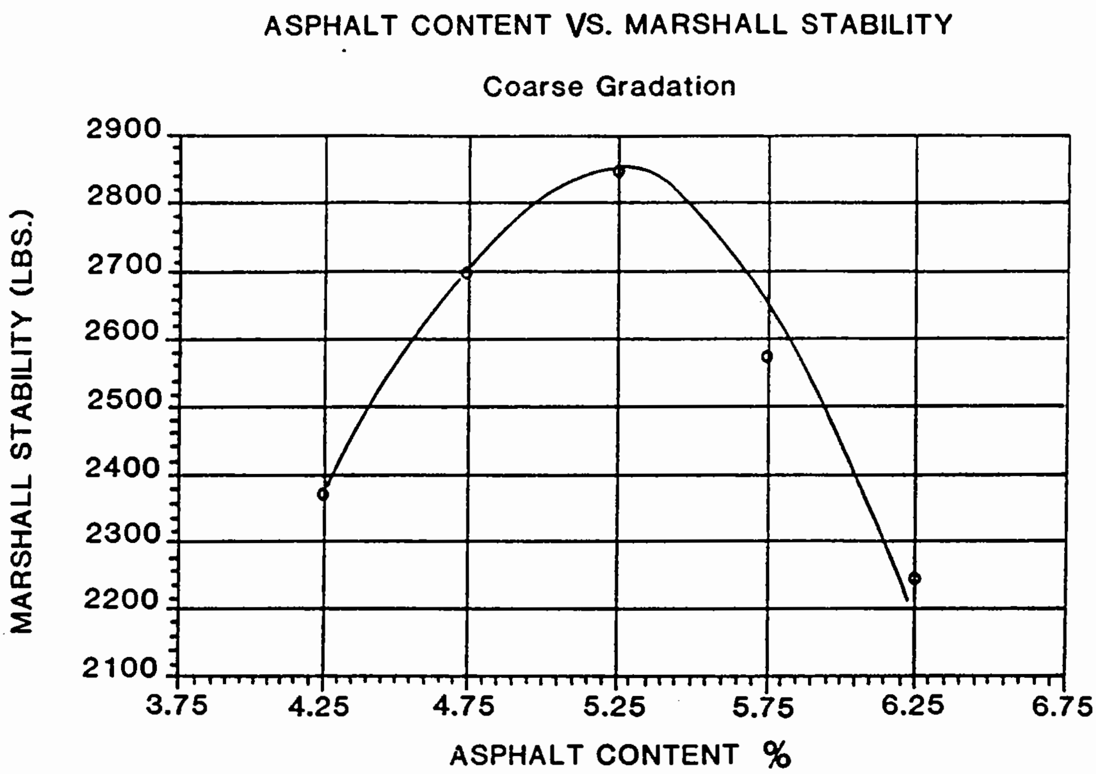

Figure C7 Asphalt content vs Marshall Stability (Coarse Gradation)

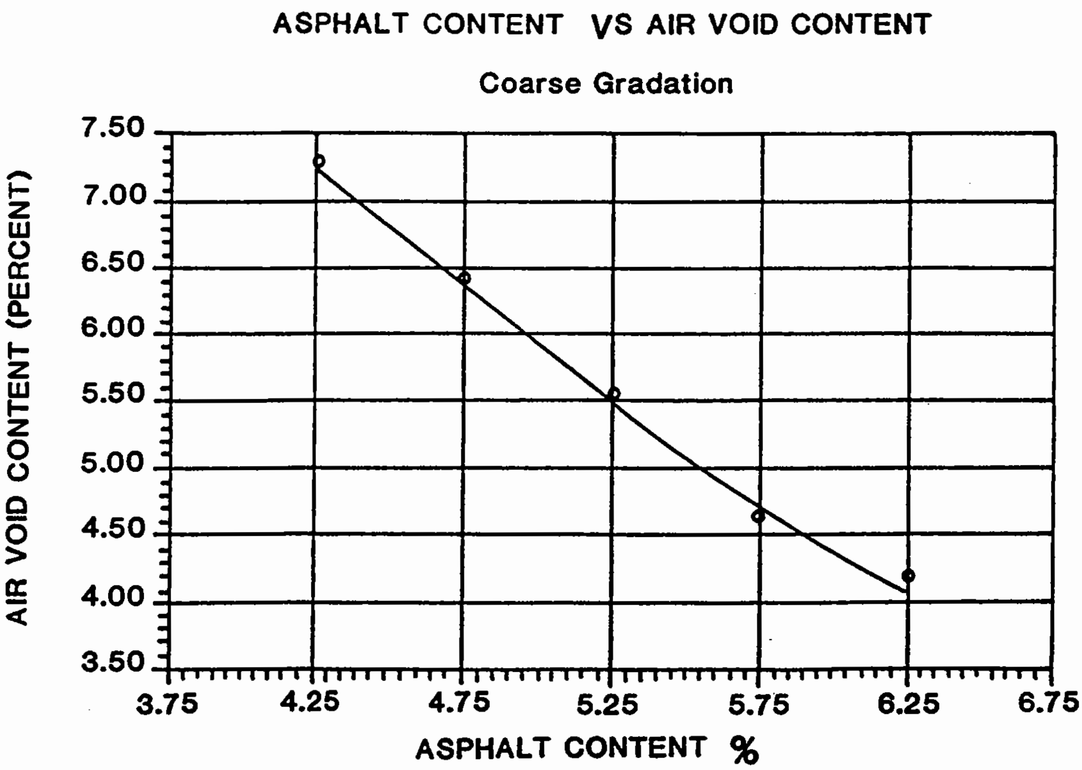

Figure C8 Asphalt content vs air void content (Coarse Gradation)

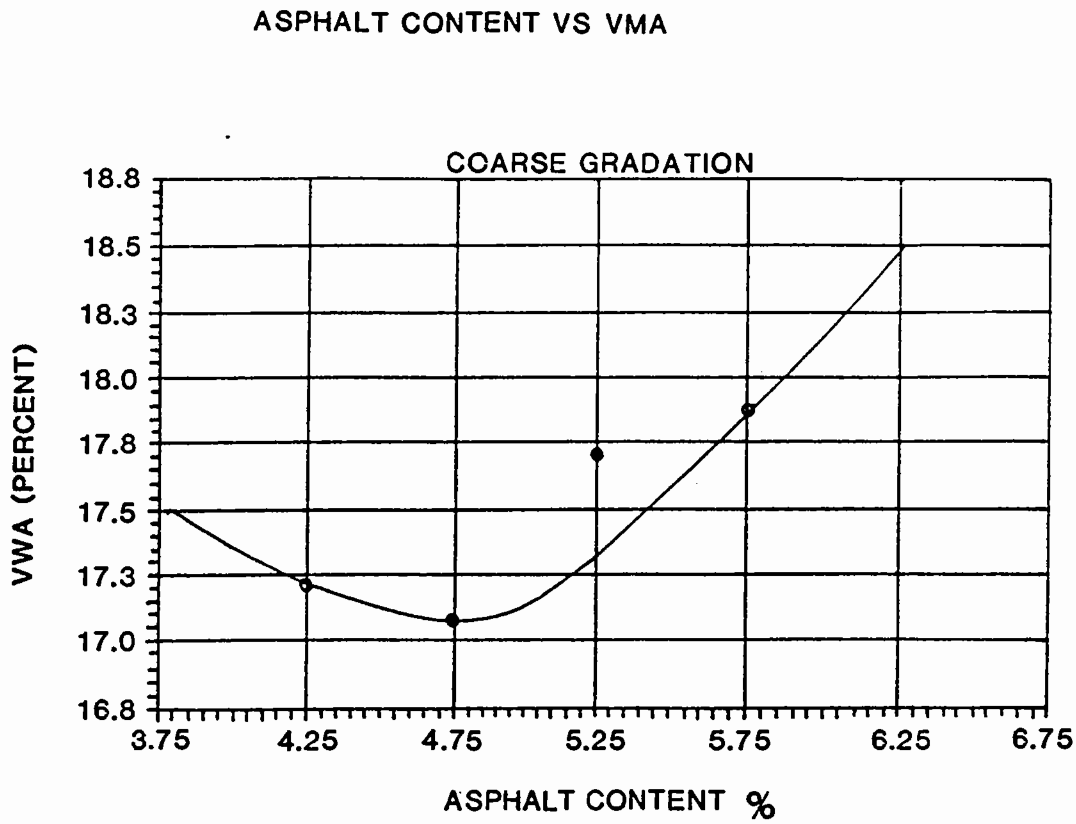

Figure C9 Asphalt content vs VMA (Coarse Gradation)

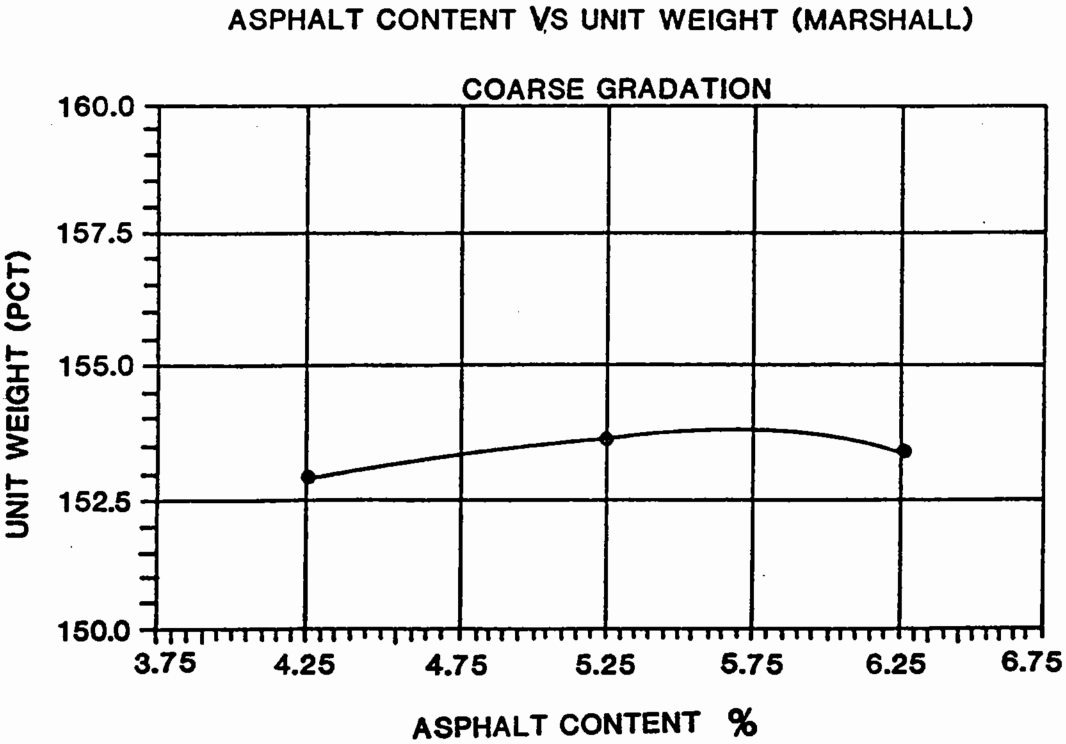

Figure C10 Asphalt content vs unit weight (Coarse Gradation)

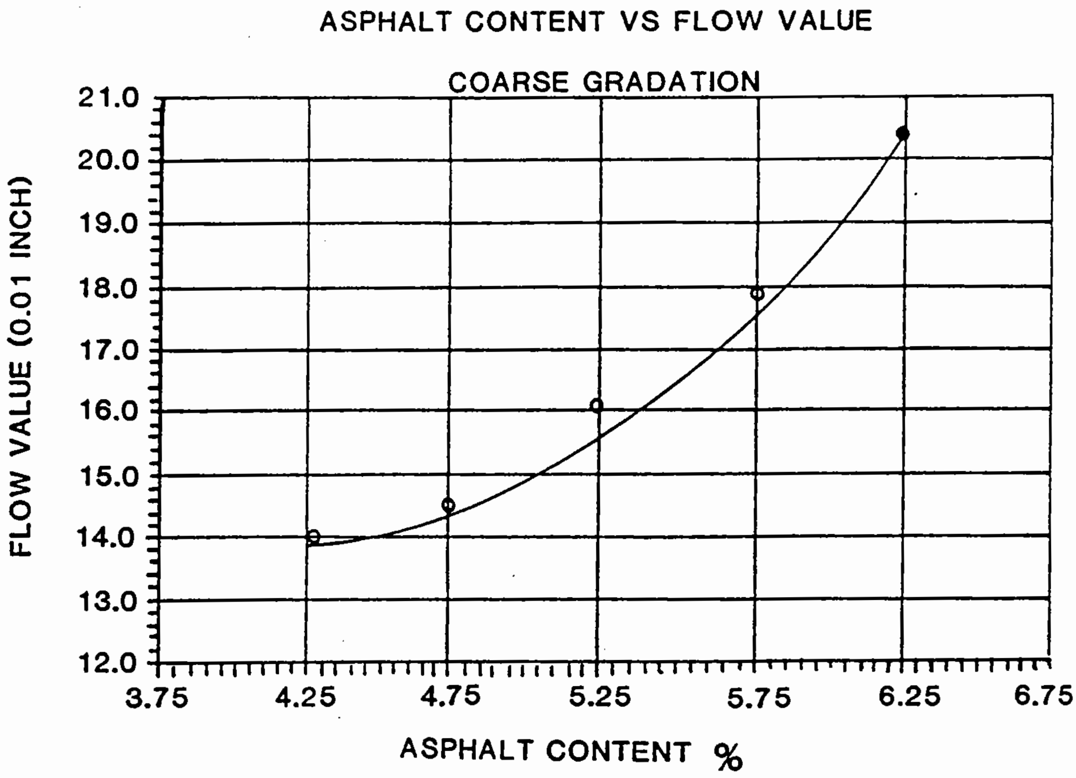

Figure C11 Asphalt content vs flow (Coarse Gradation)

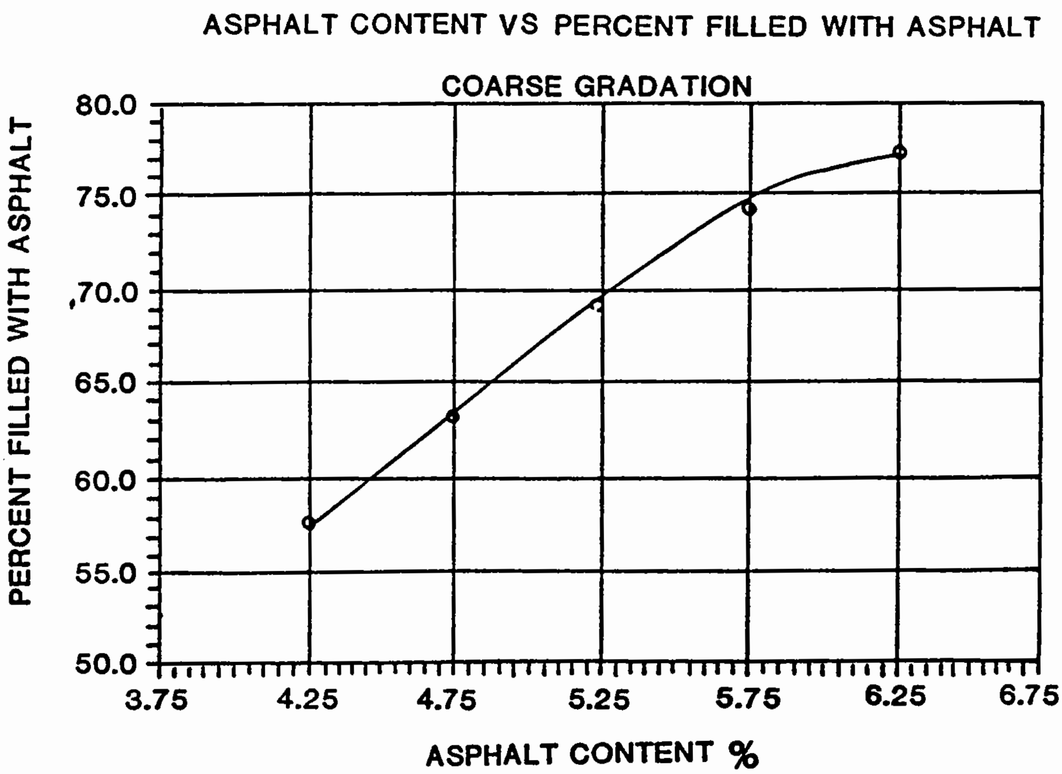

Figure C12 Asphalt content vs voids filled (Coarse Gradation)

APPENDIX D

An example of the influence of mix stiffness and volumetric proportions on the analytical design of a road pavement. The values of elastic stiffness and binder volume were taken from the results of the Nottingham and SHRP mix formulations described in Chapter II. The pavement designs were performed using the Shell package BISAR-PC and BANDS-PC.

The design package shows that the SHRP mix formulation exhibits a greater fatigue life for both pavement constructions but the Nottingham mixture has a greater resistance to permanent deformation. The design calculations show the subgrade strain to be critical in the life of the pavement.

Section B of the printouts describe the material properties and the thickness of each layer considered in the design example, with stiffness values for the asphaltic layers taken from the mix designs performed in Chapter 10. The loading conditions are shown in section C and the resulting stresses and strains at the relevant coordinates are illustrated in section D. The critical tensile strain at the underside of the asphaltic layers (position 1 or 2) was then used in the BANDS-PC software to predict the material fatigue life. These results have been tabulated underneath section D.

Chart RS depicts the relationship between subgrade strain and load applications.

For the two examples shown, which consider asphalt layers 150mm and 200mm thick, both mix formulations exhibit satisfactory characteristics for design lives in excess of 100 million standard axles.

Title: COMPARISON OF NOTTINGHAM AND SHRP DESIGNS

Start time: 9:41:59.26 DATE: 9 April 1991
Stop time: 9:42:46.99

A. DESCRIPTION OF THE SYSTEMS (Systems: 1 to 2 OF 2 Systems)

SYS.#	DESCRIPTION
1	NOTTINGHAM MIX FORMULATION
2	SHRP MIX FORMULATION

B. CHARACTERISATION OF THE CONSTRUCTIONS (Systems: 1 to 2 OF 2 Systems)

Title: COMPARISON OF NOTTINGHAM AND SHRP DESIGNS

SYS.	<<<< LAYER 1 >>>>	<<<< LAYER 2 >>>>	<<<< LAYER 3 >>>>	<<<< LAYER 4 >>>>
1	E-mod: 8800.0 MPa	E-mod: 151.9 MPa	E-mod: 70.0 MPa	
	Thick: .200 m	Thick: .200 m	Thick: infinite	
	PR : .35 -	PR : .35 -	PR : .35 -	
2	E-mod: 5000.0 MPa	E-mod: 151.9 MPa	E-mod: 70.0 MPa	
	Thick: .200 m	Thick: .200 m	Thick: infinite	
	PR : .35 -	PR : .35 -	PR : .35 -	

C. CHARACTERISATION OF THE LOADS (Systems: 1 to 2 OF 2 Systems)

Title: COMPARISON OF NOTTINGHAM AND SHRP DESIGNS

SYS.	<<<< LOAD 1 >>>>	<<<< LOAD 2 >>>>	<<<< LOAD 3 >>>>	<<<< LOAD 4 >>>>
1	LOADING (2 units)	LOADING (2 units)		
	Load: 20.0kN	Load: 20.0kN		
	Strs: 577.4kPa	Strs: 577.4kPa		
	Radius: .105 m	Radius: .105 m		
	X-coor: .000 m	X-coor: .000 m		
	Y-coor: -.157 m	Y-coor: +.157 m		
2	LOADING (2 units)	LOADING (2 units)		
	Load: 20.0kN	Load: 20.0kN		
	Strs: 577.4kPa	Strs: 577.4kPa		
	Radius: .105 m	Radius: .105 m		
	X-coor: .000 m	X-coor: .000 m		
	Y-coor: -.157 m	Y-coor: +.157 m		

```
D.  POSITIONS AND RESULTS   (Pos.: 1- 5)    (Systems: 1 to  2   OF  2 Systems)
==============================================================================
Title: COMPARISON OF NOTTINGHAM AND SHRP DESIGNS

   SIGN convention STRESSES and STRAINS: + = TENSILE     stress or strain;
                                         - = COMPRESSIVE stress or strain.

SYS.#< POSITION 1 >#< POSITION 2 >#< POSITION 3 >#< POSITION 4 >#< POSITION  5
----#--------------#--------------#--------------#--------------#--------------
  1 #COORDINATES(m)#COORDINATES(m)#COORDINATES(m)#COORDINATES(m)#
    # X-crd:  .0000# X-crd:  .0000# X-crd:  .0000# X-crd:  .0000#
    # Y-crd:  .0000# Y-crd: +.1575# Y-crd:  .0000# Y-crd: +.1575#
    # Depth:  .2000# Depth:  .2000# Depth:  .4000# Depth:  .4000#
    # Layer:layer 1# Layer:layer 1# Layer:layer 3# Layer:layer 3#
    #              #              #              #              #
    #maximum       #maximum       #maximum       #maximum       #
    #STRESSES (MPa)#STRESSES (MPa)#STRESSES (MPa)#STRESSES (MPa)#
    # Hor.X: +.9010# Hor.X: +.9065# Hor.Y: -.0011# Hor.Y: -.0012#
    # Vert.: -.0276# Vert.: -.0280# Vert.: -.0166# Vert.: -.0158#
    #              #              #              #              #
    #maximum       #maximum       #maximum       #maximum       #
    #STRAINS (um/m)#STRAINS (um/m)#STRAINS (um/m)#STRAINS (um/m)#
    # Hor.X:  +75.9# Hor.X:  +73.3# Hor.X:  +81.1# Hor.X:  +77.7#
    # Vert.:  -66.5# Vert.:  -70.1# Vert.: -229.3# Vert.: -216.9#
----#--------------#--------------#--------------#--------------#--------------
  2 #COORDINATES(m)#COORDINATES(m)#COORDINATES(m)#COORDINATES(m)#
    # X-crd:  .0000# X-crd:  .0000# X-crd:  .0000# X-crd:  .0000#
    # Y-crd:  .0000# Y-crd: +.1575# Y-crd:  .0000# Y-crd: +.1575#
    # Depth:  .2000# Depth:  .2000# Depth:  .4000# Depth:  .4000#
    # Layer:layer 1# Layer:layer 1# Layer:layer 3# Layer:layer 3#
    #              #              #              #              #
    #maximum       #maximum       #maximum       #maximum       #
    #STRESSES (MPa)#STRESSES (MPa)#STRESSES (MPa)#STRESSES (MPa)#
    # Hor.X: +.7492# Hor.X: +.7594# Hor.X:.000058# Hor.Y: -.0011#
    # Vert.: -.0384# Vert.: -.0394# Vert.: -.0216# Vert.: -.0204#
    #              #              #              #              #
    #maximum       #maximum       #maximum       #maximum       #
    #STRAINS (um/m)#STRAINS (um/m)#STRAINS (um/m)#STRAINS (um/m)#
    # Hor.X: +114.0# Hor.X: +110.0# Hor.X: +113.2# Hor.X: +107.8#
    # Vert.:  -98.6# Vert.: -105.7# Vert.: -305.1# Vert.: -286.3#
----#--------------#--------------#--------------#--------------#--------------
```

Volume perc. Bitumen	Stiffness Asphalt mix	Fatigue Strain	Fatigue Life
% v	MPa	μm/m	x 1000
8.40	8800.	73.	23500.
12.00	5000.	114.	35400.

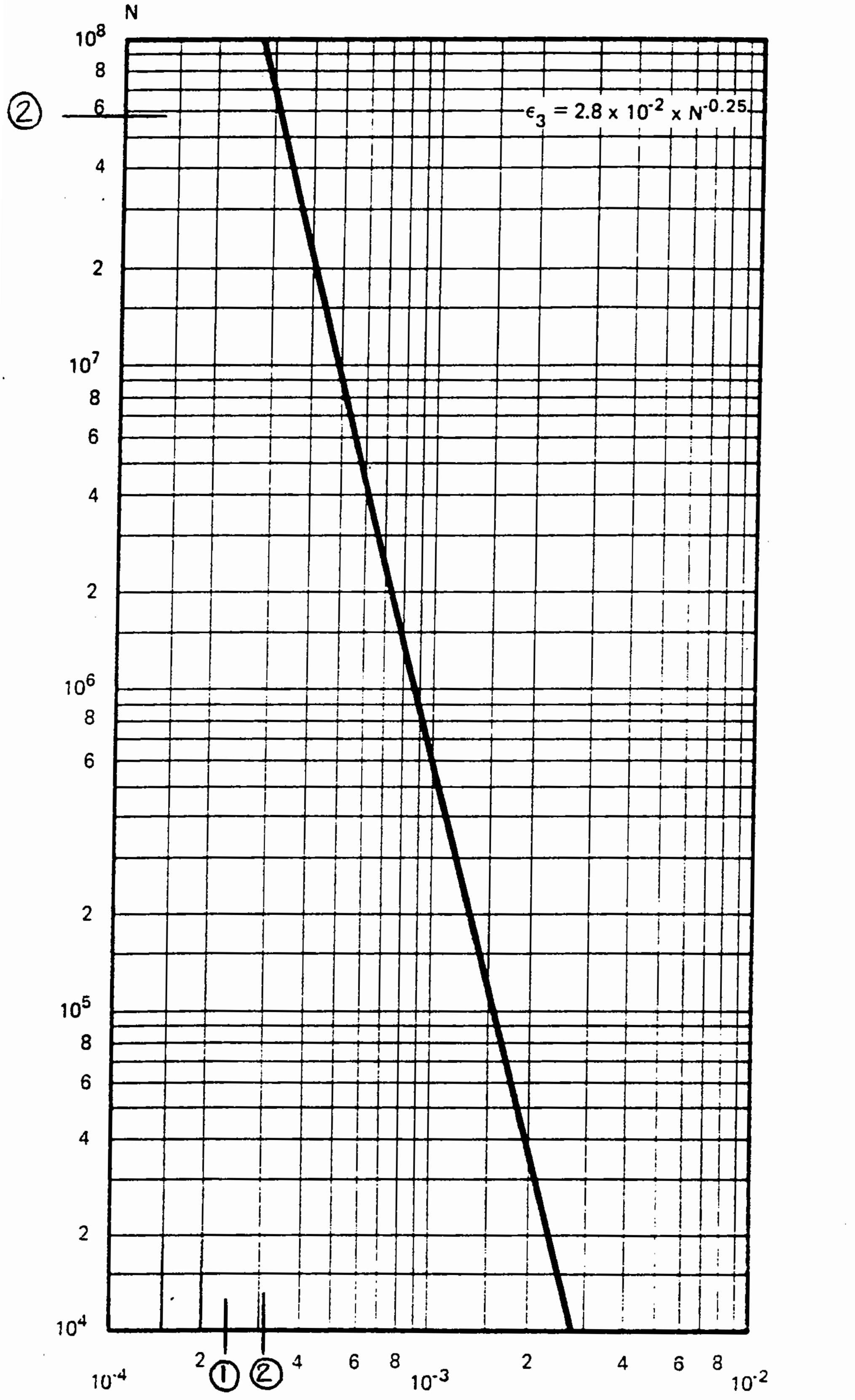

CHART RS

Relationship between permissible subgrade strain and number of load applications

```
Title: COMPARISON OF NOTTINGHAM AND SHRP DESIGNS
==============================================================================


Start time: 11: 4:10.75                                DATE:  9 April     1991
Stop  time: 11: 4:57.27


A.  DESCRIPTION OF THE SYSTEMS               (Systems: 1 to  2   OF  2 Systems)
==============================================================================

SYS.#                             D E S C R I P T I O N
----#--------------------------------------------------------------------------
    #
  1 # NOTTINGHAM MIX FORMULATION
    #
----#--------------------------------------------------------------------------
    #
  2 # SHRP MIX FORMULATION
    #
----#--------------------------------------------------------------------------


B.  CHARACTERISATION OF THE CONSTRUCTIONS   (Systems: 1 to  2   OF  2 Systems)
==============================================================================
Title: COMPARISON OF NOTTINGHAM AND SHRP DESIGNS

SYS.#<<<< LAYER 1 >>>>#<<<< LAYER 2 >>>>#<<<< LAYER 3 >>>>#<<<< LAYER 4 >>>>
----#-----------------#-----------------#-----------------#-----------------
  1 #E-mod: 8800.0 MPa#E-mod:  151.9 MPa#E-mod:   70.0 MPa#
    #Thick:   .150 m  #Thick:   .200 m  #Thick: infinite  #
    #PR   :    .35 -  #PR   :    .35 -  #PR   :    .35 -  #
----#-----------------#-----------------#-----------------#-----------------
  2 #E-mod: 5000.0 MPa#E-mod:  151.9 MPa#E-mod:   70.0 MPa#
    #Thick:   .150 m  #Thick:   .200 m  #Thick: infinite  #
    #PR   :    .35 -  #PR   :    .35 -  #PR   :    .35 -  #
----#-----------------#-----------------#-----------------#-----------------

C.  CHARACTERISATION OF THE LOADS           (Systems: 1 to  2   OF  2 Systems)
==============================================================================
Title: COMPARISON OF NOTTINGHAM AND SHRP DESIGNS

SYS.#<<<< LOAD  1 >>>>#<<<< LOAD  2 >>>>#<<<< LOAD  3 >>>>#<<<< LOAD  4 >>>>
----#-----------------#-----------------#-----------------#-----------------
  1 #LOADING (2 units)#LOADING (2 units)#                 #
    # Load:    20.0kN # Load:    20.0kN #                 #
    # Strs:   577.4kPa# Strs:   577.4kPa#                 #
    #                 #                 #                 #
    #Radius:    .105 m#Radius:    .105 m#                 #
    #X-coor:    .000 m#X-coor:    .000 m#                 #
    #Y-coor:   -.157 m#Y-coor:   +.157 m#                 #
----#-----------------#-----------------#-----------------#-----------------
  2 #LOADING (2 units)#LOADING (2 units)#                 #
    # Load:    20.0kN # Load:    20.0kN #                 #
    # Strs:   577.4kPa# Strs:   577.4kPa#                 #
    #                 #                 #                 #
    #Radius:    .105 m#Radius:    .105 m#                 #
    #X-coor:    .000 m#X-coor:    .000 m#                 #
    #Y-coor:   -.157 m#Y-coor:   +.157 m#                 #
----#-----------------#-----------------#-----------------#-----------------
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D.  POSITIONS AND RESULTS    (Pos.: 1- 5)     (Systems: 1 to  2   OF  2 Systems)
================================================================================
Title: COMPARISON OF NOTTINGHAM AND SHRP DESIGNS

   SIGN convention STRESSES and STRAINS: + = TENSILE     stress or strain;
                                         - = COMPRESSIVE stress or strain.

SYS.#< POSITION 1 >#< POSITION 2 >#< POSITION 3 >#< POSITION 4 >#< POSITION  5>
----#--------------#--------------#--------------#--------------#--------------
  1 #COORDINATES(m)#COORDINATES(m)#COORDINATES(m)#COORDINATES(m)#
    # X-crd:  .0000# X-crd:  .0000# X-crd:  .0000# X-crd:  .0000#
    # Y-crd:  .0000# Y-crd: +.1575# Y-crd:  .0000# Y-crd: +.1575#
    # Depth:  .1500# Depth:  .1500# Depth:  .3500# Depth:  .3500#
    # Layer:layer 1# Layer:layer 1# Layer:layer 3# Layer:layer 3#
    #              #              #              #              #
    #maximum       #maximum       #maximum       #maximum       #
    #STRESSES (MPa)#STRESSES (MPa)#STRESSES (MPa)#STRESSES (MPa)#
    # Hor.X:+1.2652# Hor.X:+1.3317# Hor.Y: -.0015# Hor.Y: -.0018#
    # Vert.: -.0434# Vert.: -.0454# Vert.: -.0250# Vert.: -.0235#
    #              #              #              #              #
    #maximum       #maximum       #maximum       #maximum       #
    #STRAINS (um/m)#STRAINS (um/m)#STRAINS (um/m)#STRAINS (um/m)#
    # Hor.X: +110.6# Hor.X: +108.5# Hor.X: +126.1# Hor.X: +119.7#
    # Vert.:  -90.2# Vert.: -102.8# Vert.: -347.0# Vert.: -324.9#
----#--------------#--------------#--------------#--------------#--------------
  2 #COORDINATES(m)#COORDINATES(m)#COORDINATES(m)#COORDINATES(m)#
    # X-crd:  .0000# X-crd:  .0000# X-crd:  .0000# X-crd:  .0000#
    # Y-crd:  .0000# Y-crd: +.1575# Y-crd:  .0000# Y-crd: +.1575#
    # Depth:  .1500# Depth:  .1500# Depth:  .3500# Depth:  .3500#
    # Layer:layer 1# Layer:layer 1# Layer:layer 3# Layer:layer 3#
    #              #              #              #              #
    #maximum       #maximum       #maximum       #maximum       #
    #STRESSES (MPa)#STRESSES (MPa)#STRESSES (MPa)#STRESSES (MPa)#
    # Hor.X:+1.0153# Hor.X:+1.0911# Hor.X:+.00035# Hor.X:+.00028#
    # Vert.: -.0584# Vert.: -.0627# Vert.: -.0316# Vert.: -.0295#
    #              #              #              #              #
    #maximum       #maximum       #maximum       #maximum       #
    #STRAINS (um/m)#STRAINS (um/m)#STRAINS (um/m)#STRAINS (um/m)#
    # Hor.X: +161.6# Hor.X: +159.3# Hor.X: +169.6# Hor.X: +160.0#
    # Vert.: -128.3# Vert.: -152.3# Vert.: -446.0# Vert.: -414.3#
----#--------------#--------------#--------------#--------------#--------------
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Volume perc. Bitumen	Stiffness Asphalt mix	Fatigue Strain	Fatigue Life
% v	MPa	μm/m	x 1000
8.40	8800.	110.	3020.
12.00	5000.	162.	6110.

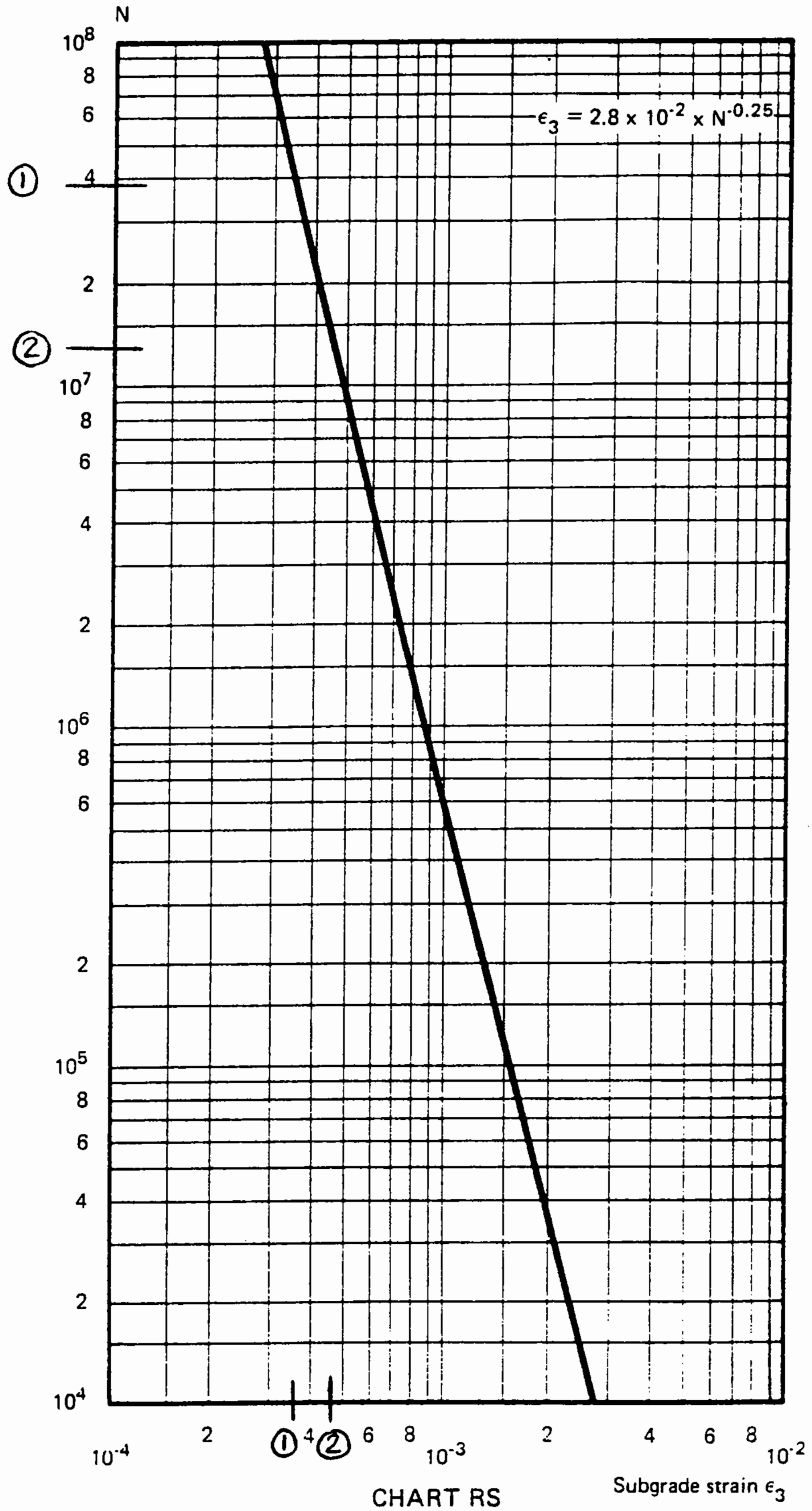

CHART RS

Relationship between permissible subgrade strain and number of load applications