Enabling the Use of Alternative Materials in Road Construction

By

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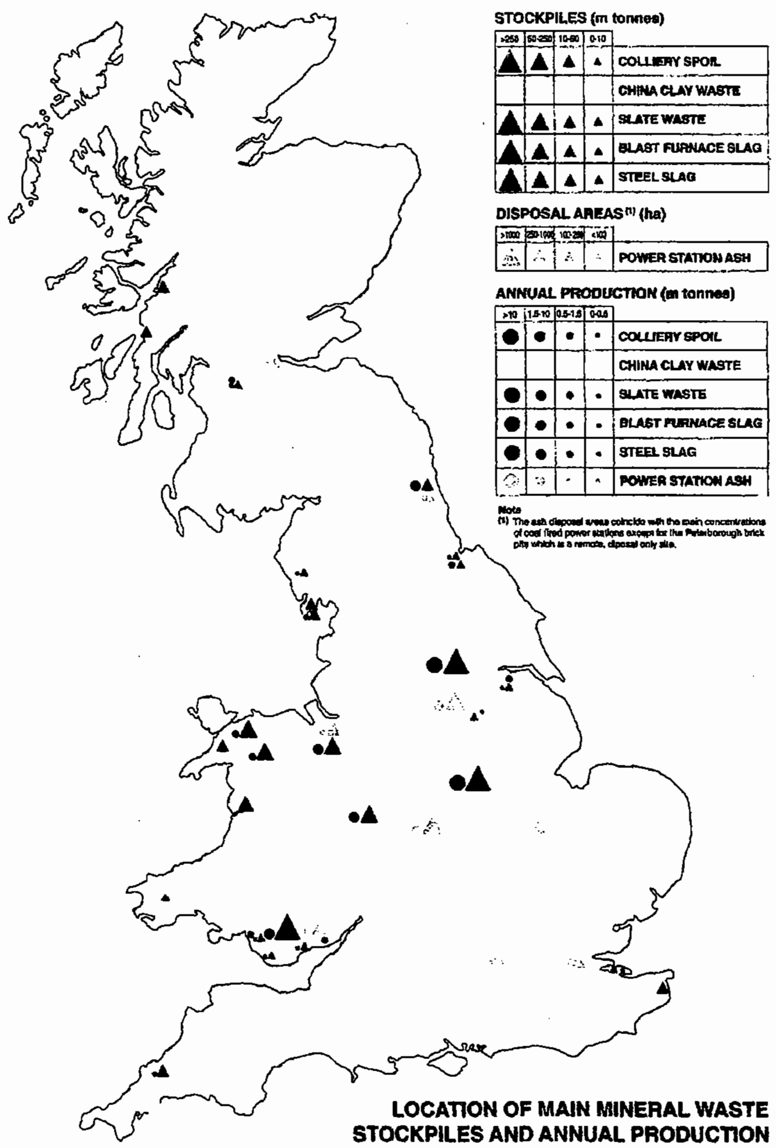

Figure 2.1 Quantities and location of main secondary materials [Whitbread, Marsay & Tunnel, 1991].

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To My Family and in

Memory of My Father

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ABSTRACT

Alternative materials represent an important potential source of aggregates for road construction. At present, their use remains limited owing to the abundance and low cost of high quality natural aggregates, low costs of landfill disposal and generally restrictive specifications for pavement materials. Nevertheless, their influence in relation to the primary aggregates is likely to increase in the future with the enforcement of more strict environmental regulations at national and European levels. The objective of this research project was to investigate a wide range of alternative materials and provide a practical framework for their assessment enabling pavement engineers to deal with most applications in road construction.

Initially secondary materials were assessed according to the present UK specifications and were found to fail the requirements in most cases. However, the current approach does not assess fundamental properties such as stiffness, resistance to permanent deformation, tensile strength, resistance to fatigue and the development of these with time, leading to an inadequate assessment of these materials. To advance towards the development of performance-based specifications repeated load triaxial and indirect tensile tests were used. For their performance the Nottingham 150×300 mm triaxial apparatus was used and an indirect tensile apparatus developed which evolved from the Nottingham Asphalt Tester used for bituminous materials. In triaxial testing, models used to study the resilient behaviour of granular materials were found to give good results for unbound but not for lightly-treated secondary materials. For these, a new resilient model was developed.

Testing and specimen preparation techniques together with performance classification systems were developed for both tests and recommendations for an overall methodology for the evaluation of secondary materials are presented. An application of this methodology was made to the study of thirteen mixtures to be considered for full-scale trials. The application of secondary materials in pavements was evaluated using analytical methods of pavement design which demonstrated the potential of these at levels in the pavement as high as the roadbase and the overall thickness reduction that may result.

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LIST OF SYMBOLS

α	coefficient characterising the reactivity of granulated blastfurnace slag
β	material constant
δ	total horizontal deformation
Δ	increment
δ_{1p}	permanent axial deformation
$\delta_{1p\,(N-1)}$	permanent axial deformation after cycle (N-1)
$\delta_{1r\,(N)}$	resilient axial deformation at the Nth cycle
δ_{3p}	permanent radial deformation
$\delta_{3p\,(N-1)}$	permanent radial deformation after cycle (N-1)
$\delta_{3r\,(N)}$	resilient radial deformation at the Nth cycle
ϵ_1, ϵ_2, ϵ_3	major, intermediate and minor principal strains
$\epsilon_{1p\,(N)}$	permanent axial strain after N cycles
$\epsilon_{1p\,(100)}$	permanent axial strain after 100 cycles
ϵ_{1p}	permanent axial strain
ϵ_{1p}^{*}	corrected permanent axial strain after N cycles
ϵ_{1r}, ϵ_{2r}, ϵ_{3r}	major, intermediate and minor resilient principal strains
ϵ_{3p}	permanent radial strain
ϵ_s	shear strain
ϵ_{sr}	resilient shear strain
ϵ_t	tensile strain
ϵ_v	volumetric strain
ϵ_{vr}	resilient volumetric strain
$\epsilon_{x\,max}$	maximum horizontal tensile strain
ϵ_x	horizontal tensile strain
$\epsilon_{y\,max}$	maximum vertical compressive strain
ϵ_y	vertical compressive strain

ϵ_z	vertical compressive strain
θ	first stress invariant, $\theta = \sigma_1 + \sigma_2 + \sigma_3 = 3\ p_{max}$
ν	Poisson's ratio
ξ	material constant
σ	normal stress
σ_0	tensile strength (monotonic loading)
$\sigma_1, \sigma_2, \sigma_3$	major, intermediate and minor principal stresses
$\sigma_{1r}, \sigma_{2r}, \sigma_{3r}$	major, intermediate and minor repeated principal stresses
σ_c	unconfined compressive strength
σ_H	horizontal normal stress
σ_N	tensile stress resulting in failure after N cycles
σ_t	tensile stress
σ_{td}	tensile strength in direct tension
σ_{tf}	tensile strength in flexural test
σ_{ti}	tensile strength in indirect tension
σ_V	vertical normal stress
σ_x	horizontal tensile stress
$\sigma_{x\ max}$	maximum horizontal tensile stress at the centre of the specimen
$\sigma_{y\ max}$	maximum vertical compressive stress at the centre of the specimen
σ_y	vertical compressive stress
$\tau, \tau_{VH}, \tau_{HV}$	shear stress
ϕ	angle of shearing resistance
a	positive material constant
A_1	material constant
A_{1c}	characteristic permanent strain
ACV	Aggregate Crushing Value
AIV	Aggregate Impact Value
b	positive material constant
B_1	material constant
C_{BX}	correction factor (tensile strain)
C_{BY}	correction factor (compressive strain)
C_{SX}	correction factor (tensile stress)
C_{SY}	correction factor (compressive stress)

Cu	coefficient of uniformity
Cv	coefficient of variation
d	diameter of the specimen
D_{10}	particle diameter at which 10 % of the material by weight is finer
D_{60}	particle diameter at which 60 % of the material by weight is finer
E	Young's modulus
EF_w	traffic load equivalence factor
f	maximum force applied
F	force required to produce ten per cent of fines
f_r	rut factor
G	shear modulus
G_1	material constant
G_a	material constant
GL	gauge lenght
H_0	vertical distance between deformation measuring points
H_M	measured horizontal deformation
I_p	plasticity index
K	bulk modulus
$K_1, ..., K_8$	material constants
K_a	material constant
m	material constant
M	slope of the failure line
m_1	percentage of fines generated during test
M_1	total mass of test specimen
M_2	mass of fines generated during test
M_r	resilient modulus
M_{rc}	characteristic resilient modulus
n	material constant
N	number of cycles
p	mean normal stress
P	vertical load
p^*	parameter related to the failure line, $p^* = S / M$
p_1	initial mean normal stress

p_2	final mean normal stress
p_a	atmospheric pressure, $p_a = 100$ kPa
P_f	friability property
p_m	average 'mean normal stress' during one stress path application
p_{max}	maximum mean normal stress experienced during a stress excursion
p_r	repeated mean normal stress
p'	effective mean normal stress
q	deviatoric stress
q_1	initial deviatoric stress
q_2	final deviatoric stress
q_f	deviatoric stress corresponding to failure
q_m	average deviatoric stress during one stress path application
q_{max}	maximum deviatoric stress experienced during a stress excursion
q_r	repeated deviatoric stress
r	repeatability
R_0	horizontal distance between deformation measuring points
R^2	correlation coefficient square
R_{fat}	fatigue ratio
R_i	compressive strength of the immersed specimens as a percentage of the compressive strength of the control specimens
s	material constant
S	intersection of the failure line with the deviatoric stress axis
S_B	Blaine specific surface of the natural fines fraction (< 80 μm)
SD	Standard deviation
S_m	Stiffness modulus
t	thickness of the specimen
TFV	Ten per cent Fines Value
W	axle load of commercial vehicle
W_s	standard axle load
x	distance to the origin (centre of the specimen)
Y_M	measured vertical deformation

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CHAPTER 1

INTRODUCTION

1.1. Background

The demand for aggregates in the construction industry has been increasing and in 1989, the total annual consumption in England and Wales reached 300 million tonnes [DoE, 1994]. Due to the subsequent recession the consumption declined and in 1991 an estimated 240 million tonnes were utilised. Nonetheless, the upwards trend is likely to continue in the future, being projected to be between 420 and 490 million tonnes by the year 2011 [Whitbread, Marsay & Tunnel, 1991]. Road construction, maintenance and widening activities are the most important consumers of aggregates as illustrated in Figure 1.1, accounting for approximately one third of the total volume.

On the other hand, there is an increasing difficulty in obtaining new planning consents for the extraction of primary aggregates, due either to the difficulty in obtaining sites for new quarries or the public concern related with the environmental impact of conventional aggregate exploitation. Alternative sources of supply need to be used and some of the present possibilities include the use of conventional crushed aggregate from coastal super-quarries, marine dredged aggregates and secondary materials.

Associated with this problem, large quantities of waste are stockpiled around the United Kingdom and more wastes are currently arising from mineral extraction industries (china clay waste, colliery spoils and slate waste), from industrial processes (power station ashes, slags from the steel making industry) and wastes from the construction / demolition industry and from municipal incinerators.

AGGREGATE COMSUMPTION IN THE UK

Source: BACMI, 1991

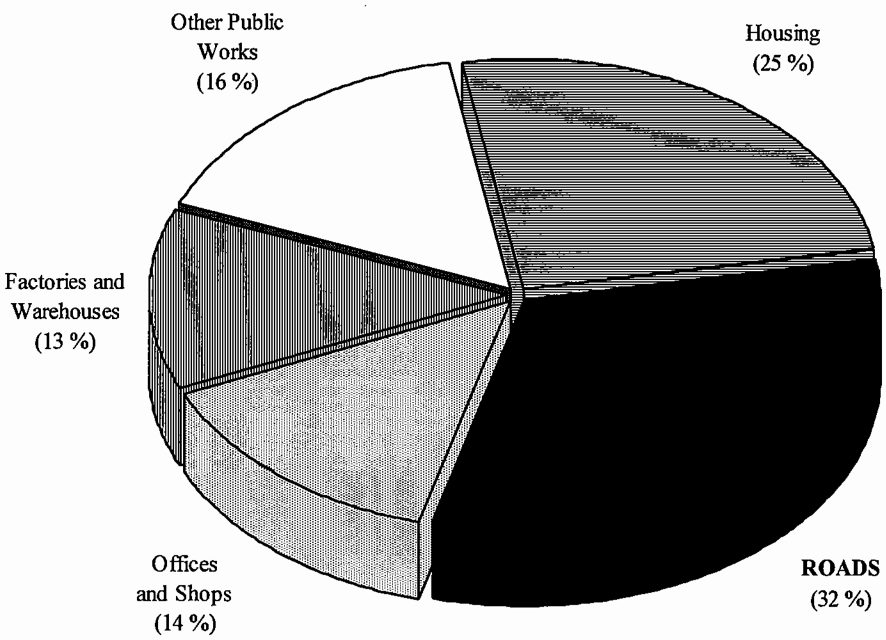

Figure 1.1 Distribution of the demand for aggregates in the construction industry.

The re-use and recycling of secondary materials remain very limited in Great Britain. In 1989 / 90, the estimated use of waste materials for aggregate replacement was 32 million tonnes which compares with a total utilisation of about 332 million tonnes, representing approximately 10 % [Whitbread, Marsay & Tunnel, 1991]. The main factors responsible for this limited utilisation vary from material to material but, in general, they find the following barriers:

- relative abundance and low cost of high quality granular materials, representing a small percentage of the total costs of the road;
- low costs of landfill disposal in comparison with some other European countries;
- inherent variability of many wastes meaning extra costs for processing and testing to assure their adequate performance;
- inability to meet DoT requirements for road construction materials and

inadequacy of existing specifications for direct application to secondary materials;

- collection, storage and processing costs;
- transportation costs from source to areas of major demand such as the South East of England;
- the reliance of engineers on experience with traditional and widely known aggregates.

The factors stated above have contributed to an increasing interest, from the British Department of Transport (DoT), Department of the Environment (DoE) industry in general and research institutions, in identifying methods to stimulate the utilisation of alternative materials in road construction. Additionally, waste management strategies are being prepared and implemented both in the UK and at European Union level.

1.2. Need for the present research

In order to decide whether to use secondary or conventional materials in pavement construction and re-construction both benefits and disadvantages should be weighed. This is not a simple judgement since some of the aspects involved can not be readily quantified in monetary terms. Most of the environmental advantages fall in this class and this is one of the main factors responsible for the evident conservatism and reluctance in the application of waste materials. A global evaluation of secondary materials for re-use and recycling should be based on technical, economic and environmental factors, and due consideration should be given to them:

- availability and economic acceptability - secondary materials need to be available in adequate quantities and at convenient locations (or to be economically transportable to sites) to justify the development of processing plants. Stockpiling or dumping costs should be taken into account in any economic analysis;
- technical adequacy - suitable physical, mechanical and chemical properties of the secondary materials (or economically modified materials) are required in order

to maintain appropriate standards of quality and performance in road construction;

- environmental acceptability - all the materials used in pavements must not be potentially harmful or environmentally damaging during construction and throughout the life of the pavement.

In the technical grounds, the object of this research, the Specification for Highway Works [DoT, 1993] is a compulsory document for all projects in the United Kingdom financed by central government. Its scope is, however, much broader and local Authorities generally draw on the requirements specified by the Department of Transport to develop their own requirements for the local road network. However, these specifications tend to be restrictive and, in some cases, excessively conservative as far as the application of alternative materials is concerned. This is illustrated by the following quotations:

> "*(...) while specifications need to accommodate an adequate margin of safety to ensure that unsuitable materials are not used and that structures are fit for purpose, in some cases these margins appear to be excessive. Examples of such overspecification were found in most types of aggregates use (...)*" [CIRIA, 1995]

> "*At present we write specifications for grading, particle crushing, soundness, etc, but often we do not have sufficient logical grounds for adopting the limits we do. Sometimes, so it appears, the limits are determined more by what the industry is capable of rather than what is theoretically justifiable.*" [Thom, 1988]

> "*Thus, at present, British practice generally requires that if waste materials are to be used in road construction they must satisfy the test requirements that are imposed on naturally occurring aggregates used for the same purpose. This is a logical and defensible approach but it puts waste at a distinct disadvantage because the test methods and the criteria used for acceptance or rejection have, with few exceptions, been developed from experience with natural aggregates.*" [Sherwood, 1992]

Clearly, there is a need for the development of new specifications for the assessment of pavement materials. This will require a redefinition of the evaluation philosophy, in order to assess the materials using properties related with performance rather than using index-type of tests. The new approach applied to the end-products to be incorporated in the pavement structure would enable a more appropriate assessment and comparison with traditional materials.

The use of alternative materials at sub-base and roadbase level represents a value-added application compared with their more frequent use as bulk fill that may represent an important contribution in making these materials competitive against conventional materials and reduce the importance of haulage costs over long distances. For this purpose, some form of stabilisation may be necessary to improve their performance. Also in this area, scope for further research was identified:

> "*The DoE report notes the very many possible materials that may fall within the scope of this proposal. Little is known, however, about the range of mixes that could be successfully employed, the appropriate mix proportions, the properties resulting from those mixes, test methods for identifying suitable/unsuitable materials etc.*" [TRL, 1994]

These materials could be used in road construction in different forms, ranging from totally unbound to heavily stabilised road concrete materials. Considering first the materials modified by light treatment, some lacunae are evident in this area with regards to material performance under repeated loading:

> "*Further research work on the development of an appropriate mathematical model for the weakly cemented material is certainly warranted if structural analysis is to be carried out on pavements incorporating this material. (...) certain areas of research should be targeted:*
>
> - *characterisation of the degree of cementation;*
> - *development of sample preparation procedure to enable good control of the quality and property of the test specimens;*
> - *use of accurate testing equipment which incorporates on-sample stress/strain measuring devices.*" [Chan & Rowe, 1994]

On the other hand, the development of testing methodologies and equipment to perform a simple test capable of assessing the mechanical properties of the materials, consistently and reliably, is of paramount importance. This is clear from the recommendations expressed in a recent report for the Department of Transport [SWK PE, 1994] and with special relevance for the study of secondary materials due to their natural increased variability:

> *"Research is recommended to advance our knowledge with respect to the manufacture and testing of laboratory specimens of hydraulically bound material to properly characterise their mechanical properties (...)"*
>
> *"(...) for the slower cementing, hydraulically bound materials to be more generally adopted, a workable procedure for proper strength and stiffness characterisation of the various mixtures was required to assist in the formulation of pavement designs. For the present report, strength / stiffness has been characterised by cube strength because no better procedure is available now."*

1.3. Objectives

This research focused essentially on the lacunae identified in the previous section. The overall aim was to develop methodologies, equipment, testing techniques and analysis tools to enable and increase the use of alternative materials in road construction. In more detail the objectives were:

- to investigate a wide range of alternative materials for road construction, in order to contribute to a better knowledge of their properties;
- to study means of improving their performance through stabilisation when necessary;
- to propose an overall methodology for the evaluation of secondary materials;
- to develop testing equipment for the performance of selected performance-based tests;
- to develop testing techniques involved in the general evaluation methodology and to draft appropriate test procedures; and
- to propose workable evaluation strategies incorporating the performance of fundamental tests and including whenever necessary the development of
  - a mathematical model for analysis of results and determination of mechanical properties;
  - a classification system for practical use on a routine basis.

1.4. Importance and benefits

The main benefits from this research are expected to be twofold. Firstly, the results will contribute to broaden existing knowledge in the field of alternative materials, especially concerning their laboratory performance under repeated loading using performance-based tests. In due course, this knowledge should contribute to future changes in the DoT Specification for Highway Works and as a result is likely to widen the market for secondary materials.

Secondly, from a more general use of these materials further benefits will result, such as:

- the reduction of the demand for primary aggregates, allowing the preservation of finite resources;
- the reduction of energy costs related to the extraction and transport of conventional aggregates;
- the reduction of the environmental costs associated with conventional aggregate quarrying;
- the reduction in environmental and economic problems associated with waste storage and dumping;
- conservation of natural resources by releasing land that would otherwise be used for quarrying aggregates or dumping waste materials;
- probable commercial benefit from the use of waste materials, since they are already financed by other industrial processes which generate them and, if not sold, they have to be stocked or disposed of, thereby incurring extra costs;
- reduction in taxation cost levied on industrial residues not beneficially used.

1.5. Thesis contents

The thesis is organised in eleven chapters, the first of which consists of this introductory chapter.

In Chapter 2, after an initial overview of the use of secondary materials in several countries, the materials considered for study are selected. A comprehensive literature review covering those materials is then presented in terms of origin of the material, quantities available both as a result of present production and existing in stockpiles, current applications, technical properties and guidelines for their use. Environmental and economic considerations are briefly discussed.

The secondary materials selected are assessed against current specifications for pavement materials, namely the Specification for Highway Works, both in an unbound form (Chapter 3) and stabilised with various road binders (Chapter 4). Due to the failure to meet the DoT requirements in both conditions, new equipment and tests had to be considered for their assessment. In Chapter 5, the present configuration of the repeated load triaxial apparatus is presented and the modifications made during this research identified. Then, the development of a repeated load indirect tensile apparatus is described, which evolved from the Nottingham Asphalt Tester used for the characterisation of bituminous materials.

Chapter 6 focuses on the repeated load triaxial tests and presents the testing techniques used and the method developed for the compaction of the triaxial specimens. A thorough review and critical appraisal of existing models for resilient behaviour is presented and a new model for treated materials developed. Permanent behaviour is also studied using the French model for permanent deformations and its limitations for the materials studied are explained. A classification system is then proposed using the results of characteristic resilient modulus and corrected permanent axial strains.

The second main test considered for the characterisation of stabilised materials in tension was the repeated load indirect tensile test, the use of which is the subject of Chapter 7. The issues involved in this mode of testing are comprehensively discussed and its main advantages and disadvantages more closely compared with the direct tensile and beam flexure tests. The techniques for specimen preparation, testing and analysis of results are described for the determination of the following relevant properties to characterise stabilised materials: indirect tensile strength, Poisson's ratio, stiffness modulus, durability, resistance to fatigue and development of properties with time.

In Chapter 8, the techniques developed in previous chapters and the repeated load indirect tensile apparatus were used to conduct a comprehensive laboratory assessment of thirteen mixtures, with the ultimate objective of selecting six to be used in full-scale trials in the Pavement Test Facility of the Transport Research Laboratory. The testing programme included non-standard compactibility tests, compressive strength of cubes and tensile strength, stiffness, fatigue properties and durability assessment using repeated load indirect tensile tests. A new technique for the compaction of indirect tensile specimens was developed using the standard compactibility test rig. Additionally, a new classification system based on stiffness modulus, indirect tensile strength and resistance to fatigue is proposed and based on this combined approach the final mixtures were then selected.

After an overall discussion of the results presented in Chapter 9, also including an application of the results to pavement design using analytical methods, the overall methodology proposed for the evaluation of secondary materials is outlined in Chapter 10. The final conclusions of this research project are presented in Chapter 11 together with recommendations for future work.

CHAPTER 2

BASIS OF MATERIAL SELECTION

2.1. Introduction

Alternative aggregate materials, also designated as secondary materials, is a term used to encompass all those aggregates formed of, or derived from, wastes, discarded materials or by-products of industrial processes (domestic, industrial, mining and mineral by-products), resulting from human activity, that can be in someway re-used or recycled thereby playing a major role in waste management. The word 'secondary' is used to differentiate from 'primary' aggregates which are derived from geological sources with the essential aim of producing a pavement aggregate.

There is some terminology in the waste materials field that should be clarified since sometimes different terms are used indistinctly for the same product when, in fact, they correspond to different concepts. A first distinction is usually made between by-products and waste materials. By-products are materials generated as a result of the manufacturing process of another product, and in that sense, they are wastes resulting from a production process (for instance, power station ashes and metallurgical slags). Wastes are remaining materials after the consumption cycle is finished, that is, they result from a consumption process (such as municipal incinerator ashes). This distinction has also been made in terms of market value [Bertolini, 1992] by-products being the ones with positive market value and waste materials those having null or negative market value. Sometimes the boundary between both is not clear and the phrase 'waste materials' tends to be used in a broad sense in order to encompass all kinds of waste, although with understandable reluctance from waste producers hoping to find new markets for their residues.

A distinction should also be made between the concept of re-use, recycling and disposal. Re-use assumes that the material is used in the same application or function; recycling presumes selecting and / or processing the material by crushing, breaking or modifying it in order to be used in a different application; and disposal means dumping the material. The incineration of municipal solid wastes should be treated as a way to dispose of materials, unless the resulting ashes are used in some way.

There are many industries generating wastes and thus a wide range of secondary materials is available with potential for use in highway construction. Some of those more frequently referred to in current literature are:

Municipal and domestic wastes

- municipal incinerator ashes: incinerator fly ash and incinerator bottom ash;
- scrap tyres;
- recycled glass;
- recycled plastic;
- waste oils;

Industrial wastes

- by-product gypsum, including essentially fluorogypsum, flue gas desulphurisation (FGD) gypsum and phosphogypsum for road applications;
- cement and lime kiln dust;
- construction and demolition wastes: crushed concrete, crushed brick, crushed rubble, recycled asphalt pavements (RAP), recycled concrete pavements (RCP);
- metallurgical wastes: ferrous slags (blastfurnace slag and steel slag), non-ferrous slags, foundry wastes, steel mill wastes;
- petroleum contaminated soils;
- power station ashes: pulverised fuel ash and furnace bottom ash;
- recycled and waste fibres;
- roofing shingle waste;

- silica fume;

Mining and mineral wastes

- china clay waste;
- colliery spoils: unburnt colliery shale (minestone), burnt colliery shale;
- quarry wastes (for example, those resulting from the extraction and processing of conventional crushed aggregates);
- slate waste;
- spent oil shale;
- mine refuse;
- tailings.

2.2. Overview of the use of secondary materials

In Europe, significant levels of waste materials re-use / recycling have been achieved by many countries such as Belgium, Denmark, France, Germany, The Netherlands and the United Kingdom. A number of countries have developed specifications for these materials and have their own laws regulating waste disposal together with administrative measures to implement their use: in Denmark a tax of about £ 0.25 / tonne was introduced on primary aggregates and a charge of approximately £ 12 / tonne on landfill waste disposal [Whitbread, Marsay & Tunnel, 1991]; in Germany and The Netherlands a proportion of all materials used in a project must be secondary materials [Sherwood, 1995]. A summary of the production and utilisation of some secondary materials in those countries is presented in Table 2.1 [Chi, 1992; Gorlé, 1992; Krass, 1992; Thijs, 1992; Whitbread, Marsay & Tunnel, 1991].

The levels of utilisation are not as high as desirable or possible, and the non-existence or inadequacy of present specifications has been recognised in some countries. In the United Kingdom in particular, the Specification for Highway Works [DoT, 1993] promulgated by the Department of Transport, the Scottish Office Industry Department, the Welsh Office and the Department of the Environment for Northern Ireland, (but usually known as the DoT Specification), is a compulsory specification for all projects financed by central Government. This Specification was written, and successively revised, based on years of experience and research studies essentially with conventional materials for road construction and does not have special requirements for secondary materials. Furthermore, most of those requirements concern the properties of the material to be applied in a particular pavement layer rather than the properties of the final product. Until end-product specifications or performance-based specifications are developed, secondary materials must comply with the existing Specification for Highway Works, otherwise they cannot be applied even if they are perfectly acceptable in terms of performance.

Table 2.1 Summary of the annual production and utilisation of some secondary materials in the EU.

Country			PFA	FBA	IFA	IBA	CS	CDW
B	Production	x10^3 tonnes	850	106	25	300	0	3000
	Volume used	x10^3 tonnes	723	94	0	0	---	---
		%	85	89	0	0	---	---
D	Production	x10^3 tonnes	---	400	270	2600	64800	32600
	Volume used	x10^3 tonnes	---	300	0	1800	13600	3700
		%	---	74	0	69	21	11
DK	Production	x10^3 tonnes	890	100	---	415	---	---
	Volume used	x10^3 tonnes	783	88	---	270	---	3670
		%	88	88	---	60	---	---
F	Production	x10^3 tonnes	2535	447	200	1800	4700	20-25000
	Volume used	x10^3 tonnes	1175	> 224	---	---	7990 *	---
		%	70	> 50	---	---	170 *	---
GB	Production	x10^3 tonnes	10500	2600	---	160	45000	24000
	Volume used	x10^3 tonnes	4600	1140	---	---	2800	11000
		%	44	44	---	---	6	46
NL	Production	x10^3 tonnes	970	100	85	650	---	8000
	Volume used	x10^3 tonnes	970	70	35	46	---	> 4000
		%	100	70	41	7	---	> 50

Key:

PFA	Pulverised Fuel Ash	B	Belgium
FBA	Furnace Bottom Ash	D	Germany
IFA	Incinerator Fly Ash	DK	Denmark
IBA	Incinerator Bottom Ash	F	France
CS	Colliery Spoils	GB	Great Britain
CDW	Construction and Demolition Waste	NL	The Netherlands
---	No information	EU	European Union
*	Considering the use of stockpiled material		

The national efforts are now reinforced by the European Committee for Standardisation (CEN, Comité Européen de Normalisation) of the European Union. There are three CEN Technical Committees working in areas related with secondary materials, under which several Sub-Committees and Working Groups are operating:

- CEN TC 154 - "Aggregates";

- CEN TC 227 - "Road construction and maintenance materials" in particular Working Group 4 -"Unbound, hydraulic-bound, waste and marginal materials" which focus specially on secondary materials for road construction and maintenance; and
- CEN TC 292 - "Characterisation of waste products".

These Committees have been drawing up European Standards which, after formal approval, will replace national standards of all member countries of the European Union. A considerable effort is being done at this level in both producing the necessary standards and specifications and funding research activities that could improve European expertise in this field.

In the United States of America the annual generation of solid wastes is now approximately 4.5 US billion tonnes and continues to increase every year [Collins & Ciesielski, 1992]. A large number of States have developed solid waste regulations and are reviewing and modifying existing specifications in order to allow and increment the use of secondary materials.

Surveys on the use of waste materials and by-products in highway construction, sponsored by the Transportation Research Board and the US Department of Transportation, Federal Highway Administration, were recently conducted [Ciesielski, 1993; Collins, 1993; Ruth, 1993]. All fifty States and the District of Columbia were sent questionnaires with the purpose of evaluating the current practice of those States in terms of waste materials disposal, re-use and recycling. As an example, a summary of the results regarding the utilisation of secondary materials with potential application in hot mix asphalt concrete pavements is presented in Table 2.2.

The results of these surveys are reported essentially in terms of number of States using and researching secondary materials and current practice in terms of re-use, recycling and disposal. The actual volumes of materials currently used and those accumulated over the years remain relatively unknown. Some remarkable aspects resulting from those surveys are [Ciesielski, 1993; Collins, 1993; Ruth, 1993]:

- at least 75 %, on average, of all pavement materials have been either re-used or recycled. This includes materials used in all pavement layers from surface to subgrade and materials used in the shoulders;
- between 75 and 100 % of the asphalt surfaces and asphalt basecourses have been recycled in the majority of States and most of the remaining material re-used;
- unbound granular materials of base and sub-base layers have been typically re-used or disposed in embankments or fills, and stabilised materials have been mainly used in embankments, though other solutions were sometimes employed;
- recycled asphalt pavements and scrap tyres are by far the most popular materials for using in hot mix asphalt concrete pavements, being used by 44 and 38 States respectively, although some of them still consider their use of scrap tyres to be experimental.

Table 2.2 Summary of survey results on waste materials and by-products in the USA [Ciesielski, 1993].

Waste	Annual Quantity million tonnes	No of States conducting research	No of States using
Domestic	**34**		
Household and Commercial Refuse			
Glass	11	7	6
Plastic	13	---	3
Incinerator Ash	7.8	---	2
Scrap Tyres	2.2	34	38
Industrial	**206**		
Coal Ash	65	10	9
Blastfurnace Slag	15	11	13
Steel Slag	7	8	9
Non Ferrous Slags	9	---	1
Reclaimed Asphalt Pavement	91	32	44
Reclaimed Concrete Pavement	3	---	7
Foundry Wastes	9	5	1
Roofing Shingle Waste	7	---	2
Mineral	**1395**		
Waste Rock	925	---	---
Mill Tailings	470	5	7
Total	**1635**		

In the State of New Jersey, USA, where the problem of waste disposal reached crisis proportions in the 1970's and 1980's, legislation was prepared in order to enforce the recycling of as much waste materials as possible and that "*private citizens be required to separate glass, aluminum, plastic and newspapers from their garbage and recycle it through Municipal and County recycling centers*" [Justus, 1993]. Various incentive plans were also developed to encourage the use of specific recycled products, for example, asphalt concrete with 10 % glass (glassphalt). The Department of Transport of New Jersey specified that it would pay $ 1.00 per tonne if this solution was adopted and planned to extend this incentive programme to all construction projects awarded during 1994 [Justus, 1993].

In Canada a survey of Transportation Departments conducted in all ten Provinces plus Yukon and North West Territories, concluded in 1991, demonstrated the use of a wide range of secondary materials [Mackay & Emery, 1993]. The results of the survey, summarised in Table 2.3, also evidence the lack of specifications for those materials.

Table 2.3 Summary of survey results on waste materials and by-products in Canada.

Rank	Material	Users	Specifications
1	Old Asphalt	10	3
2	Old Concrete	4	1
3	Blastfurnace Slag		
	Air-Cooled	2	1
	Pelletised	1	1
	Granulated	1	1
4	Fly Ash	4	1
5	Steel Slag	4	1
6	Silica Fume	4	1
7	Nickel and Copper Slags	2	1
8	Bottom Ash	2	---
9	Mine Waste Rock	4	---
10	Waste Tires	2	---
11	Lime and Cement Kiln Dust	1	---
12	Waste Foundry Sand	1	---
13	Waste Glass	1	1
14	Waste Shingles	1	---
15	Miscellaneous	NA	NA

In conclusion, waste materials management is far from being a localised problem / challenge or specific to any country in particular. On the contrary, a large number of countries are putting a considerable effort into the reduction of wastes generated and into the effective re-use / recycling of waste materials. Another important aspect is that there is still a considerable amount of work to be done in order to generalise their use in the construction industry, with focus on the development of appropriate specifications and design criteria to ensure their adequate performance. Furthermore, the vast majority of secondary materials are currently used in low-value applications and if uses of higher commercial value were found and proven they would further encourage the use of these materials.

2.3. Potential of secondary materials

There is a wide range of secondary aggregates available as described before and a vast field of applications. Some of the most important uses are:

- embankments as bulk and selected fill materials;
- unbound roadbases, sub-bases and capping layers;
- as cement bound material (CBM) for sub-bases and roadbases;
- drainage layers and as lightweight drainage material behind structures;
- gabions;
- pipe bedding;
- block-making industry, namely the manufacture of lightweight blocks;
- production of lightweight aggregate;
- structural concrete and building mortar;
- cement manufacturing industry;
- as a partial replacement of cement;
- as a partial replacement of sand for grading correction;
- as a filler or as an aggregate in bituminous materials.

Some materials have higher potential than others and some may not be adequate for all uses referred before. This will be considered in detail in the next sections where the analysis of each material is carried out.

In general, sub-base and capping layers are privileged layers for their application due to the lower specification requirements in those layers, and because they represent, together with imported fill, the largest requirement for aggregates in pavement construction. To illustrate this the design thickness of capping and sub-base may vary from 150 mm to 750 mm [DoT, 1994].

A summary of the potential for use in pavement foundations of a wide range of secondary materials is presented in Table 2.4 [BS 6543, 1985; Sherwood, 1995].

Table 2.4 Summary of the potential of secondary materials for use in pavement foundations.

Material	Capping layer Unbound	Sub-base Unbound	Cement-Bound Materials
Unburnt colliery spoils	Low	None	Some
China clay sand	High	Some	High
Slate waste	High	High	Some
Pulverised fuel ash	Low	Low	High
Furnace bottom ash	Some	Some	High
Blast furnace slag	High	High	High
Steel slags	Low	Low	Low
Crushed concrete	High	High	High
Asphalt planings	High	High	Low
Demolition wastes	Some	Some	Low
Municipal incinerator ashes	Some	Some	None
Burnt colliery spoils	High	Some	High
Spent oil shale	High	Some	High

Note: Potential classified as *High*, *Some*, *Low* or *None*.

2.4. Selection of materials

Currently, the utilisation of alternative materials in road construction is limited (see Section 2.2). Furthermore, a large number of waste materials is generated. For these reasons, it was decided to study a wide range of materials representative of the main possible sources for road construction. However, a selection was necessary in order to narrow the number of materials to be included. The principal criteria for selection were:

- availability in terms of imbalanced production / utilisation;
- quantities of waste existing in stockpiles (Table 2.5 and Figure 2.1 illustrate the current situation);
- potential for the replacement of primary aggregates in pavement foundations (Table 2.4);
- interest demonstrated from waste producer industries in the study of their products. This aspect was also carefully considered since without the willingness from those industries to contribute to this process through the creation of infrastructures for commercialisation, the distribution of secondary materials and the implementation of quality control schemes, a widespread use of alternative materials will not be possible.

The materials selected comprise both secondary aggregates and secondary binders as shown below. Some materials can be used for both purposes, as a main aggregate and as a binder, namely pulverised fuel ash and gypsum.

Secondary aggregates

- Unburnt colliery spoils ('minestone')
- China clay sand
- Slate waste
- Power station ashes
    - Pulverised fuel ash and
    - Furnace bottom ash
- Air-cooled blastfurnace slag

- Air-cooled steel slag
- Gypsum

Secondary binders

- Pulverised fuel ash
- Granulated blastfurnace slag
- Ground granulated blastfurnace slag
- Gypsum
- Cement kiln dust

These include the most important secondary materials in terms of volume available at the present time in Great Britain, as illustrated in Table 2.5, for which the development of new applications and the intensification of the current ones are a priority. Construction and demolition wastes and asphalt road planings seem to be more effectively used at the moment and the volumes available are reduced.

In the following sections the selected materials are described in terms of the industrial processes which generate them, availability, location, present use in the construction industry and main characteristics of interest for pavement foundations.

Table 2.5 Current volumes of secondary materials in Great Britain [Whitbread, Marsay & Tunnel, 1991].

Materials	**Stockpiles (million tonnes)**	**Annual arisings (million tonnes p.a.)**	**Annual utilisation (million tonnes p.a.)**
Colliery spoil	3600 * 2000 on tips	45	2.8
China clay sand	600	27	1.5
Slate waste	400 - 500	6	0.5
Power station ashes (pfa & fba)	200	13	5.7
Blastfurnace slag	20	4	4
Steel slag	11	2	0.4
Construction & demolition wastes	-----	24	11
Asphalt road planings	-----	7.5	6
Others	100	-----	0.1

Key: * including reclaimed tips.

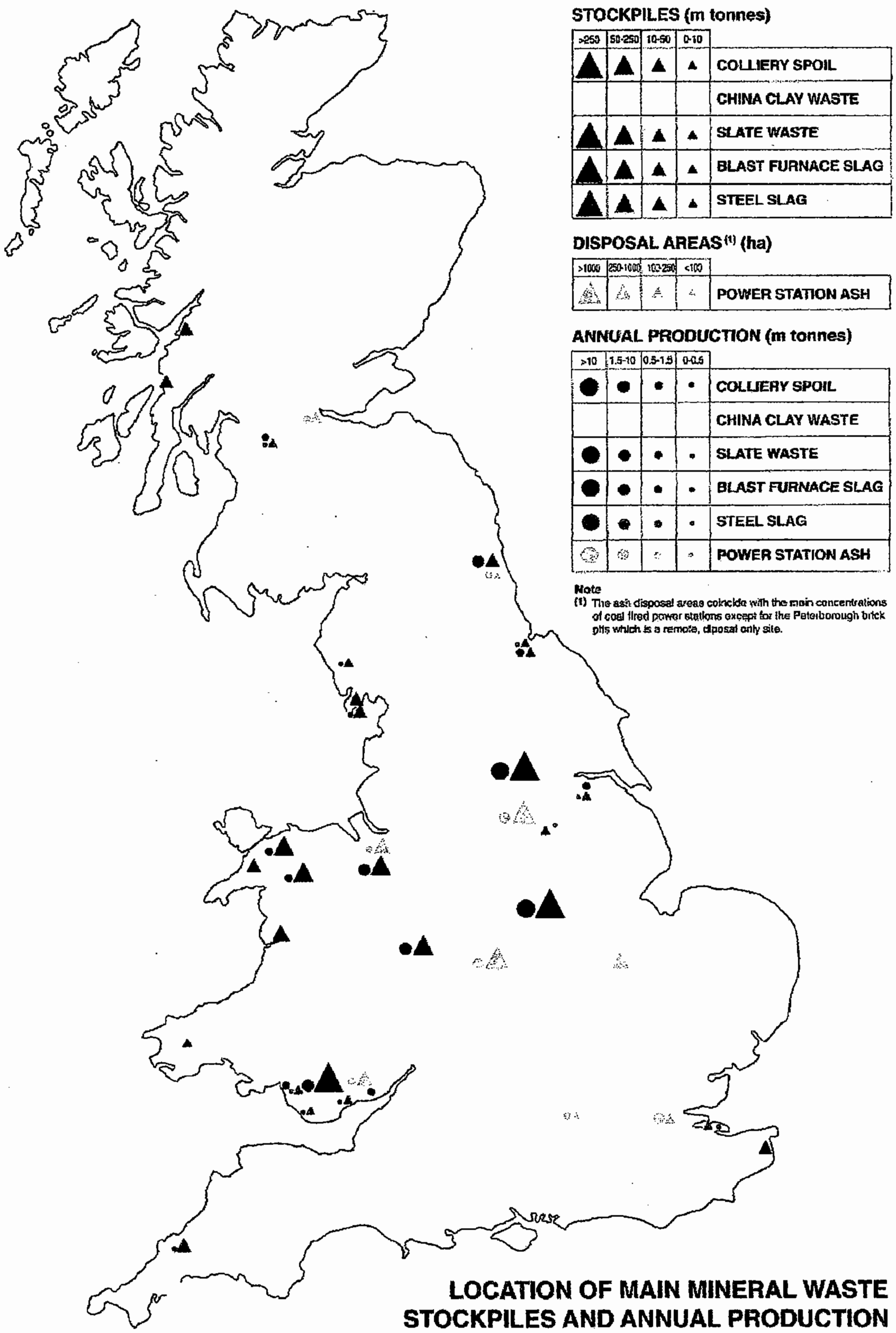

Figure 2.1 Quantities and location of main secondary materials [Whitbread, Marsay & Tunnel, 1991].

2.5. Colliery spoil

2.5.1. Origin

Colliery spoil is the waste produced by the coal mining industry, mainly composed of mudstones, shales and seatearths. The process of extracting the coal from deep mines with the constant need to reach and maintain the access to the coal seam generates a considerable amount of spoils. However, the most important proportion of waste is extracted from the mine with the coal brought to the surface and separated from it in the coal cleaning plant.

There are two main types of colliery spoils: unburnt and burnt. Burnt colliery spoils result from the spontaneous combustion of the raw material, and are becoming more scarce due to factors such as [Sherwood, 1995]:

- improvements in coal / waste separation techniques reducing the amount of combustible material remaining in the spoil;
- safety aspects taken into consideration for the construction of spoil heaps, decreasing the probability of spontaneous combustion;
- higher demand of this material for road construction.

Unburnt colliery spoil, commercially known as minestone, is in general a very variable material in composition as a result of tipping in the same heaps all spoils arising from mining activity. This is the material which will be studied in this project and, thus, special attention will be given to its characteristics rather than to the ones of burnt colliery spoil.

2.5.2. Quantities and location

According to a report published by the Department of the Environment (DoE)

[Whitbread, Marsay & Tunnel, 1991] the annual arising of colliery spoil is estimated at 45 million tonnes and despite the decline in the coal mining industry, this material is likely to continue as the largest waste material available in Great Britain, as a result of past coal mining activities. The proportion of waste to coal generated rose from 4 % in 1920 to 53 % in 1988 / 89, increasing significantly between 1950 and 1970 with the introduction of mechanised methods of mining [Sherwood, 1995]. In terms of volume, it is estimated the existence of some 2000 million tonnes (3600 including reclaimed tips) of colliery spoil in stockpiles in England and Wales [Whitbread, Marsay & Tunnel, 1991] forming by far the largest source of waste material. The material is available in the coalfields, with the largest volumes in Yorkshire, Nottinghamshire and South Wales.

2.5.3. Current utilisation

In total just some 2.8 million tonnes are used each year [Whitbread, Marsay & Tunnel, 1991] representing 6.2 % of the current production. This compares with levels of utilisation of 21 % in Germany and 170 % of the total production in France, taking into account material used which was already stockpiled (Table 2.1). Naturally, these percentages should be considered together with actual volumes of material which, in Great Britain, are considerably larger.

Colliery spoil has been used in road construction mainly as a bulk fill material, where less demanding requirements of existing specifications permit the use of both burnt and unburnt colliery spoils. However, minestone is specifically excluded from all applications as selected granular fill, selected cohesive fill (except when used as fill for structures), as unbound or stabilised capping material, and as unbound sub-base material [DoT, 1993]. On the other hand, burnt colliery spoil possesses, in general, better physical and mechanical properties and for this reason may be used in roadbases and sub-bases of pavements. Another possible application is as cement bound material for sub-bases and roadbases since their requirements do not exclude in advance any material, specifying, instead, its properties and those of the stabilised end-product.

2.5.4. Properties

This material exhibits a certain number of characteristics which have to be carefully studied if it is to be applied in road construction. The main problems that can preclude its use concern:

- heterogeneity in quality and composition, essentially due to the process of waste generation. Nowadays, modern production techniques have reduced this problem to a certain extent and some processing is already being carried out, mainly in terms of grading control (Figure 2.2). For the large volumes of material already in stockpiles the problem still remains;

Figure 2.2 Processing of minestone at Gascoigne Wood Mine, Yorkshire.

- wet weather placement is sometimes referred to as a problem, leading to its exclusion or imposing extra costs [Whitbread, Marsay & Tunnel, 1991];
- high sulphate content mainly with burnt colliery spoil, in which sulphates can occur in large concentrations as a result of the oxidation of pyrites during

combustion [Sherwood & Ryley, 1970]. This is not generally a problem with minestone [Rainbow, 1989]. For determining the sulphate content one must bear in mind that when the material is to be used in a stabilised form the important requirement is the determination of the total sulphate content (in terms of acid-soluble content), and when utilised in an unbound form, the water-soluble sulphate content should be determined because these reflect the availability of sulphate in these different uses;

- sulphides content in the form of iron pyrites. Unlike sulphates, they can occur in appreciable concentrations in the unburnt colliery spoil [Rainbow, 1989] and less significantly in the burnt spoil (because of the oxidation of pyrite to sulphates during combustion);
- high frost susceptibility, more important for burnt colliery spoil than for minestone [Sherwood, 1995].

The presence of high concentrations of sulphates and/or pyrite in the colliery spoil result in considerable expansion of the stabilised material. Carr and Withers [1987] identified two different types of expansion in the case of cement stabilisation:

- short term expansion, as a result of the hydration of the clay minerals within the raw material. It is possible to minimise this expansion by mixing sand in high percentages (> 30 %) with the other components; and
- long term expansion, due to sulphate attack to the cement matrix of the stabilised material. In this case the use of binders such as sulphate resisting cement and pulverised fuel ash are beneficial.

The short term durability is essentially controlled by properties of the minestone such as: particle size distribution, plasticity, sulphate content and slaking resistance [Thomas, Kettle & Morton, 1987]. Although these index properties may be used to indirectly estimate the expansion and the effect of immersion, long term durability and geotechnical stability studies are essential for a complete evaluation of cement-stabilised minestone with regards to durability [Thomas, 1986]. Seven-day immersion periods are reported not to be sufficient to assess the full extent of the chemical attacks which may take months rather than days.

Unburnt colliery spoils are excluded from several applications in the construction of pavement layers as referred above. The exclusion as unbound material is justified because the coarse particles are, in general, aggregations of smaller particles, resulting in questionable long term particle stability [Sherwood, 1995]. However, the prohibition of using it as stabilised capping seems strange since the same material can be applied in the more demanding sub-base layer as long as it meets all requirements for cement bound materials.

The durability test specified for cement bound materials, or immersion test, should be able to anticipate any problem of the stabilised material and, thus, its exclusion for capping layers, in advance, cannot be justified. In this standard test, the compressive strength of two identical sets of specimens is determined: one set after 14 days of curing in the standardised method, at constant moisture content, and the second, after immersion for 7 days after 7 days of standard curing. The index R_i is defined as the compressive strength of the immersed specimens as a percentage of the compressive strength of the control specimens and is a measure of the effect of immersion in water.

Some burnt colliery spoils possess pozzolanic properties, equivalent to those manifested by silicoaluminous pulverised fuel ashes, which have not been fully considered in pavement construction [Chi, 1992].

2.5.5. Technical guidelines

For a preliminary assessment of minestone for cement bound materials the following guidelines are indicated [Rainbow *et al.*, 1984]:

- sulphate content < 1 % - suitable for stabilisation;
- particle size distribution - passing 75 μm < 20 %;
- target moisture content used in the field - OMC + 2%.

With respect to the levels of stabilisation necessary, some of the percentages of added cement suggested in the literature are reproduced below:

- 5 to 10 % by weight [Rainbow *et al.*, 1984];
- 5 to 10 % for sub-base and base layers [BS 6543, 1985];
- 6 to 10 % required for stabilisation to be used as a base material [OECD, 1977];
- 10 % [Kettle, 1990] to meet the strength requirement for soil cement which was a minimum unconfined compressive strength of 2.8 MPa at 7 days [DoT, 1986].

It is worth stressing that the referred limit value of 2.8 MPa contrasts with 4.5 MPa required for CBM1 by the present DoT Specification for Highway Works [DoT, 1993], corresponding to a substantial increase in the strength requirement for cement bound materials. The values of the unconfined compressive strength were found to be affected by particle size and mould size, with higher strengths achieved for lower particle sizes and larger moulds [Kettle, 1990]. The latter is a result of increased particle fracture when compacted in smaller moulds.

The mixing moisture content has been reported to influence the strength and durability of cement-stabilised minestone [Thomas, Kettle & Morton, 1990]. Although in the short term moisture contents above optimum levels reduce strength, in the long term moisture contents of up to 2 or 3 % above produce higher strength and resistance to the effect of immersion, freeze-thaw and wet-dry cycles. On the other hand, water contents below optimum level lead to a considerable reduction both in terms of strength and durability. Carr & Withers [1987] also found that minestone was suitable for stabilisation when compacted at the maximum dry density using a moisture content ranging from the optimum moisture content (OMC) and OMC + 2 %.

Thus, it is important to carry out a strict site control regarding moisture content and density, in order to assure a successful application of minestone. Because this is not always the case, cement-stabilised minestone is best suited for less demanding applications or sites with short design life where small expansions may not be a problem.

2.6. China clay waste

2.6.1. Origin

Geologically, china clay or kaolin is formed during the cooling process of granite, resulting from the action of steam and carbon dioxide on the orthoclase feldspar fraction in the granite. Its extraction is achieved by applying high-pressure jets of water against the faces of the rock in open pits. This process breaks the rock forming a slurry which is pumped to a processing plant. At this stage the material goes through two different processes: first, to separate the bigger particles from the sand waste, and second, to separate fine clayey sand and mica residue from the china clay sand by de-watering the residual slurry (Figures 2.3 and 2.4).

China clay is used in the ceramic and paper industries and, as in many other mineral extraction industries, the exploitation process gives rise to the generation of large quantities of waste materials. These wastes are generated at a ratio of 1 tonne of china clay to 9 tonnes of wastes composed as follows [OECD, 1977]:

- 3.7 tonnes of coarse sand;
- 2 tonnes of waste rock;
- 2 tonnes of overburden;
- 0.9 tonnes of micaceous residue.

Apart from being the principal component of wastes, china clay sand has good engineering properties (see following sections) and, therefore, more potential for use in road construction.

2.6.2. Quantities and location

In terms of volume, china clay waste represents the second largest waste material in the UK.

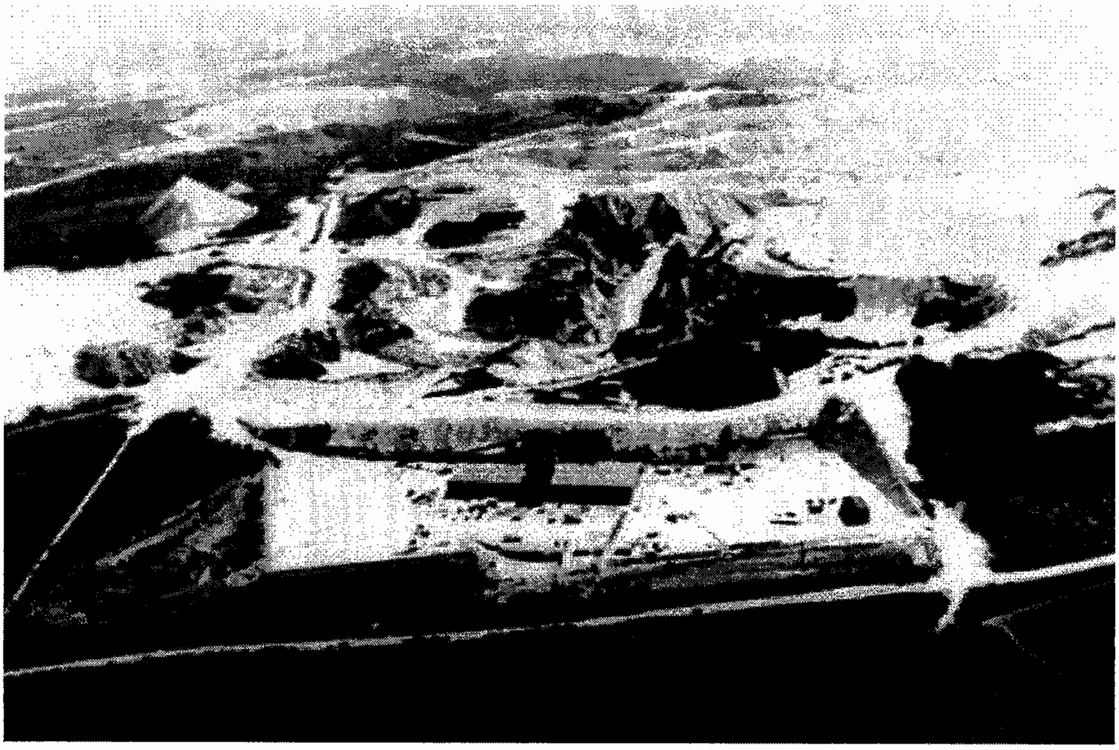

Figure 2.3 China clay workings (courtesy of Camas Aggregates).

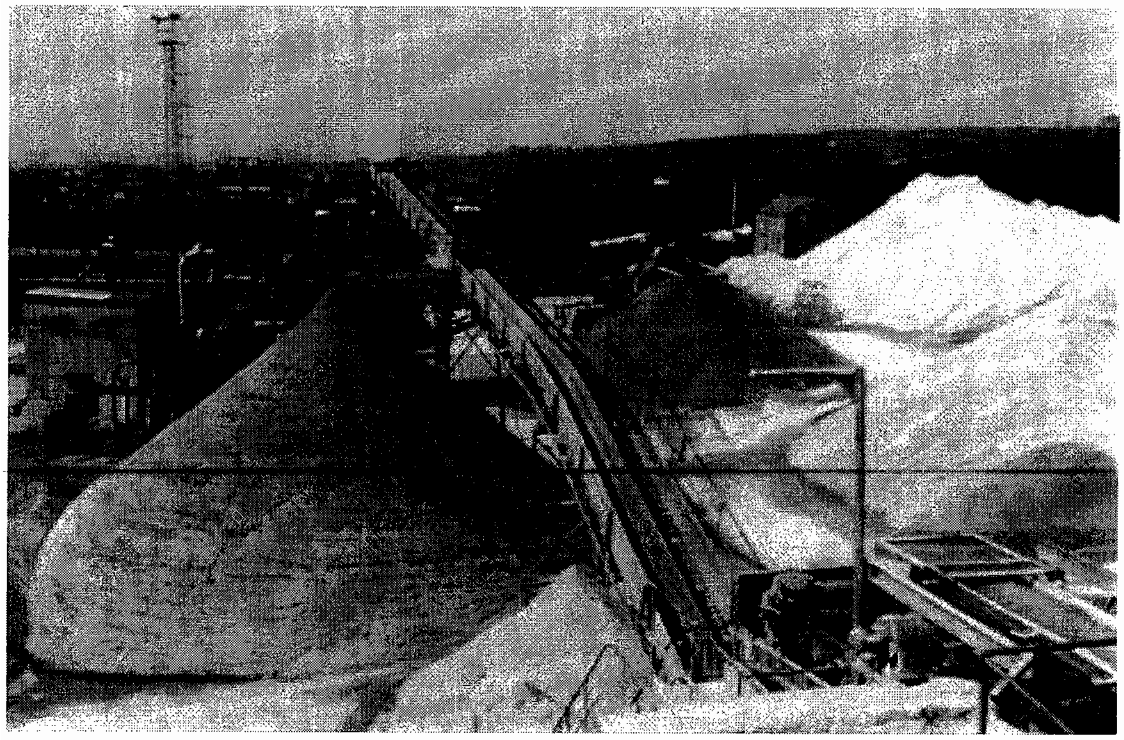

Figure 2.4 Processing of china clay sand (courtesy of Camas Aggregates).

It is located in the south-west of England, in Cornwall and Devon areas and it is estimated that some 600 million tonnes of material exist in stockpiles and 27 million tonnes are still produced each year [Whitbread, Marsay & Tunnel, 1991].

2.6.3. Current utilisation

According to the same report of the DoE, there is an estimated annual volume of material used in the construction industry of 1.5 million tonnes. The main reason for this small utilisation is its remote location in an area where the demand for aggregates is very limited.

The field of applications of china clay sand in the local construction industry includes its use in the block-making industry for the production of calcium silicate bricks, in structural concrete and in building mortar. Specifically in the construction of local roads it has been applied in embankments, unbound sub-bases and stabilised with cement in roadbase layers.

2.6.4. Properties

The properties of this material are very similar to a conventional sand with some special aspects related to the presence of mica. Some difficulties to compact the material have been reported [OECD, 1977] principally with sands having high mica content. Despite this, china clay sand is chemically inert, that is, no problem is expected due to its chemical composition.

Mica is formed by any of a group of minerals consisting of hydrated aluminium silicates, which may contain in addition small amounts of elements such as titanium, iron and magnesium. It has an unusually flat particle shape and a layered structure with special characteristics of resilience when loaded perpendicularly to the cleavage planes.

The effects of mica on the properties of road construction materials have been studied by Tubey and Webster [1978] using china clay sands from different sources, and artificially increasing the mica content by adding different quantities of fine and coarse mica. To separate the effects of the mica content from those of the change in grading as a result of the addition of fines, samples of china clay sand were also studied replacing the addition of mica with the addition of silicas of a similar particle size distribution. In summary, the following conclusions were drawn:

- particle size distribution tests making use of sedimentation methods cannot be used with materials containing mica due to its shape, structure and ratio surface area / volume (specific surface area). Mica particles are very thin differing significantly from the spherical shape for which Stokes' law governing sedimentation techniques apply. Moreover, in particles with multiple flakes, considerable amounts of liquid can be retained between individual flakes. Finally, the specific surface area is high and as a result mica particles can float in liquids with low density;
- the addition of mica decreases the density obtained after compaction for a given compactive energy and, consequently, a reduction in strength (using the California Bearing Ratio test - CBR) was observed. This is due to the properties of resilience of the mica which allows the material to deform and recover part of the deformation after compaction, but principally because of the change in grading resulting from the addition of fines. This effect is more significant with coarse mica;
- the presence of mica does not increase the material degradation under compaction;
- fine mica increases frost heave beyond the values that can be expected due to the change in grading. It is suggested that fine mica increases the number of pores with capillary dimensions, thereby, augmenting suction and facilitating water penetration into the material;
- the influence of the presence of mica on the strength of cement or lime-stabilised china clay sands is limited to the effects resulting from the changes in densities. When adequately compacted, the presence of mica does not have any detrimental effect;

- in conclusion, the usual percentages of mica in china clay sands are low and, therefore, it is not probable that mica will constitute a problem for most applications as a road construction material.

Other characteristics of poor workability and the requirement of high cement content for stabilisation have also been reported [Whitbread, Marsay & Tunnel, 1991]. Although some china clay sands may require relatively high cement contents in order to be used as cement bound material, which are not necessarily the ones with the highest mica content, they can normally be economically stabilised [Tubey, 1978].

China clay sand has already been investigated in the past as a road construction material by means of laboratory tests to determine the characteristics of typical samples and complemented by information resulting from previous experience on the use of this material in the South-West of England [Tubey, 1978]. The laboratory phase consisted essentially in performing those tests currently used for the characterisation of both unbound and cement bound materials in order to verify their suitability against existing specifications, namely: particle size distribution, specific gravity, compaction tests, CBR tests, breakdown under compaction, plasticity, frost susceptibility and 7-day compressive strength tests. Even though for one of the china clays tested the percentage of fines for selected granular fill use and frost heave value fell outside the limits of the DoT Specification [DoT, 1993], in general, the investigation confirmed the suitability of the material for most uses in road construction.

2.6.5. Technical guidelines

A range of stabilisation levels for china clay sands is suggested in the literature for bases and sub-bases. Percentages of cement of 6 % are indicated in the British Standards [BS 6543, 1985] as a first estimate when there is neither experience from previous work nor other information is available. More usual proportions are considered to be between 2 and 4 % [Tubey, 1978].

2.7. Slate waste

2.7.1. Origin

Slate waste is the waste resulting from the slate quarrying industry. The main application for slate has been as a roofing material and, for this reason, only the rock suitable for splitting is used. All the weathered rock near the surface, material with poor cleavage or with joints closely spaced, and fragments from blasting result in waste production [Watson, 1980]. Furthermore, the subsequent processes of splitting and trimming the large blocks of good slate for the production of roofing slates generate further large quantities of waste. The overall ratio of waste to final product is about 20 to 1 and, as a result, huge quantities of waste are accumulated in stockpiles.

2.7.2. Quantities and location

The estimated volume of slate waste in Great Britain is between 400 to 500 million tonnes in stockpiles resulting from past production, with approximately 6 million tonnes arising annually [Whitbread, Marsay & Tunnel, 1991], being responsible for the third largest volume of dumped waste in Great Britain (Figure 2.5).

Slate waste is concentrated in remote areas of the country around the quarries. North Wales has been by far the main producing area. Other areas where the material is available are: Scotland, the Lake District and Cornwall.

2.7.3. Current utilisation

The utilisation of slate waste has been very limited both in quantity and in area. Only approximately 0.5 million tonnes is currently being used, which is just a small percentage of the annual production, leaving all the stockpiled material untouched.

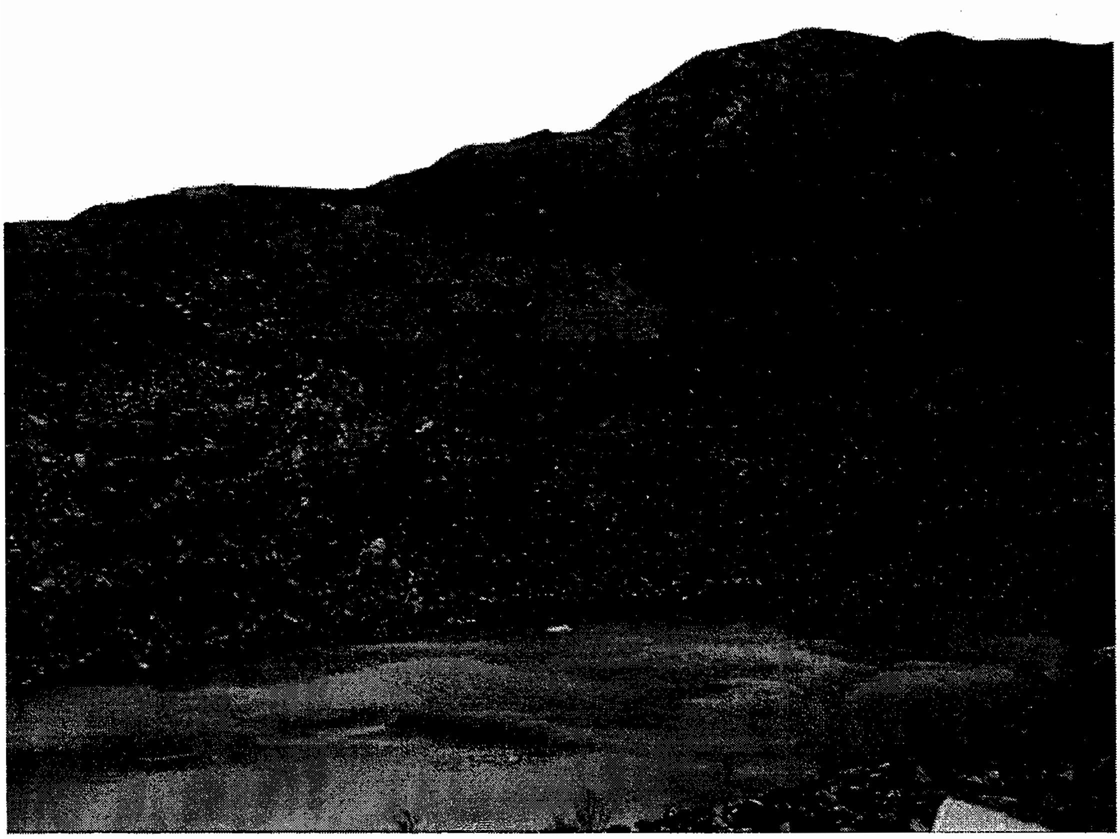

Figure 2.5 Slate waste extraction at Penrhyn Quarry, Gwynedd, North Wales.

The main applications of this material are as pipe bedding, drainage material, embankment fill, capping layers and, over the past ten years, as Type 1 aggregate for unbound granular sub-bases. The latter has been produced at Penrhyn Quarry and applied in roads in North Wales.

2.7.4. Properties

Goulden [1992] carried out a study on slate wastes from various sources performing a comprehensive conventional testing programme. Initially slate wastes from six different quarries were studied for various physical, chemical and mechanical properties and, due to the consistency of the material, the slate waste from Penrhyn Quarry was chosen as a representative sub-base Type 1 material for further testing. The main conclusions that resulted from this study were:

- the particles of slate waste exhibit high flakiness and elongation indexes;

- no evidence was found in compaction trials of detrimental effects of the high flakiness and elongation values. In particular neither excessive transient deformation nor rutting was observed during the passage of the compaction roller indicating good internal friction of the material;
- water absorption values were particularly low, which is in accordance with the use of slate as a roofing material, and so the compacted material is not likely to be affected by moisture content oscillations;
- this low water absorption, combined with the non-plastic nature of the fraction under 425 μm, gives slate waste an advantage over other conventional aggregates with regards to weathering resistance and durability. This was also confirmed by the results of the magnesium sulphate soundness test, where very high percentages were obtained;
- particle strength values were lower than what may be expected for other conventional aggregates;
- high permeability assures good surface water drainage;
- to be considered non-frost susceptible [Roe & Webster, 1984] the percentage passing 75 μm has to be limited to 6 %;
- values obtained in CBR tests were typical of good quality pavement aggregates and above the minimum of 30 % specified for Type 2 sub-bases [DoT, 1993].

Slate aggregate has been used as a sub-base material mainly in North Wales. One of the roads in which it was applied failed prematurely as a result of heavy container traffic exploiting deficiencies in the overlying wet mix macadam (comprising a conventional aggregate). During reconstruction some tests were carried out to assess the performance of the slate material performance in the field [Goulden, 1992]. Those tests consisted of:

- measurements of the sub-base reaction modulus using plate bearing tests; and
- grading analysis of the sub-base material after seven years in service.

The sub-base reaction moduli varied from 378 to 646 MPa [Goulden, 1992] confirming the good condition of the slate aggregate. The degradation of the material was very small and at levels less than might be expected for conventional aggregates. It remained stable during reconstruction.

2.7.5. Technical guidelines

This aggregate is processed (by selection and crushing, Figure 2.6) as a granular sub-base Type 1 and Type 2 according to the DoT Specification [DoT, 1993] and it does not differ significantly from other conventional aggregates in this respect. The main controls that have to be made at the source concern the grading of the aggregate to meet the DoT limits and the percentage of fines (passing 75 μm) that has to be limited to approximately 6 % to assure non-frost susceptibility.

For the compaction of this aggregate in the field, vibratory rollers have been found to be superior to smooth-wheeled or grid rollers producing higher densities and being able to compact thicker layers [Goulden, 1992].

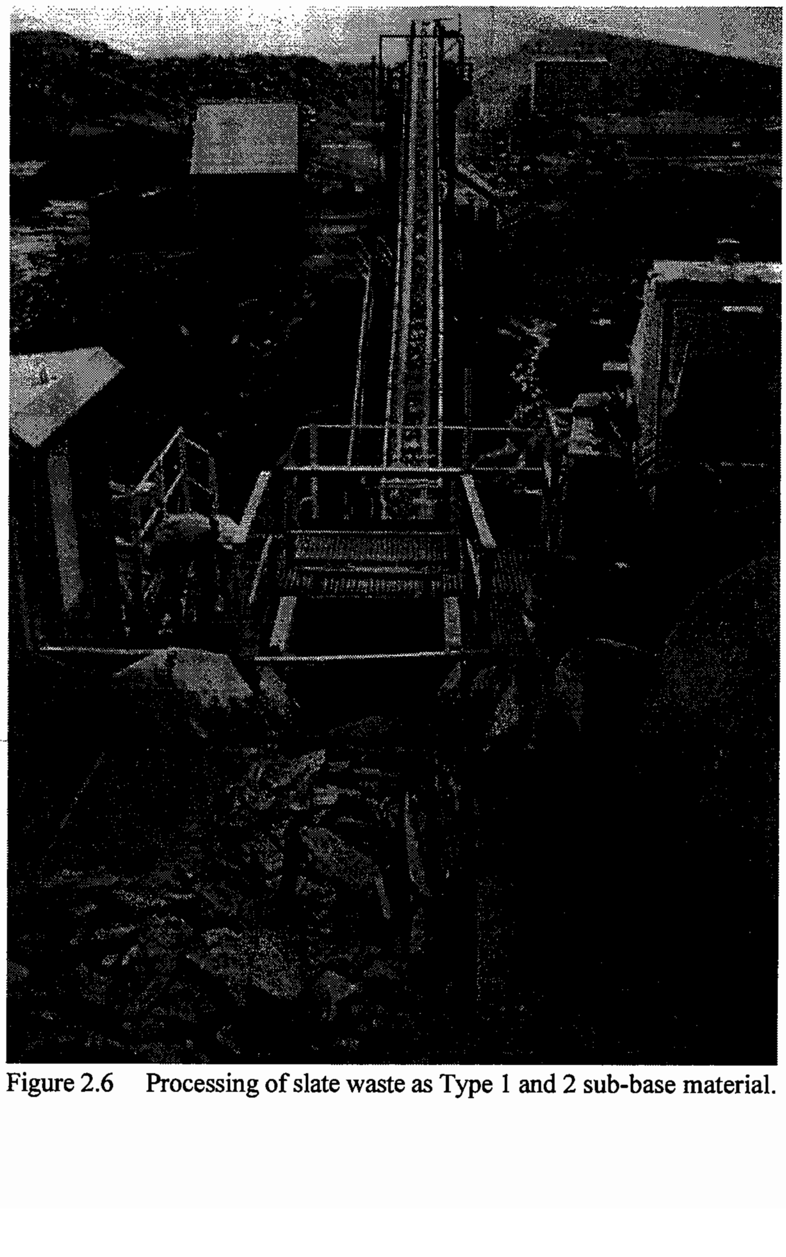

Figure 2.6 Processing of slate waste as Type 1 and 2 sub-base material.

2.8. Power station ashes

2.8.1. Origin

Power station ashes are by-products of the coal-burning power generation industry. Coal-fired power stations generally use pulverised coal which is burnt in a furnace generating two types of ashes. The first is a very fine ash which is carried off the furnace by the flue gases and extracted from them using mechanical arresters or electrostatic precipitators. This ash is called pulverised fuel ash or fly ash. The second type of ash is known as furnace bottom ash. It is formed by the coarser and heavier particles of ash, which fall to the bottom of the furnace into a hopper.

Physical characteristics of power station ashes depend on factors such as: power station (type size and arrangement of furnace) where they are generated, the source, type and degree of pulverisation of coal used, conditions of firing and collection and processing methods. The source and type of coal are one of the most important factors.

2.8.2. Quantities and location

According to the DoE report mentioned previously [Whitbread, Marsay & Tunnel, 1991], in 1988/89 the total production of ash was 13.2 million tonnes, fly ash accounting for 75 to 80 % of the total amount and bottom ash for the remaining 20 to 25 %. According to the same report, the estimated volume of ash stockpiled is 200 million tonnes, whilst the volume annually used in the construction industry is 5.7 million tonnes.

Coal fired power stations are essentially concentrated in the South Yorkshire, Nottingham and Derby areas close to the coal mines and hence power station ashes are most readily available in these locations. Nevertheless, other power stations exist around the country and, in addition, many stations that no longer generate electricity have stocks of ash.

2.8.3. Current utilisation

Pulverised fuel ash is used, primarily, in the manufacturing of lightweight blocks. But there are some other important applications such as: in the cement manufacturing industry; in structural concrete, as a partial replacement of cement or as a partial replacement of sand for grading correction purposes; in the production of lightweight aggregate; as a filler in bituminous materials; as bulk and selected fill materials; and in stabilised sub-base and base layer construction.

Some problems with the use of stabilised fly ash in sub-bases and roadbases have been reported [Thijs, 1992]: in Denmark it has not been used successfully due to "*technical and economical factors*"; in The Netherlands failures due to the formation of multiple horizontal layers occurred in cement-stabilised fly ash, some years after construction, and for this reason it is not used for this purpose. After a reported failure by liquefaction of a conditioned fly ash stockpile, its use for road embankments in Belgium has been restrained by legislation [Gorlé, Verhasselt, Thijs & Kerkhof, 1992]. This situation was overturned by means of laboratory and field research.

Furnace bottom ash is mostly used in the block-making industry for the production of lightweight blocks (as fly ash), as lightweight drainage material behind structures, and for specific backfilling operations. Formerly, it found use in pavement foundations as capping and sub-base but increasingly restrictive specifications have limited its use in these areas. Car parks and cycle tracks are examples of its continuing use in pavements.

2.8.4. Properties

2.8.4.1. Pulverised fuel ash

The main components of pulverised fuel ash are silicon dioxide (SiO_2), aluminium oxide (Al_2O_3) and iron oxide (Fe_2O_3). Typical chemical compositions are shown in Table 2.6. Based on the type of coal and the content of those three compounds, fly ashes are classified in the American Standards as follows [ASTM C618, 1989]:

- class F, silicoaluminous fly ash, with pozzolanic properties, usually produced from anthracite or bituminous coal, and having a minimum (SiO_2 + Al_2O_3 + Fe_2O_3) content equal to 70 %;
- class C, sulphocalcic fly ash, with pozzolanic and hydraulic properties, usually produced from lignite or sub-bituminous coal, and having a minimum (SiO_2 + Al_2O_3 + Fe_2O_3) content equal to 50 %.

Table 2.6 Typical chemical composition of pulverised fuel ash.

Component	%
$S_i\,O_2$	48.5
$Al_2\,O_3$	25.5
$Fe_2\,O_3$	12.1
Ca O	2.5
Mg O	1.5
Na_2 O	1.0
K_2 O	3.0
Ti O_2	1.1
S O_3	1.0
Cl^-	0.06

Data supplied by Powergen, Ratcliffe-on-Soar Power Station, Ratcliffe, Nottingham.

Fly ashes classified as class C have higher percentages of free lime (some may contain more than 10 % and as high as 60 % [Thijs, 1992]) having cementitious properties in addition to the normal pozzolanic characteristics of class F fly ash. For this reason, pulverised fuel ash class C, also called hydraulic fly ash, can also be used as an activator of secondary binders.

The percentages of SiO_2 and Al_2O_3 together with the percentage of calcium oxide (CaO), allow the representation of fly ashes in a triangular diagram. Figure 2.7 is an example of this representation, including the chemical composition of several binders [SETRA & LCPC, 1980].

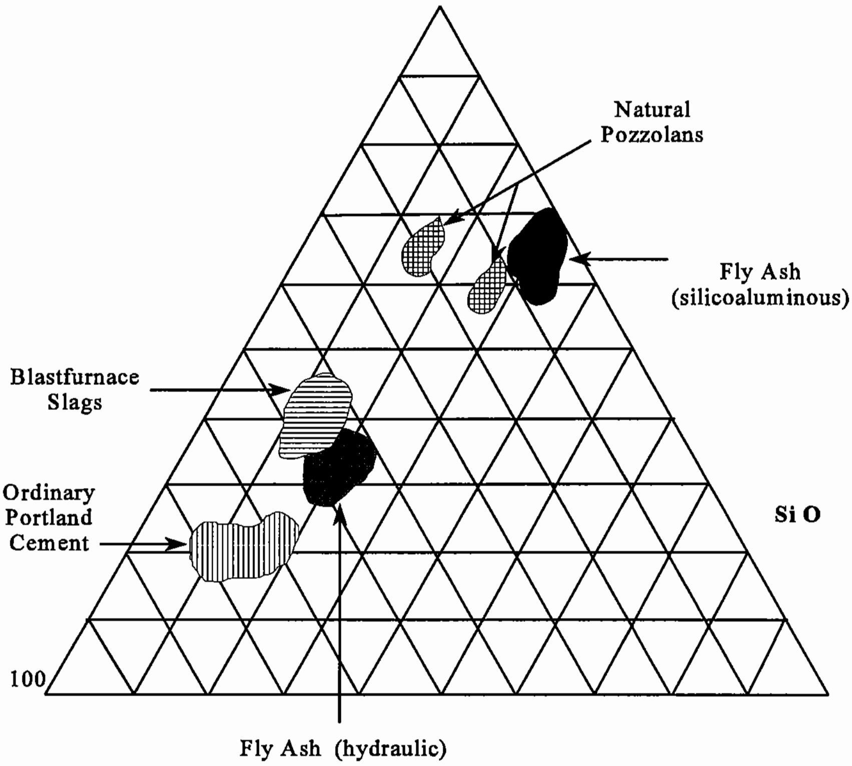

Figure 2.7 Triangular representation of several binders [Setra & LCPC, 1980].

Chemically fly ash does not pose major problems [Sherwood, 1995]:

- it contains sulphates, sometimes in high concentrations. Nevertheless, as the permeabilities of compacted fly ash are low, the possibilities of problems arising from the presence of sulphates are more limited. In the case of sulphocalcic fly ashes [Thijs, 1992], which may contain high sulphate contents (up to 12 % has been reported), their addition in the manufacturing of cement has to be limited to avoid the risk of swelling;
- it contains free lime and, consequently, many fly ashes are alkaline. When in contact with metals such as aluminium fly ashes can provoke corrosion.

An important issue is the method used to store pulverised fuel ash. That procedure affects the final properties of the material and some sort of temporary storage cannot be avoided since the peaks in production and consumption are not simultaneous. On the one hand, production increases during winter when more energy is required to be generated to meet the consumer's demand, on the other hand the highest levels of activity in the construction industry are observed in summer, when production is at its lowest level. Pulverised fuel ash can be stored and then supplied in the following forms:

- in the dry state, directly from the storage silos where it is collected;
- conditioned, that is with controlled amounts of water added to avoid dust during storage and transport (Figure 2.8); or
- from lagoons, sometimes used for fly ash storage.

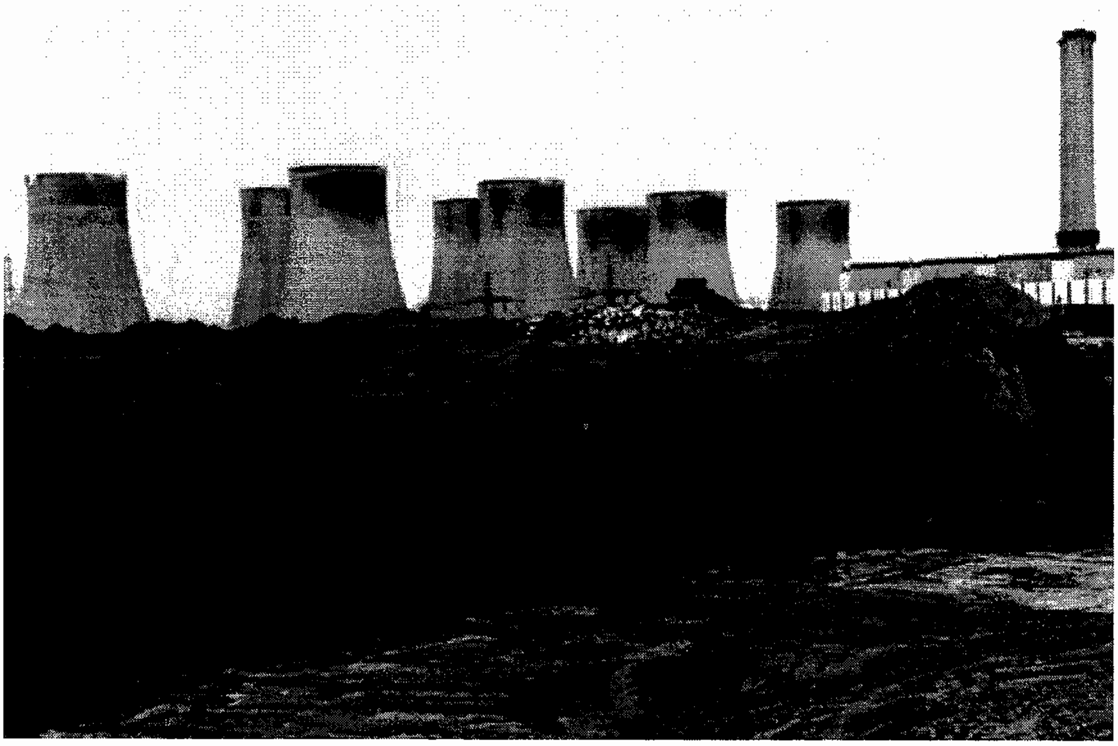

Figure 2.8 Conditioned fly ash, Ratcliffe-on-Soar Power Station, Nottingham.

When stored in dry conditions the best uniformity and quality is achieved but very high costs are involved in the construction of appropriate silos. When conditioned fly ash is to be used in pavement applications its moisture content needs to be closely monitored since excessive water contents may result in premature hydration of calcium oxide. This is of major importance when fly ashes with high CaO content are to be used. Besides, variations in moisture may cause variations in the moisture content of the mix, reduce workability and inhibit a suitable combination of mix components. More precautions are necessary when using lagoon fly ashes due to their higher variability. They are coarser than dry or conditioned fly ash and their coarseness increases with proximity to the discharges. This type of fly ash is often mixed with furnace bottom ash.

When used in concrete several benefits can be expected from the inclusion of fly ash although it must be taken into account that some of those benefits will not be observed in some cases depending on the fly ash and cement used and field conditions. A general list of benefits is presented below:

- enhanced workability due to the shape and fineness of the particles;
- decreased bleeding as a result of the reduction in water demand;
- lower shrinkage;
- reduced heat of hydration;
- increased long term strength;
- improved chemical resistance of concrete (for example to chemical attacks of sulphates and alkalies) and, thus, more durable concretes;
- lower permeability;
- possibility of lower costs by the partial replacement of cement.

For use in pavement construction, one of the most attractive characteristics of the pulverised fuel ash is its pozzolanity. A pozzolana can be defined as a siliceous and aluminous material, not having cementitious properties, but containing certain components which, at ordinary temperatures and in the presence of water, chemically react with calcium hydroxide forming compounds with high stability in water and with cementitious properties [LNEC, 1962].

The pozzolanic reactions can only develop after hydration has commenced, an aspect that affects the performance of the mixtures. Hence, these materials do not show any binding properties unless combined with others, usually called activators. These have high percentages of calcium oxide which produces the required calcium hydroxide in the presence of water. Pulverised fuel ash reacts with that compound, producing further cementitious materials and producing higher strength, stability and durability in the long term. Therefore, materials treated with fly ash exhibit reduced early age strength, but higher strength in the long term when compared with materials stabilised with hydraulic binders.

The pozzolanic activity of fly ashes and their fineness are closely related, and it is generally accepted that the finest fly ashes have the highest pozzolanic activities and initial bearing capacity [Bolt, 1987]. The percentage of material retained on the 45 μm sieve and the Blaine specific surface are good indicators of that activity.

For use in stabilised sub-bases and roadbases, the use of pulverised fuel ash may have significant benefits:

- possibility of cost and energy savings;
- grading correction;
- increase in the long term strength;
- improvement of the resistance to chemical attacks, for example of sulphates;
- the ability to re-cement cracks in the presence of moisture, when surfaces remain in contact and when unreacted fly ash and activator are still available.

On the other hand, some precautions are necessary when used as a secondary binder to allow the development of the minimum strength which is required before being able to support in-service conditions. Cold weather significantly delays strength developments, and in this case the use of cement is preferable to the use of lime to increase short term strength. In addition, the moisture content used for compaction should be maintained during the curing period to ensure an adequate development of chemical reactions and, hence, strength gains. This may be assisted by sealing the stabilised layer, using bituminous emulsions.

Pulverised fuel ash also has some advantageous physical effects when used for the stabilisation of soil and granular materials. The shape of the particles, predominantly spherical, have a plasticising action resulting in a substantial reduction of water demand; the low particle density has the effect of increasing the relative volume of cementitious materials in the mix. These characteristics can help to reduce the effects of the lower early-age strength.

Materials stabilised using fly ash usually have better cracking behaviour than cement-stabilised materials. This is because the pozzolanic reactions with calcium hydroxide produced during the hydration of cement are slower than the hydraulic reactions developed whenever cement is used as stabiliser.

In addition to its pozzolanity, some fly ashes possess self-cementing properties after being compacted. In spite of these properties, it is believed that unbound fly ash is not adequate for capping layer construction and thus will not be appropriate for sub-base layers because of [Thijs, 1992]:

- high frost susceptibility;
- the possible loss of bearing capacity experienced when this material becomes in contact with water, for which the failures by liquefaction reported earlier in this section are an example; and
- the very low values obtained from soaked CBR tests [Littleton & Willavise, 1992].

The particle density of fly ash is very low when compared with other conventional materials and when compacted produces a lightweight layer, important when the road is constructed on highly compressive soils.

2.8.4.2. Furnace bottom ash

Furnace bottom ash has been studied at the University of Nottingham for the last few years with focus on those properties of relevance for its application as granular sub-base material. A considerable amount of work has been done involving laboratory testing, by both conventional characterisation and classification tests and repeated load triaxial tests. The likely performance in the field has been investigated by studying bottom ash in trial strips and in the Pavement Test Facility of the University of Nottingham, with special emphasis on permanent deformation characteristics (rutting), self-cementation properties and material degradation under compaction. This work has been amply reported in technical reports and conference papers [Dawson, Brown & Thom, 1989; Dawson, 1989;

Bullen & Dawson, 1990; Dawson & Bullen, 1991; Dawson & Nunes, 1993] and a summary of the main conclusions are presented below.

From the chemical point of view bottom ash is very similar to fly ash. However, it differs significantly in physical and mechanical properties, due to the coarser grading of the bottom ash. The material has a more uniform grading curve (with less coarse and less fine material) than the conventional Type 1 aggregate. This contributes to a rather poor permanent deformation behaviour and to the difficulties in compaction but, on the other hand, generates high permeabilities and a high toleration to an increase in fines resulting from the degradation of the material. The high permeability will permit free-drainage of the compacted sub-base and wet-weather workability. Most conventional Type 1 materials are not free-draining [Dawson, Brown & Thom, 1989].

Low Ten per cent Fines Values are obtained but no degradation or mechanical instability was perceived during compaction and trafficking - probably an effect of the prevalent grain size and grading [Dawson, 1989].

Bottom ash has low density, important where the stress levels applied to the underlying soils are critical. It also leads to lower transportation costs for a given volume required.

When compared with conventional aggregates, compaction to very low air void contents is difficult. The addition of water and the use of pneumatic-tyred compactor may be beneficial. Bottom ash has reasonable, although not large, laboratory CBR values for appropriate levels of compaction.

Furnace bottom ash has reasonable resilient modulus but not high when compared with conventional aggregates. On the other hand, bottom ash has only a mediocre resistance to permanent deformation especially when tested in dry conditions. When wet, its performance is similar to a conventional aggregate. In order to limit permanent deformation the density should be maximised.

Overall, the problems associated with a less broadly-graded aggregate having rather weak particles appear to be offset, to a large extent, by the advantages deriving from its smaller mean particle size and high permeability.

2.8.5. Technical guidelines

2.8.5.1. Pulverised fuel ash

Several guidelines for the application of pulverised fuel ash in pavements have been developed in several countries. In The Netherlands technical and environmental guidelines have been formulated for the application of cement bound fly ash in road bases based on road trial sections. The methodology entails the performance of a suitability test for material approval, laboratory tests and environmental guidelines as follows [Bolt, 1987]:

Suitability test

- Proctor cylinders are compacted at 3 % below the optimum moisture content, using 1000 g of fly ash and 150 g of Portland cement. The amount of water necessary is mixed with fly ash 3 days before testing;
- specimens are stored at 20 °C in plastic bags;
- compressive strength is determined after 7 days curing and should be higher than 3 MPa;

Laboratory test

- proctor cylinders made as for suitability test using percentages of cement equal to 6, 10 and 14 % (proportions of the fly ash dry mass);
- the required cement content for a roadbase is the one corresponding to a compressive strength of 5 MPa after 28 days of curing;

Environmental guidelines

- roadbases where fly ash is used should be above the ground water table and under an impermeable surfacing.

The compaction at 3 % below the optimum value seems to be low due to the considerations presented before concerning the possibility of loss of bearing capacity and frost susceptibility. The results will not reflect the full potential of the material.

In Belgium the following acceptance criteria for dry fly ash used as a pozzolana were drawn [Gorlé, Verhasselt, Thijs & Kerkhof, 1992]:

- loss on ignition $\leq$ 7 % (preferably 5 %);
- Opticompact test - h $\geq$ 20 mm [CRR, 1988];
- SO_3 content $\leq$ 1.5 %.

Concerning levels of treatment, there is a wide range of possibilities according to the type of fly ash, its intended application and criteria adopted which can be more or less conservative:

- percentages of cement between 5 and 10 % are generally suggested for the stabilisation of pulverised fuel ash for sub-bases and base layers [BS 6543, 1985; Thijs, 1992];
- fly ash with 3 to 5 % of quicklime or 5 to 10 % cement and compacted at the optimum moisture content are reported to achieve compressive strengths between 3 and 9 MPa, splitting tensile strengths between 0.4 and 1 MPa and resilient moduli between 3 and 5 GPa [Gorlé, Verhasselt, Thijs & Kerkhof, 1992];
- when used for the stabilisation of other materials in combination with lime, typical proportions range from 12 to 14 % of fly ash and 3 to 5 % of lime. However, mixes with adequate quality for base layers have been produced using 8 to 15 % of fly ash and 2 to 8 % of lime [Boles, 1986]. To improve early strength 0.5 to 1.5 % of Portland cement may be used as well.

In order to reduce the problem of low initial strength and accelerate the strength development process some additives may be used [Gorlé, Verhasselt, Thijs & Kerkhof, 1992]. In Belgium 1 to 2 % of calcium chloride is added, resulting in faster strength development, improving at the same time the long term compressive strength (50 to 100 %), immediate frost heave resistance and workability. In France gypsum is used with the same goal.

For in situ compaction, pneumatic-tyred rollers are reported to achieve the best results with mixtures having fly ash as aggregate, and vibratory rollers are recommended for coarser materials [Thijs, 1992].

2.8.5.2. Furnace bottom ash

Due to the relative modest performance of this material under repeated loading in terms of resilient modulus and permanent deformation its use in an unbound form should be restricted to low trafficked roads. For its use in the principal road network some form of stabilisation is required.

This material should be compacted at relatively high densities to minimise the poor permanent deformation behaviour and achieve a reasonable load carrying ability. For this purpose, the use of pneumatic-tyred compactor may be beneficial [Dawson & Bullen, 1991].

2.9. Blastfurnace slags

2.9.1. Origin

Blastfurnace slag is a by-product of the iron manufacturing industry, produced when iron ore is transformed into metallic iron (called pig iron at this phase of the production process) in a blastfurnace.

The blastfurnace is continuously fed with iron ore, coke (used as a fuel) and fluxing stone (limestone or dolomite). Pre-heated air is blown into the blastfurnace, working at temperatures between 1300 and 1600 °C, and as a result the oxygen reacts with the carbon from the coke producing heat and carbon monoxide. Iron ore is formed of oxides of iron, silica and alumina. On the one hand, the oxides of iron combine with the carbon monoxide giving rise to metallic iron and carbon dioxide. On the other hand, silica and alumina react with the resulting compounds from the calcination of the fluxing stone, which are calcium and magnesium oxides, producing slag [Lee, 1974]. At this stage, the slags form a liquid layer that floats on top of the liquid iron inside the furnace and are then removed using different notches at different levels in the blastfurnace.

There are gases under pressure in the liquid slag and the method used for cooling it, together with its chemical composition, will determine the amount and size of bubbles that are not able to escape from the liquid before solidification and, consequently, will influence properties such as porosity and density.

Blastfurnace slag is produced in three main forms:

- air-cooled;
- foamed or expanded;
- granulated and pelletised.

In the case of air-cooled slag the liquid slag is poured into pits and left to cool slowly in the open air and, when it solidifies, but it is still hot, water is spread on the material in order to develop cracks. These will facilitate its breakage, crushing and screening according to normal quarry procedures, producing a material very similar to a natural rock in texture and gradation.

For the production of granulated slag the liquid slag is suddenly quenched in water or steam causing its fragmentation. The formation of crystals is hindered, due to the speed of the cooling method, producing a glassy material in small granules (0 to 3 mm).

Pelletised slag is produced by spraying controlled quantities of water onto the stream of molten slag, and projecting the mixture into an atmosphere super-saturated with water, using a drum turning at high speed. The plastic slag is then divided into particles ranging from 0 to 13 mm. The larger material is formed of expanded particles of lower density. It is possible to control the proportion of material under 3 mm produced by modifying the conditions of operation. This fraction of material is similar to granulated slag and is used as a binder.

Foamed or expanded slag is obtained by cooling the molten slag using small quantities of water, in jets into the liquid slag while it is discharged into a reservoir. This method produces large quantities of steam within the molten slag thus forming an expanded material, with very low densities.

Figure 2.9 illustrates the manufacturing process of both blastfurnace and steel slags.

2.9.2. Quantities and location

An estimated 20 million tonnes of blastfurnace slag exist in stockpiles, and 4 million tonnes of this material are currently produced each year, all of which is actually used [Whitbread, Marsay & Tunnel, 1991]. Approximately 72 % of the total production consists of air-cooled blastfurnace slag, 25 % of granulated slag and around 3 % of foamed slag.

This material is produced by British Steel integrated steelworks in four different locations in Great Britain namely, Teesside, Scunthorpe, Llanwern and Port Talbot.

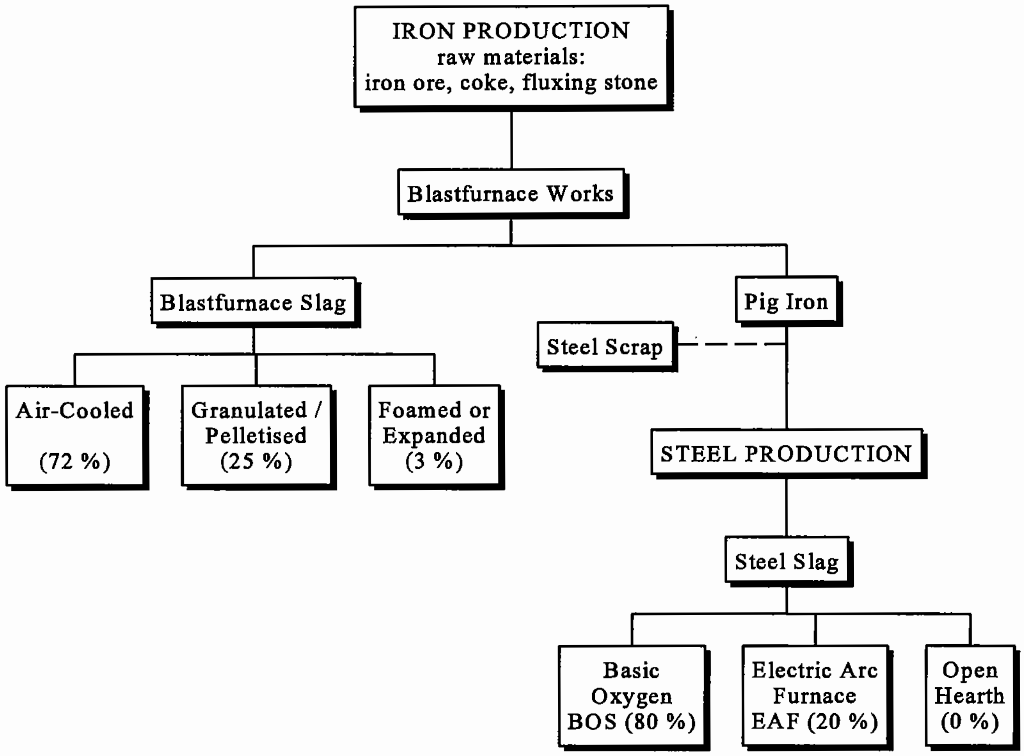

Figure 2.9 Manufacturing process of blastfurnace and steel slags.

2.9.3. Current utilisation

The main uses of blastfurnace slags in the United Kingdom are in road construction as an aggregate, for cement manufacture in blended cements, and in general building as a lightweight aggregate. However, this material is widely used in other applications, especially in some European countries where special techniques and composite materials have been developed. End-applications for this material include:

- cement manufacture, as referred above, where granulated and pelletised blastfurnace slags may be included in percentages from 5 % up to 85 %;

- in road construction it may be used at different levels of the pavement structure, including surfacing layers, as an aggregate and as a binder for the stabilisation of other materials;
- in lightweight blocks manufacture, made of pelletised slag and cement;
- in different types of concrete;
- in railways as ballast, essentially crushed air-cooled blastfurnace slag;
- in embankments particularly as lightweight fill;
- drainage blankets;
- as a sand for blasting; and
- in others less significant applications.

2.9.4. Properties

It is composed primarily of oxides of calcium and magnesium (CaO and MgO) with silica (SiO_2) and alumina (Al_2O_3) accounting for approximately 95 % of the slag composition [Lee, 1974]. Typical composition and ranges are presented in Table 2.7.

Table 2.7 Typical chemical composition of blastfurnace slag [Lee, 1974].

Component	**% by mass**
Lime (CaO)	36 - 43
Silica (SiO_2)	28 - 36
Alumina (Al_2O_3)	12 - 22
Magnesia (MgO)	4 - 11
Total sulphur (as S)	1 - 2
Total iron (FeO or Fe_2O_3)	0.3 - 1.7

Chemically this material does not constitute any significant problem and unrestricted use in pavement applications have been suggested [Baldwin, Addis, Clark & Rosevear, 1995]. Besides, blastfurnace slags are produced complying with quality requirements specified by the British Standard Institution [BS 1047, 1983; BS 3797, 1990; BS 6699, 1992] further limiting the possibility of chemical problems for materials complying with those specifications. Some of the specification requirements for air-cooled blastfurnace slag with importance for pavements are listed below:

- bulk densities of compacted material in the range 10 to 14 mm, must not be lower than 1.1 Mg/m^3;
- stability against two types of unsoundness (iron unsoundness and 'falling' or 'dicalcium silicate' unsoundness);
- total sulphur content must not be greater than 2.75 % for bound materials, and the water soluble sulphate content must not exceed 2.0 g/L for unbound materials;
- water absorption values must not exceed 10 %;
- flakiness index of coarse aggregate must not exceed 35;
- ten per cent fines value must not be less than 50 kN.

Nevertheless, in the case of material resulting from past production, additional precautions have to be taken since the variability of older stocks is considerably higher than for current production [Sherwood, 1995].

Air-cooled blastfurnace slag has a group of characteristics that make it particularly suitable for the replacement of naturally occurring aggregates and can be easily processed in order to produce a material complying with current specifications [Lee, 1974]:

- good particle shape and surface roughness, enhancing its frictional properties and the adhesion between particle and binder;
- low coefficient of thermal expansion;
- high fire resistance, of particular interest when used in concrete structures and in the manufacture of building blocks.

On the other hand, its porosity is high and, accordingly, high water absorption values are common for this material. However, this characteristic is a result of the vesicular nature and is not an indicator of poor durability. On the other hand, it results in higher binder contents being necessary for its stabilisation, in particular when bituminous binders are used, and additional quantities of water for concrete. Moreover, the high surface area, which is a direct effect of the surface roughness, reinforces the high binder demand.

Granulated blastfurnace slag, having a chemical composition similar to cement, possesses hydraulic properties when combined with a basic activator such as lime, cement, hydraulic fly ash, calcium or sodium hydroxide and other products formed of lime, gypsum and soda [Chi, 1992]. Its commercial value is too high to be used as an aggregate in road construction and, thus, it is used as an additive for cement or more generally as a binder. The latent hydraulic properties will depend on factors such as chemical composition, conditions of granulation and fines content. However, being a surface phenomenon the reactivity of the slag is essentially a function of the specific surface of the material after compaction, characterised by the coefficient α, which is defined as follows:

$$\alpha = S_B \ P_f \ 10^{-3} \qquad 2.1$$

where: S_B Blaine specific surface (in cm^2/g) of the natural fines fraction (< 80 μm);

P_f friability properties, that is, the percentage of fines (< 80 μm) generated in a standard crushing test, to simulate the generation of fines during compaction.

According to this coefficient granulated slags are classified into four groups [OECD, 1977]:

- class 1, $\alpha < 20$, slag not used in road construction;
- class 2, $20 < \alpha < 40$, slag most frequently used for gravel-slag and sand-slag mixtures;
- class 3, $40 < \alpha < 60$, used with materials difficult to handle;
- class 4, $\alpha > 60$, used only in exceptional circumstances.

The activity of the slags may still be improved by partial grinding (approximately 10 % of fines generated) or total grinding, and in this case ground granulated blastfurnace slag is obtained. During this process, an activator such as cement or lime may be introduced to produce a material with self-binding properties.

In summary, granulated blastfurnace slag with its slow and progressive setting and hardening has the particularity of combining flexibility during construction, good early-age mechanical stability (similar to a clean and angular sand) the ability to adapt during that period to permanent deformations that may occur in the subgrade and besides to be able to offer some capacity of self-healing. Reduced sensitivity to weather conditions and workability are also important factors reinforcing the utility of this alternative material.

Pelletised blastfurnace slag is composed of material under 12 - 13 mm. The fine fraction, 0 to 3 mm, possesses identical hydraulic properties to those of granulated blastfurnace slag and so it can be used for the same purposes in the cement manufacture and road construction. Its activity can be increased by grinding, as for granulated slag. The coarser fraction, being of expanded particles, possesses very low densities and is used as lightweight aggregate.

Foamed blastfurnace slag is formed of expanded particles with a cellular structure as a result of the process used for cooling the molten slag. Generally, it is crushed after hardening and presents the following properties: low densities, good thermal insulation and high fire resistance [OECD, 1977].

Regarding the variability of blastfurnace slags from different sources, it was shown that slags produced in a particular steelwork are highly consistent and that variability is comparable to that obtained in the exploitation of conventional aggregates [Lee, 1974]. The slag properties are primarily dependent upon the conditions of operation of the furnace. Hence, if the type of iron ore used as a raw material changes there will be significant variations in slag properties, larger than the one obtained when slags from different furnaces but the same material sources are compared. Typical properties of blastfurnace slags are presented in Table 2.8.

Despite the high values presented for water absorption, it has been experimentally demonstrated that blastfurnace slag is not frost susceptible. It can be used in base and sub-base layers and the performance of the frost heave test is superfluous [Croney & Jacobs, 1967].

Table 2.8 Typical properties of blastfurnace slag [Lee, 1974].

Property	Unit	Value
Specific Gravity		2.38 - 2.76
Bulk Density *	Mg / m^3	1.15 - 1.44
Water Absorption	%	1.5 - 5
Aggregate Crushing Value		25 - 39
Aggregate Impact Value		21 - 42
Ten per cent Fines Value	kN	70 - 160
Aggregate Abrasion Value		5 - 31
Polished Stone Value		50 - 63

Key: * 14 mm single size material.

2.9.5. Technical guidelines

When bound applications of blastfurnace slag are intended, binder contents for sub-base and base construction are as follows [BS 6543, 1985; Chi, 1992; Sherwood, 1995]:

- the British Standards propose 5 % of Portland cement as a first estimate for the determination of the optimum moisture content;
- the "*grave-laitier*" (gravel-slag) is one of the most common materials used in France for road construction. It is a stabilised natural aggregate using a binder composed by
    - 8 to 20 % of granulated blastfurnace slag,
    - 8 to 15 % of partially ground granulated blastfurnace slag and 1 % of lime or gypsum with soda as activator, or
    - 3.5 to 5 % of ground granulated blastfurnace slag;
- ground granulated blastfurnace slag (ggbs) is used in South Africa for the stabilisation of aggregates in a similar technique to the French "*grave-laitier*". The proportions of 80 % of ggbs and 20 % of lime are used;
- a combination of granulated blastfurnace slag (15 %) and phosphoric slag (85 %), which has a chemical composition similar to blastfurnace slag, is used in Holland.

Despite being widely proven in other countries, gravel-slag mixtures are not so common in the United Kingdom. This material has some special characteristics that make it highly attractive for application in pavements, and the extensive experience acquired in France, where it is widely used, provides strong evidence that severe problems do not occur. Some of the main characteristics of the gravel-slag combination for road applications are [OECD, 1977]:

- binder distributed more evenly in the mass of the aggregate and in higher quantities, giving the long term benefit of being able to re-cement cracks in the presence of moisture, when surfaces remain in contact;
- the long setting time of gravel-slag mixtures enables the short term storage of this material without major problems and a more flexible organisation of the construction works. In addition, it will allow the progressive increase of resilient moduli of layers where it was applied;
- the observance of a curing period in which construction equipment cannot traffic the compacted layer is not necessary, being especially attractive for strengthening works while traffic is maintained, because of the short term performance provided by aggregate interaction;
- heavy rains are not considered to be a major problem during construction works as typical gradings allow the excess of water to drain off and the compaction to continue;
- the setting phase is halted with very low temperatures, but continues once they rise;
- strength developments occur over long periods of time (one year and more), and are not influenced by deferrals in setting due to low temperatures;
- they possess good permanent deformation characteristics and thus rutting is not normally a problem;
- indicative values for some mechanical properties of this material after setting and hardening are: 6 - 10 MPa compressive strength, 0.6 - 1.5 MPa tensile strength and elastic moduli of 20 GPa approximately. These values correspond to the end of the strength development period and thus they should be considered as indicative maximum values.

Other techniques are used, specially in France, to extend the use of blastfurnace slags in road construction [OECD, 1977]: "*all-slag gravel-slag*", similar to gravel-slag described earlier but in which the aggregate used is blastfurnace slag; ready-mixed gravel-slag, in which the material is produced in a fixed plant where the slags are generated and then transported to the construction place; pre-crushed granulated blastfurnace slag in which, to enhance the reactivity of the slag, the material is lightly crushed in order to reduce its dimensions obtaining a material ranging from 0 to 2 mm, with 10 to 20 % of fines (< 90 μm).

2.10. Steel slags

2.10.1. Origin

Steel slag is a by-product of the steel manufacturing industry. It may be produced from pig iron, from steel scrap, or a combination of both (Figure 2.9). For the manufacturing of steel, it is necessary to remove the excess of carbon and silicon from the metallic iron by oxidation and the addition of other components [Lee, 1974]. Lime or dolomite used as a flux combine with the oxidised constituents forming steel slag. Chemically, steel slags are composed predominantly of lime, silica and iron oxide.

There are three principal types of furnace for producing steel: open hearth, basic oxygen and electric arc (Figure 2.9). In Great Britain, however, only the last two types are presently producing slags (BOS and EAF slags respectively).

2.10.2. Quantities and location

Existing steel slags in stockpiles amount to 11 million tonnes, according to a recent appraisal for the Department of the Environment [Whitbread, Marsay & Tunnel, 1991], of which 10 million tonnes are of BOS slags and 1 million tonne of EAF slags. The annual arisings of BOS slag are of 1.6 million tonnes, of which 0.2 million are utilised. The corresponding numbers for EAF slags are 0.4 million tonnes of production and 0.2 million of material used.

Steel slags are produced in the four British Steel integrated steelworks as mentioned for blastfurnace slag, and in the following areas where electric arc furnaces are in operation: Sheffield / Rotherham area, South Wales and Sheerness (Thames Estuary).

2.10.3. Current utilisation

Steel slags are used as aggregate, as ballast for railways, in pavement foundations and in wearing courses of bituminous materials, where any expansion that may happen (see next Section) will not be so damaging.

As a result of unstable and expansive behaviour, BOS (/LD) slags are "*absolutely unsuitable*" for Portland cement concrete or any other bound rigid mixture such as lean concrete or other pavement materials stabilised with hydraulic or pozzolanic binders [Verhasselt & Choquet, 1989; Gorlé, Verhasselt, Thijs & Kerkhof, 1992].

2.10.4. Properties

Presently, approximately 10 % of the annual production of BOS slags is used. The main problem inhibiting further use is related with their chemical composition, namely the free lime content (1 to 15 % and sometimes up to 20 %), magnesia, dicalcium silicate and residual iron oxides and iron that may hydrate or oxidise. The free lime and magnesia when in contact with water may cause significant expansion which can lead to the disintegration of the particles. The reaction of calcium oxide with water and its transformation into calcium hydroxide is, in general, fast and the expansion will be experienced in a few weeks, unless those reactions are entrapped within the particles and for this reason retarded in time. On the other hand, the reaction of magnesium oxide develops over a long period.

To reduce this expansion hazard, slags are stored outside, open to the environment, for periods of time of up to one year. This allows them to weather forcing the reactions of the components that may cause expansion or instability to develop during that period. Other techniques adopted in order to minimise expansion or instability problems are:

- the mixture of steel slag with conventional aggregates;
- its use in coated materials and the interposition of a sand layer, with 150-200 mm

minimum thickness, separating the steel slag from the overlaying material.

This last technique was found to effectively decrease or totally solve the problem of permanent damage in the pavement structure [Verhasselt & Choquet, 1989]. This sand layer has the following functions: to partially accommodate the expansion of the steel slag layer, to spread the stresses generated during that process and to fill some of the voids and cracks created in the slag layer.

Early research in Canada [Emery, 1975] presented experimental results suggesting that ageing, the increase in particle size (that is, limiting the amount of fines) and treatment with sulphuric acid would decrease the expansion of steel slags.

In relation to the influence of steel slag size on expansion, the results obtained in Canada are opposed to the outcome from case studies accomplished in Belgium, where comprehensive research on the behaviour of steel slags as a pavement material has been conducted [Choquet, 1984; Verhasselt & Choquet, 1989; Verhasselt, 1991; Gorlé, Verhasselt, Thijs & Kerkhof, 1992]. With respect to the laboratory study of BOS slags the approach consisted of:

- performance of disintegration tests, which involved the immersion of material samples in water, at 70 °C, with the purpose of accelerating the mechanism;
- swelling tests on samples with satisfactory performance in the disintegration test, based on the immersion of compacted specimens for 6 - 7 months, at 20 °C;
- quantification of the changes in particle density, before and after immersion in water at 60 - 70 °C, measured on the same sample of slag;
- visual examination of material and of compacted specimens after immersion in water.

As part of field studies performed, distress was observed in embankments, sub-bases and roadbases where unbound steel slags were used. The following aspects were pointed out as responsible for those situations: irregular distribution of free lime, weathering of materials not complete or inadequate and grading (> 30 mm) considered to be too coarse. Indeed, it is generally agreed that by using finer gradings (< 20-25 mm) it is possible to obtain a more efficient weathering of the slags, lower stresses will result from the expansion and cracking of particles, and the stresses in the contact points between particles will be reduced, resulting in a more uniform distribution of the stresses due to expansion.

EAF slag has lower free lime contents which are therefore responsible for its higher use in the construction industry. Again they are processed by weathering, crushing and screening.

Typical compositions and properties of steel slags are presented in Tables 2.9 and 2.10, respectively.

Table 2.9 Typical chemical composition of steel slag [Lee, 1974].

Component	**% by mass**
Lime (CaO)	40 - 50
Silica (SiO_2)	10 - 20
Alumina (Al_2O_3)	3 - 4
Magnesia (MgO)	2 - 9
Total sulphur (as S)	0.2 - 0.9
Total iron (FeO or Fe_2O_3)	12 - 20

Table 2.10 Typical properties of steel slag [Lee, 1974].

Property	**Unit**	**Value**
Specific Gravity		3.1 - 3.5
Bulk Density *	Mg / m^3	1.60 - 1.76
Water Absorption	%	0.2 - 2
Aggregate Crushing Value		12 - 25
Aggregate Impact Value		18 - 24
Aggregate Abrasion Value		3 - 4
Polished Stone Value		53 - 72

Key: * 14 mm single size material.

Steel slag particles possess good angularity. In comparison with blastfurnace slags, they have higher densities, lower water absorptions and higher particle strength, resistance to abrasion and skid resistance [OECD, 1977]. These properties make the slag more suitable for wearing courses. In addition, the incorporation of steel slags in a bituminous medium will hinder the access of water to the slag particles and, thus, will decrease the risk of expansion. The relatively high density of steel slags represents an inconvenience regarding transportation costs, which are quantified in volumetric terms.

2.10.5. Technical guidelines

Some recommendations for the use of steel slags exist in the bibliography. Once again, it is worth emphasising that the use of steel slags in rigid bound layers is often reported to be unacceptable, whether using either hydraulic or pozzolanic binders. This is of paramount importance when using BOS slags, with high free lime contents. In Belgium the following guidelines are suggested for unbound roadbases and sub-bases [Verhasselt & Choquet, 1989; Gorlé, Verhasselt, Thijs & Kerkhof, 1992]:

- free lime content should not be higher than 4.5 % when it is produced;
- gradings used should not exceed the maximum diameter of 20-25 mm;
- steel slags should be allowed to weather in the open air for a period of one year, stockpiled for this purpose in its final grading. Its acceptance should be conditional upon admissible behaviour in an accelerated swelling test;
- the incorporation in the pavement structure of a sand layer separating steel slags and overlaying materials is beneficial for reducing or eliminating possible damage of the surfacing layer. Its thickness should depend on the thickness of the steel slag layer subject to a minimum of 150 - 200 mm;
- an experimental study of the slags proposed for use should always be carried out and the volumetric stability of the final mixture should be verified by performing a swelling test.

Two types of tests are proposed to evaluate the volumetric stability of steel slags [CRR, 1993]:

- in the swelling test, a cylindrical specimen is prepared using the Standard Proctor compaction and it is immersed in water at 50 °C. Then, the linear expansion of this specimen is measured once a day, for 28 days, and should be lower than 1.5 %. This test is essentially applicable to particle size distributions up to 25 - 30 mm;
- alternatively, when the maximum dimension of the slag particles is higher than 30 mm, a disruption test may be performed. It is based on the quantification of the amount of material disintegrated after an immersion period of 14 days, in water at 60 °C. The maximum value should not exceed approximately 1 %.

2.11. Gypsum

2.11.1. Origin

By-product gypsum encompasses a wide range of products generated from different industrial processes. The most common for pavement applications are: flue gas desulphurisation gypsum, phosphogypsum and fluorogypsum. Phosphogypsum and fluorogypsum result from the production of phosphoric and hydrofluoric acids, respectively.

Flue gas desulphurisation (FGD) gypsum is defined by Eurogypsum, representing the European gypsum industry, as:

> *"Gypsum from flue gas desulphurisation is a moist, finely divided, crystalline, high purity calcium sulphate dihydrate ($CaSO_4\,2H_2O$). It is specifically produced in a flue gas desulphurisation process incorporating limestone scrubbing, a refining process involving oxidation followed by gypsum separation, washing and dewatering."*

FGD gypsum, also called desulphogypsum or synthetic gypsum, is generated in power stations burning lignite or sulphur-contaminated coals. When these are burnt, the existing sulphur is transformed into sulphur oxides, principally sulphur dioxide, which may contribute to the formation of acid rain when released to the atmosphere. As a result of environmental legislation limiting the emissions of sulphur dioxide, gas scrubber systems were introduced in power stations to remove the sulphur from the flue gases. The process consists of forcing the reaction of those gases with a neutralising agent, usually a limestone or lime slurry, inside the scrubber systems. Dependent on the technology employed, the by-product generated can be FGD gypsum.

2.11.2. Quantities and location

The information regarding quantities of FGD gypsum is scarce but according to National

Power [Westaway, 1996] the production may be estimated to be over a million tonnes per annum. In addition, more gypsum is currently imported to the UK from mainland Europe.

Being a by-product of the power generation industry, this material is essentially available in the same locations as the coal burning power stations and thus at the same locations as power station ashes.

2.11.3. Current utilisation

Cement-stabilised gypsum has been used in the USA and the existing field experience has demonstrated that when correctly designed it may be used in layers as high in the pavement structure as roadbases. As an example, it has been used in combination with cement and hydraulic fly ash (class C) for this purpose.

Flue gas desulphurisation gypsum is also used for the production of a wide variety of building plasters, for which they can totally replace natural gypsum. In combination with anhydrite it is used as a setting retarder in the cement industry, for the production of wallboards, un-fired brick and masonry components, and in agriculture to improve soil characteristics [Winkler, Gruber, Hammerschmid & Rentz, 1989; Saylak, Scullion & Golden, 1993].

By-product gypsum is used in France to expedite strength development of gravel-slag mixtures and to improve their long term compressive strength, immediate frost heave resistance and workability.

2.11.4. Properties

In the United States of America, research has been conducted on the use of phosphogypsum in roadbase construction. Some of the factors that showed significant effect on strength were [Saylak, Taha & Little, 1988]:

- acidity of the material as characterised by its pH value. Material recently produced, with lower pH values (2.5 approximately), developed lower strength than stockpiled material with higher pH values (5.5 approximately);
- correct grading and shape of the particles have an important effect on compaction, leading to higher densities and more stable materials.

Comparatively, flue gas desulphurisation gypsum presents a suitable grading and pH value immediately after production. The main difference between recently produced and aged flue gas desulphurisation gypsum consists in its content of $CaSO_3$, as shown in Table 2.11, and its transformation into $CaSO_4$ which is relatively fast. Therefore, there is not any special advantage in ageing this material and, on the contrary, it causes significant reductions in strength [Saylak, Taha & Little, 1988].

Table 2.11 Example of physical and chemical properties of flue gas desulphurisation gypsum [Saylak, Taha & Little, 1988].

Property	New Gypsum	Aged Gypsum
Moisture Content (%)	13.5	13.4
pH Value	7.7	6.6
Specific Gravity	2.33	2.64
Fineness (< 45 μm, %)	98	95
Fineness Modulus	2.77	2.84
Components (%)		
$CaCO_3$	1.4	3.0
$CaSO_3$	7.0	0.5
$CaSO_4$	92	96

It has been demonstrated that by-product gypsum may be used at sub-base and roadbase levels when stabilised with Portland cement or pulverised fuel ash, either individually or combined [Taha, 1989; Saylak, Scullion & Golden, 1993]. In France, by-product gypsum is particularly indicated for the stabilisation / activation of fly ash [SETRA & LCPC, 1980].

2.11.5. Technical guidelines

A recommended procedure for stabilised dihydrate gypsum mixtures and several guidelines developed in the USA are presented below [Saylak, Taha & Little, 1988; Saylak, Scullion & Golden, 1993].

Gypsum selection

For preliminary gypsum selection, only a material with pH values above 5.0 should be considered, and the particle size distribution should be without agglomerations of particles retained on the 4.75 mm sieve. For gypsum with pH values under 5.0, the acidity may be reduced by chemical neutralisation or simply by washing. In this case it is advisable to replace the fines lost during the washing process with original material.

Gypsum is highly sensitive to moisture, and whether water was used to reduce acidity or control dust, it should be taken into account when considering the moisture content for compaction. Excessive moisture may weaken the strength development process and / or cause serious problems of expansive behaviour. An adequate sealing of the stabilised layer is vital and in this case bituminous emulsions should not be used since they tend to supply the mixture with extra water.

The temperature used to determine the moisture content of the material should not exceed 40 °C, to prevent the removal of chemically combined water and its transformation into hemihydrate.

Binder selection

Cement with high percentages of tricalcium aluminate (C_3A) may be responsible for expansive behaviour and crackings in cement bound gypsum. Indeed, it may react with sulphate generating ettringite crystals and cause expansion [Deussner, Neinz & Ludwig, 1985]. However, ettringite cannot be formed without excessive water or tricalcium aluminate and so the use of cements with C_3A contents under 7 % is recommended or the use of sulphate resistant cements. Moreover, if high sulphate resistant cements are employed, which contains percentages of C_3A under 3 %, the possibility of expansive behaviour is negligible [Chang & Mantell, 1990].

Pulverised fuel ash is also suitable for the stabilisation of gypsum combining pozzolanic activity and fines correction, which may be particularly important for washed gypsum. However, the use of fly ashes with high calcium oxide contents, above 17 % (class C hydraulic fly ashes) is recommended.

Strength

Further strength enhancements are obtained by creating a granular matrix in the gypsum stabilised mixtures. This may be attained by adding other materials to the composition of the mixture such as a graded sand or furnace bottom ash. As a first estimate, a cement content of 7 % and gypsum / bottom ash ratio of 1:1 may be used since they were used satisfactorily as roadbases in pavement test sections.

2.12. Cement kiln dust

2.12.1. Origin

Cement kiln dust (ckd), also known as cement flue ash or cement precipitator ash, is a by-product of the cement manufacture industry. Similar to what happens with pulverised fuel ash, the gases generated during the production of Portland cement carry very fine and light particles from the kilns. This material is extracted from the gases, to prevent air pollution using electrostatic precipitators, cyclone separators, scrubbing towers and bag filters.

The proportion of dust produced may be considerable, from 5 to 15 % of the cement-clinker production [Sherwood, Tubey & Roe, 1977]. However, part of this material is re-used in the kilns and only the material having high alkali content, that is not suitable for re-use, is available as a secondary binder.

2.12.2. Quantities and location

Cement kiln dust is available where it is generated in cement works. Early estimates of 1976 regarding the quantities of this material were reported to be an annual production of 0.4 million tonnes and the existence of 3 million tonnes in stockpiles [Sherwood, Tubey & Roe, 1977].

Recent information supplied by the British Cement Association and Blue Circle Industries indicate the following volumes produced annually in the stated cement works [Coole, 1995]:

- Weardale Works (County Durham) .... 25 000 tonnes
- Westbury Works (Wiltshire) .... 50 000 tonnes
- Northfleet Works (Kent) .... 75 000 tonnes

- Masons Works (Suffolk) .... 50 000 tonnes

2.12.3. Current utilisation

Stockpiled cement kiln dust has been used in the past in embankments and in bituminous mixes as a filler (replacing natural fines and other conventional fillers such as limestone dust, Portland cement and hydrated lime) [Sherwood, Tubey & Roe, 1977; OECD, 1977]. Other possible areas of use are reported to be in roadbase layers, mixed with power station fly ash for the stabilisation of aggregates, in pollution control and in agriculture [Concrete Construction, 1979].

2.12.4. Properties

The cement kiln dust available corresponds to the fraction of dust with high alkali contents. Yet these alkali salts are highly soluble and for this reason the material allowed to weather will have reduced alkali and sulphate contents when compared with recently produced kiln dust [Sherwood, Tubey & Roe, 1977]. On the other hand, its cementitious properties after being stockpiled for a long period will be much diminished due to the hydration of the free lime, dramatically limiting its interest as a secondary binder. The typical chemical composition of cement kiln dust in comparison with Portland cement is presented in Table 2.12.

When cement kiln dust is used in concrete as a partial replacement for Portland cement, the following effects are observed [Ravindrajah, 1982]: retarded setting, increased water demand for the same consistency and reduced strength. These reductions, however, will not be significant for percentages of cement replacement up to 15 %.

Some properties reported for application as a fill material in embankments are [Sherwood, Tubey & Roe, 1977]: good workability independent of the weather, high CBR values (80 %), and very low field dry densities, between 0.97 Mg/m^3 and 1.18

Mg/m^3, when compacted using moisture contents ranging from 37 to 49 %.

Table 2.12 Typical chemical composition of cement kiln dust and Portland cement [Sherwood, Tubey & Roe, 1977].

Components	Cement Kiln Dust (%)	Portland Cement (%)
SiO_2	15.3	22.0
Al_2O_3	2.1	5.5
Fe_2O_3	1.7	3.0
CaO	43.0	64.1
MgO	0.9	1.4
SO_3	6.2	2.1
K_2O	6.0	0.5
Na_2O	0.7	0.2

An experimental study was carried out to ascertain the adequacy of cement kiln dust as a stabilising agent [Sherwood, Tubey & Roe, 1977]. Several samples were taken from different cement works and then used to stabilise a well-graded sand. The results obtained in terms of unconfined compressive strength were compared to the ones obtained with control specimens, in which the same sand was stabilised with cement. None of the ckd specimens reached the minimum compressive strength of 2.8 MPa, after 7 days curing, as stipulated by the then DoT Specification for Road and Bridge Works [DoT, 1976], and the best result achieved with cement kiln dust was just 20 % of that realised with Portland cement. As a result of this study cement kiln dust was considered to be unsuitable as a binder to replace cement.

Despite the negative results when used individually as a binder, it has been used with pulverised fuel ash for the stabilisation of pavement aggregates for roadbases [Miller, Bensch & Colony, 1980]. In the post-construction observation and testing, it was found that deflections in the pavement decreased with the curing time, no cracking or surface damage was observed (with the exception of a localised area) and there was evidence of self-healing properties of the mixture. Similar mixtures studied in laboratory were found to have identical behaviour to that of corresponding lime - fly ash - aggregate mixtures [Collins & Emery, 1983]. In some cases they developed higher initial strength.

2.12.5. Technical guidelines

The results of these studies seem to indicate that cement kiln dust should be combined with another binder to achieve better results. Optimum binder proportions reported in the bibliography are [Collins & Emery, 1983]:

- 66.6 % of ckd to 33.3 % of fly ash; or
- 50 % of lime kiln dust to 50 % of fly ash.

Those percentages compare with 20 to 25 % of lime and 75 to 80 % of fly ash in lime - fly ash - aggregate mixtures. This confirms that more cement kiln dust than lime will be required for similar stabilisation of the same type of aggregate.

2.13. Environmental considerations

There is some concern related with the application of secondary materials in road construction, in particular those associated with ground water pollution due to leaching. It is argued that it is preferable to concentrate those materials susceptible to leaching into landfills, in a known place, where they can be controlled, rather than to disperse them in the construction of roads. However, some materials do not represent any danger for the environment because they do not contain toxic substances, or if they do, either the concentration of compounds is low (under acceptable limits) or the toxic compounds do not leach out of the aggregate particle. For other materials, if appropriate measures are taken their safe application is also possible, but special caution is needed mainly when the road is neighbouring rivers or ground water sources of public water supply [Baldwin, Addis, Clark & Rosevear, 1995].

Regulatory leaching tests have been developed in several countries using single batch extraction techniques. However, the leachate concentrations obtained in these tests are only representative of the particular conditions in which the tests are carried out, not reflecting the actual environmental impact of their use in road construction [van der Sloot, 1990]. Performance assessments are necessary for environmental aspects in the same manner as for the technical ones. Methodologies have been described involving the evaluation of bound and unbound wastes, using realistic gradings, degrees of compaction and physical and hydrological testing conditions simulating more closely the ones in the field [Bridges, 1995; Nunes, Bridges & Dawson, 1996].

A summary of the experiences on pollution potential of the secondary materials mentioned in this project is presented below.

The use of slate waste and china clay sand as wastes resulting from quarrying naturally occurring materials do not constitute any threat to the environment. The environmental damage occurs because of the extraction of the primary slate and china clay respectively.

Colliery spoils have been used in several countries, such as France, Germany and the

United Kingdom, and in spite of having high concentrations of sulphates and/or pyrite, they have not caused any environmental problem [Chi, 1992].

As far as power station ashes are concerned, and due to the nature of these materials, they give rise to more environmental concerns when large scale applications in road construction are intended, in particular related with the possibility of leaching of heavy metals and sulphates. Their leaching behaviour has been studied in The Netherlands [Bolt, 1987] where road trial sections were built and observed, each of them incorporating an 'environmental section'. These were specially designed to allow the water percolating through the pavement structure to be collected and analysed to measure the concentrations of several chemical compounds. It was concluded that during the first two years the leaching behaviour of cement bound roadbases (of fly ash or sand / fly ash) was equivalent to the one of sand stabilised with cement and, in the long term, leaching decreased progressively. In spite of these results, according to environmental legislation, bound applications of fly ash are permitted provided that the material is above ground water level and below a sealing surface. Regarding unbound applications of fly ash they must be completely isolated from the environment and positioned above ground water level. Bottom ash, on the other hand, has leaching characteristics similar to other materials currently used in The Netherlands, such as granulated blastfurnace slag.

In Belgium, leaching tests on several fly ashes have confirmed that the risk of ground water pollution is very small and that it is further reduced when fly ash is used in stabilised materials [Gorlé, Verhasselt, Thijs & Kerkhof, 1992]. No environmental protection measures are required when used in bound materials in proportions up to 20 %. Nevertheless, for other applications fly ash must be placed above ground water table and covered with an impermeable material, such as a wearing course layer, a geomembrane or plastic soil.

The presence of sulphur in blastfurnace slags is of some concern. From the environmental point of view, it may be responsible for generating leachates that could contaminate the ground water table. Besides this, it may attack concrete products or metallic items and it may cause swelling when in contact with water. However, the presence of sulphur is limited to low contents by present specifications [BS 1047, 1983;

BS 3797, 1990; BS 6699, 1992]. Consequently, if adequate drainage is provided leached substances will not reach dangerous concentrations [Lee, 1974]. To diminish the risk of attack to concrete products, a sulphate resistant cement could be used.

In respect of steel slags, the risk of pollution is limited, as it is also for blastfurnace slags. It is, however, necessary to restrict the contents of free lime to values under approximately 10 % [Choquet, 1984].

By-product gypsum is reported to cause negligible environmental pollution with the exception of areas where it may become in contact with sources of drinking water supply [Saylak, Scullion & Golden, 1993]. Leachate analyses were performed on samples and individual components of the mixtures used in a test section and compared with US Environmental Protection Agency standards. The maximum limits for drinking water were largely exceeded as far as sulphate concentration is concerned. On the other hand, the concentration of heavy metals were under the maximum permitted values.

Another type of pollution associated with waste materials re-use and recycle is the pollution of the atmosphere. This can be in the form of gases from burning colliery spoils, as a result of spontaneous combustion, or in the form of dust. Dust control problems occur during storage, transport and placement when using pulverised fuel ash and other materials with very fine and light particles such as cement, lime and cement kiln dust. Special measures can be taken in order to minimise them. For storing purposes, the use of silos eliminates this problem or, if this facility is not available, fly ash for example can be conditioned to minimise dust. For transport and placement, the material should be covered in lorries and the use of special enclosed lorries with bottom unloading facilities is preferred; the addition of controlled amounts of water, which should be taken into account when considering the moisture content for compaction, is also possible.

Overall, it is important to study the leaching potential of secondary materials and methods to limit harmful effects on the environment resulting from their application in roads. Clearly, when the raw materials are not suitable due to environmental issues, their stabilisation will go some way to reducing problems, producing better materials from both technical and environmental aspects.

2.14. Economic considerations

2.14.1. Industry structure

The aggregates industry is analysed using the Five Forces Model proposed by Porter [1980] with special focus on the secondary materials sub-industry. This model represents a powerful tool to understand the environment of a company within a particular industry, enabling it to identify and focus on the most important issues facing the industry. It considers five forces controlling the company as described below.

1. Entry Barriers

In the aggregates industry in general the entry (and exit) barriers are high due to the rising levels of initial investment required. In particular for the secondary materials sub-industry the entry is further blockaded by the current industry's *status quo*. In fact, despite the benefits of using secondary materials their application remains very limited in the UK, in practice restraining entry. The main factors responsible for this situation were mentioned in Chapter 1.

2. Substitute Products

Available substitutes for conventional (or primary) aggregates are, essentially, secondary materials (there are some artificial aggregates that may also be considered as substitutes but their quantitative importance is very small and in some cases they are manufactured with secondary materials). The barriers mentioned before limit the potential entry of firms to the industry bringing substitute products. Transport costs constitute an important factor and, in some areas, it may be responsible for the availability of only one type of aggregate even though it may be available from several suppliers.

3. Power of Suppliers

The power of suppliers is perceived as being low. Most of the companies in this industry are simultaneously suppliers and contractors. Indeed, they directly extract the rock in the quarries, process the material and apply it in road construction.

4. Power of Buyers

With the exception of private roads having a small weight in the overall market value, the bodies responsible for the construction and maintenance of the road network, representing the buyers in this industry, are the Department of Transport for trunk roads and motorways and Local Authorities for principal, classified and unclassified roads.

Ultimately, the buyers or clients are the central and local government engineers responsible for the projects. Although contractors may present alternative solutions for particular projects, it is to the engineer to decide whether or not the alternative solution should be accepted and most engineers are conservative as far as innovation and the use of new materials are concerned. Overall cost is generally of paramount importance but performance aspects need to be assured.

In brief, the power of buyers in this industry is high allowing them to exert pressures on prices and on the type of solutions adopted.

5. Competitive Rivalry

The aggregate industry is dominated by a small number of large companies that are simultaneously suppliers and contractors for road construction / maintenance and responsible for most of the total industry output. They sell homogeneous products and are characterised by having a small number of contracts of relatively high value. As a result, competition in quality is difficult and competition in price is risky, except for companies really having a cost advantage. Little incentive is felt for competition and for conducting research in order to develop new and better products resulting in a generally slow rate of technical development.

In general, the competition in the secondary materials sub-industry is restricted to a small number of major companies linked to the industrial processes generating by-products. Again, transport costs play a major role and as a result they hinder competition in some localities.

The Sixth Force

The analysis of the secondary materials sub-industry will not be complete without considering the effect of political and environmental forces representing the most important components of the sixth force [Dobson & Starkey, 1994]. These forces are of major importance and will probably have a determinant influence in the way the equilibrium between forces shaping the industry will be achieved in the future.

In summary, the aggregate industry is fairly static and dominated by a small number of large companies selling undifferentiated products. Market share is successfully gained only by companies having a strong competitive advantage in terms of global costs of production and transport to the areas where the aggregates are used. The current equilibrium is favouring producers of conventional or primary aggregates but once cost and performance aspects are assured it may be changed to the advantage of the secondary materials sub-industry. Political and environmental inputs may also force in that direction.

2.14.2. Strategic options

The main issues to create and sustain competitive advantage that should be addressed by a company within or willing to enter the secondary materials sub-industry are:

- costs of production and transport;
- materials' performance;
- political and environmental forces.

As to costs of production and transport secondary materials have a potential advantage only in terms of production, since the materials are already financed by other industrial processes which generate them and when they are not sold the firms will incur extra costs to stock or dispose of them. Geographic factors and transportation costs which may account for 50 % of the total sale price are here determinant and the creation of infrastructures for efficient methods of transport seems to be essential. For road transport, the economic distance tends to be relatively small, around 20 to 50 miles, depending on factors such as type of material and processing costs. Rail transport increases that distance considerably but it is by sea that those distances are highest. As an example, the Glensanda coastal superquarry, in Scotland, covers the South East region. Theoretically, with this new concept of superquarries as long as the rock deposits are in the proximity of the coast they can be put into the world market economically.

Economies of scale may be exploited by developing large and highly capital intensive corporations encompassing business units in the areas of extraction, transport and, possibly, of ready-mix concrete. The latter would represent a related diversification in relation to the present typical configuration of most of the companies in this field.

With regards to materials' performance, some advances have been made and it is hoped that this research will contribute towards the drafting of performance-based specifications that will reduce present entry barriers and will encourage product development and innovation. The Department of Transport Specification for Highway Works [DoT, 1993] already covers some areas in which the requirements are essentially specified in terms of end-product and where secondary materials may find fields of application. There is some evidence from this work that secondary materials are of adequate technical properties for use in road construction and when this is not the case they may be treated in order to improve their properties. It is important, however, that firms pursue rigorous quality control to assure more homogenous and higher quality material.

Regarding political and environmental forces as part of the industry's environment, it is possible to attempt to shape it in the industry's favour. The means to achieve this will be by exerting pressure towards environmental awareness and lobbying in the political arena to change the present equilibrium in the industry. There is already some willingness from the Department of the Environment in restricting the extraction of conventional aggregates in some regions [FT, 1994].

In conclusion, firms in the secondary materials industry may need to pursue a *cost leadership strategy* in the short run in order to create cost advantage in relation to primary aggregates and reduce present barriers. In the long run, if superior products are developed and their performance is proven, then some *differentiation* may be possible and buyers may be willing to pay a premium for materials, resulting in better roads with higher life and lower maintenance costs.

CHAPTER 3

PRESENT APPROACH TO THE USE OF UNBOUND SECONDARY MATERIALS

3.1. Introduction

Pavements are multi-layer systems formed of a number of layers of compacted unbound aggregates and/or bound materials, with increasing level of requirement, and consequently of material quality, from bottom to top. The three main components of the pavement are foundation, roadbase and surfacing. There are three types of pavement structures generally considered (Figure 3.1):

- *flexible*, in which the surfacing, and often the roadbase, are of bituminous bound materials the remaining layers being of unbound material;
- *rigid*, composed of a top layer of high quality pavement concrete forming roadbase and surfacing and built on a bound sub-base. The concrete layer may be
    - jointed unreinforced (URC);
    - jointed reinforced (JRC);
    - continuously reinforced (CRCP);
- composite pavements, a combination of previous types and sub-divided into
    - *flexible composite* formed by a bituminous surfacing and upper roadbase (if used) built on cement bound roadbase or lower roadbase (cement bound materials types 3 or 4 - CBM3 or CBM4); and

- *rigid composite*, comprising bituminous surfacing built on roadbases of continuously reinforced concrete.

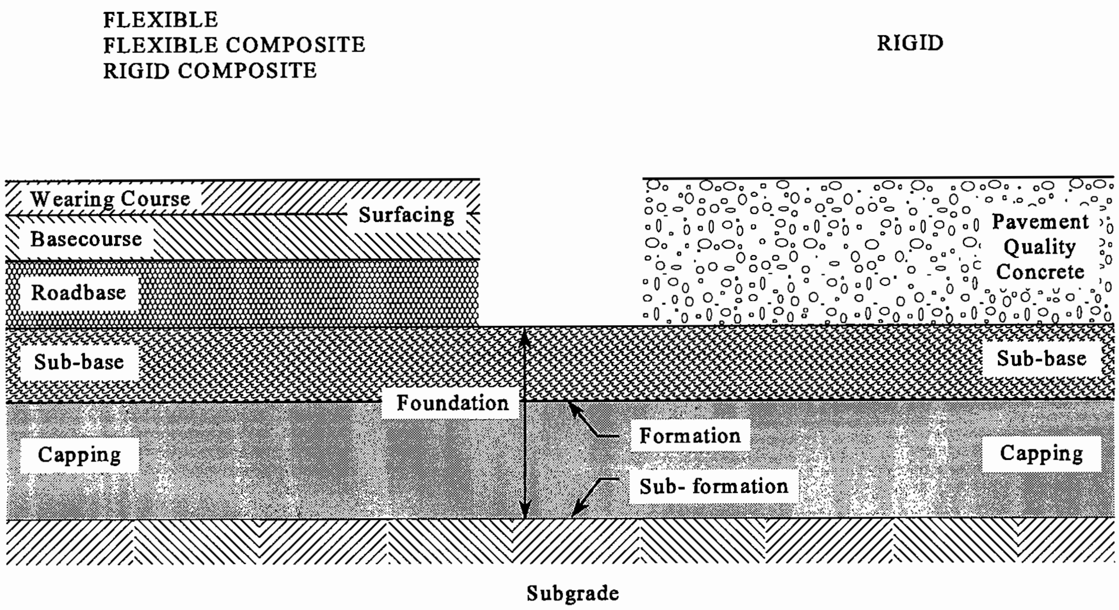

Figure 3.1 Typical pavement structures.

In order to evaluate the performance of secondary materials for use in pavement foundations their relevant properties have to be determined so that their suitability for pavement construction can be assessed. The relevant properties to be determined should be those which best characterise the likely in-service performance and these depend on the functions of the layer for which the material is envisaged. On the other hand, the functions of the various materials depend on the level where they are used in the pavement structure. For unbound applications of secondary materials the only possible layers are capping and sub-base, since at roadbase level the use of unbound materials is not generally permitted in the UK. The functions of aggregates for sub-bases and capping layers can be summarised as follows in approximate order of significance:

- to enable the placement and compaction of road materials in higher layers. Placing demands appropriate tolerances which can be lost if the foundation 'bounces'. Compaction needs a firm base to work against;

- to provide a working platform for construction traffic preventing excessive rutting;
- to provide a drainage layer for the pavement structure, helping to drain the subgrade and reducing the detrimental effects of water on pavement performance;
- to contribute to the structural performance of the pavement, by spreading the load, due to traffic, which comes down through the completed pavement structure. By so doing they will prevent overstressing of the underlying soil;
- to operate as an insulating layer against frost. The importance of this function depends on the climate. In the United Kingdom the DoT Specification [DoT, 1993] requires that the material must be non-frost susceptible if used within the depth of frost penetration, currently specified as 450 mm from the final surface or 350 mm if the mean annual frost index is less than 50.

To effectively fulfil their role, the materials to be applied in those layers need to have a set of basic properties relevant for the functions mentioned above. These properties are:

- load spreading ability - the important property of the aggregate which controls load spreading is the resilient modulus;
- resistance to permanent deformation to prevent rutting essentially under construction traffic (since stress levels in sub-bases and capping layers under the completed pavement are considerably lower);
- good permeability, to dissipate any water which has percolated through upper layers or entered during construction, to allow a rapid dissipation of the pore pressures generated during traffic loading on saturated soils and reduce the effects of frost action;
- appropriate grading - the effects on performance of compaction under moisture content with small deviations in relation to the optimum value will be less significant in well-graded materials;
- non-frost susceptibility if the layer concerned is to be applied in the conditions mentioned above.

3.2. Present assessment technique

Presently the assessment of unbound materials for road pavements is done according to the Specification for Highway Works. Figure 3.2 and Table 3.1 contain a summary of the requirements for unbound capping materials, classes 6F1 and 6F2, and for unbound granular sub-base materials, Type 1 and Type 2 [DoT, 1993 & 1993 i]. For comparative purposes typical values of a conventional aggregate (a crushed limestone) are included in Table 3.1. Sub-base material Type 1 is a good quality granular material that can be used in all roads (except when rigid pavement structures are constructed, where unbound sub-bases are not permitted in the UK roads), having higher requirements than sub-base material Type 2. The workability of the latter can be poor in wet weather (especially when a fine grading is used) and its use in flexible and flexible composite pavements is restricted to design traffic loadings of less than 5 msa at opening [DoT, 1994].

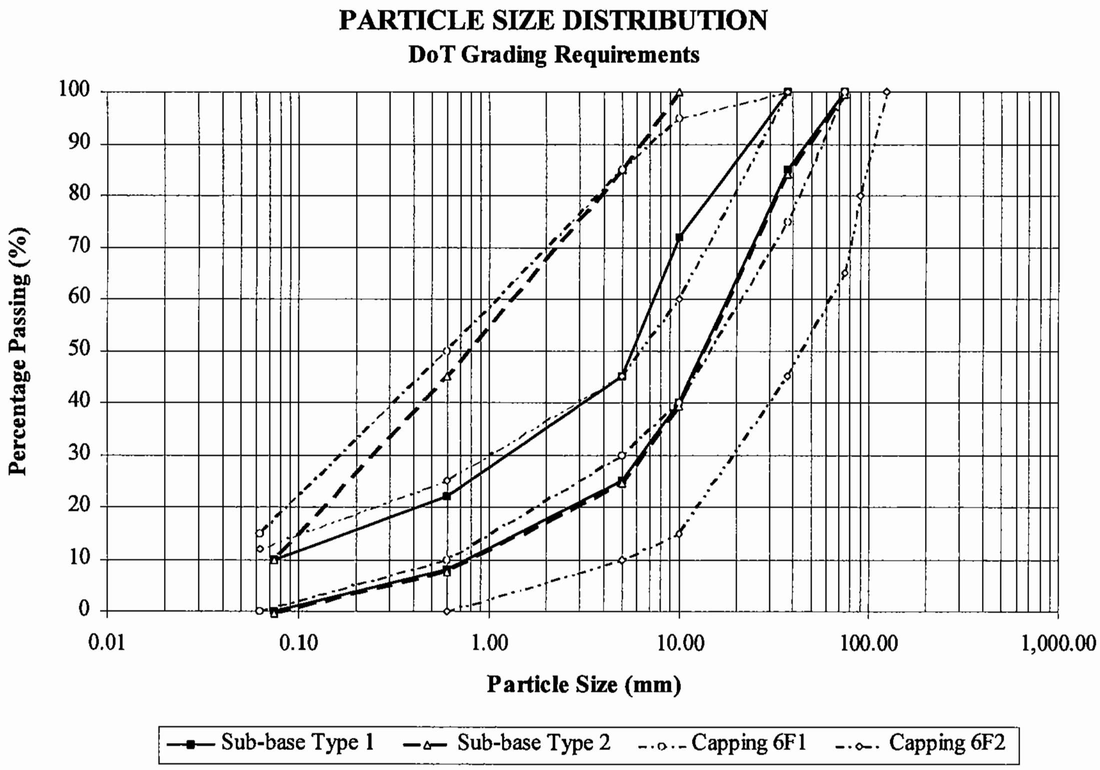

Figure 3.2 Summary of grading requirements for unbound sub-base and capping layers.

Table 3.1 Summary of DoT requirements for unbound capping and sub-base layers.

Property	**DoT Limit**				**Crushed Limestone (indicative values)**
	Capping (1)		**Sub-base** (2)		
	6F1	**6F2**	**Type 1**	**Type 2**	
Flakiness Index	NS		NS		----
Elongation Index	NS		NS		----
Plasticity of < 425 μm fraction	NS		Non Plastic	$I_p < 6$	Non Plastic
Water Absorption (%)	NS		≤ 2 (3)		1.2
Particle Density (Mg/m^3)	NS		NS		2.60
Aggregate Crushing Value ACV (%)	NS		NS		22
Aggregate Impact Value AIV (%)	NS		NS		23
Ten Per Cent Fines Value TFV_{dry} (kN)	NS		NS		> 120
Ten Per Cent Fines Value TFV_{soaked} (kN)	≥ 30	≥ 50 (4)	≥ 50		> 100
Magnesium Sulphate Soundness, S (%)	NS		≥ 65		97
Sulphate Content (g/litre)	≤ 1.9 (5) or ≤ 0.25 (6)		≤ 1.9 (5)		----
Optimum Moisture Content OMC (%)	NS		NS		7.0
Maximum Dry Density MDD (Mg/m^3)	NS		NS		2.30
California Bearing Ratio C B R (%)	NS		NS	≥ 30,T>2msa ≥ 20,T<2msa	> 70
Permeability (m/s)	NS		NS		2×10^{-4} (8)
Frost Heave (mm)	≤ 15 (7)		≤ 15 (7)		6

Key: ---- = Not measured NS = Not Specified

(1) ***Permitted materials***: Class 6F1 - any material, or combination of materials, other than unburnt colliery spoil, argillaceous rock and chalk; Class 6F2 - any material, or combination of materials, other than unburnt colliery spoil and argillaceous rock.

(2) ***Permitted materials***: Type 1 - crushed rock, crushed slag, crushed concrete or well burnt non-plastic shale; Type 2 - natural sands, gravels, crushed rock, crushed slag, crushed concrete or well burnt non-plastic shale.

(3) In routine testing, if water absorption value of the coarse material is > 2 % the soundness test should be carried out [DoT, 1992]. This requirement was removed in the 1993 amendments [DoT, 1993].

(4) Minimum recommended value [DoT, 1991].

(5) For materials other than slag, when placed within 500 mm of cement-bound materials, concrete pavements, concrete structures or concrete products.

(6) When deposited within 500 mm of metallic items forming part of the Permanent Work.

(7) If used within 450 mm of the designed final surface or 350 mm when the mean annual frost index is < 50.

(8) Constant head vertical permeameter (100 mm diameter).

3.3. Testing programme

A testing programme was set up with the intention to perform a complete characterisation of all materials studied, covering all tests required in the present specification for unbound capping and sub-base materials. The aim of this phase was to ascertain the feasibility of using the materials in unbound layers assessing them against current specifications. The following tests were included in this part of the work and were performed in accordance with the appropriate British Standard which are indicated in brackets:

- particle size distribution [BS 812 : Part 103 : 1985]
- descriptive tests
    - flakiness index [BS 812 : Part 105.1 : 1989]
    - elongation index [BS 812 : Part 105.2 : 1990]
- plasticity [BS 1377 : Part 2 : 1990]
    - liquid limit, plastic limit, plasticity index
- water absorption [BS 812 : Part 2 : 1975]
- particle density [BS 812 : Part 2 : 1975]
- particle strength
    - Aggregate Crushing Value, ACV [BS 812 : Part 110 : 1990]
    - Aggregate Impact Value, AIV [BS 812 : Part 112 : 1990]
    - Ten per cent Fines Value, TFV [BS 812 : Part 111 : 1990]
- durability test
    - magnesium sulphate soundness [BS 812 : Part 121 : 1989]
- sulphate content (water-soluble) [BS 1377 : Part 3 : 1990]
- compactibility test [BS 5835 : Part 1 : 1980]
- California bearing ratio, CBR [BS 1377 : Part 4 : 1990]
- permeability [BS 1377 : Part 5 : 1990]
- frost heave [BS 812 : Part 124 : 1989]

Particle size distribution was determined in dry conditions except for minestone and china clay sand for which both dry and wet conditions were used.

Some of the tests listed above are not commonly used neither in continental Europe nor in the United States, namely the particle strength tests and frost heave test. A full description of these may be found in the corresponding standards. In addition, although the magnesium sulphate soundness test is also used in other countries the results are reported differently, which may cause some confusion when comparing results. Hence, the results presented here are in accordance with BS 812 [1989] where the soundness value is calculated as the mass of material retained at the end of the test divided by the initial mass of the test specimen (expressed in percentage). In some other countries it is determined as the mass of material lost during the test as a percentage of the initial mass of the test specimen [NP 1378, 1976; ASTM C88, 1990]. An approximate relationship between those results may be obtained by subtracting one value from 100 % to obtain the other. Note, however, that some test conditions may be slightly different.

The selected secondary aggregates (Chapter 2) were evaluated using the tests listed above. Slags (blastfurnace and steel) and gypsum were not considered for study at this stage in an unbound form. With regard to the slags they were excluded for practical reasons with the aim of reducing the number of materials and not based on technical grounds since they have been used in pavement layers without stabilisation. On the other hand, gypsum is not adequate for unbound applications, which is confirmed by durability results obtained in this project on stabilised gypsum. The sources of the materials studied were as follows:

- *minestone (MI)*
  supplied by British Coal Corporation, Gascoigne Wood Mine, Selby, Yorkshire;
- *china clay sand (CC)*
  supplied by Camas Aggregates, Lee Moor Quarry, Lee Moor, Devon;
- *slate waste (SW)*
  supplied by Alfred McAlpine, Penrhyn Quarry, Bethesda, Gwynedd;
- *power station ashes (FA & BA)* - pulverised fuel ash and furnace bottom ash;
  supplied by Powergen, Ratcliffe-on-Soar Power Station, Ratcliffe, Nottingham;
- *granite Type 1 (T1)* (to act as a control material)
  supplied by Mountsorrel Quarry, Leicestershire.

3.4. Results on unbound materials

The results obtained in the conventional tests for aggregate materials used in capping and sub-base layers are presented in Figures 3.3 to 3.6 and Tables 3.2 and 3.3.

The grading curve of the conventional crushed granite is also presented in Figure 3.7. This material was not exhaustively tested against the specification limits because it is produced as a Type 1 sub-base material meeting all DoT requirements. Granite will be further studied for the assessment of its mechanical properties using more fundamental tests (Chapter 6) to act as a control material regarding the performance of secondary materials. For this reason, only the properties necessary for the compaction of specimens in the laboratory were determined. The following results were obtained:

- water absorption 0.6 %
- particle density 2.71 Mg/m^3
- optimum moisture content 6.5 %
- maximum dry density 2.19 Mg/m^3.

Although blastfurnace slag was not studied at this stage in an unbound form, for the reasons explained before, its grading curve is presented in Figure 3.8. This material was assessed after stabilisation and the results presented and discussed in Chapter 8.

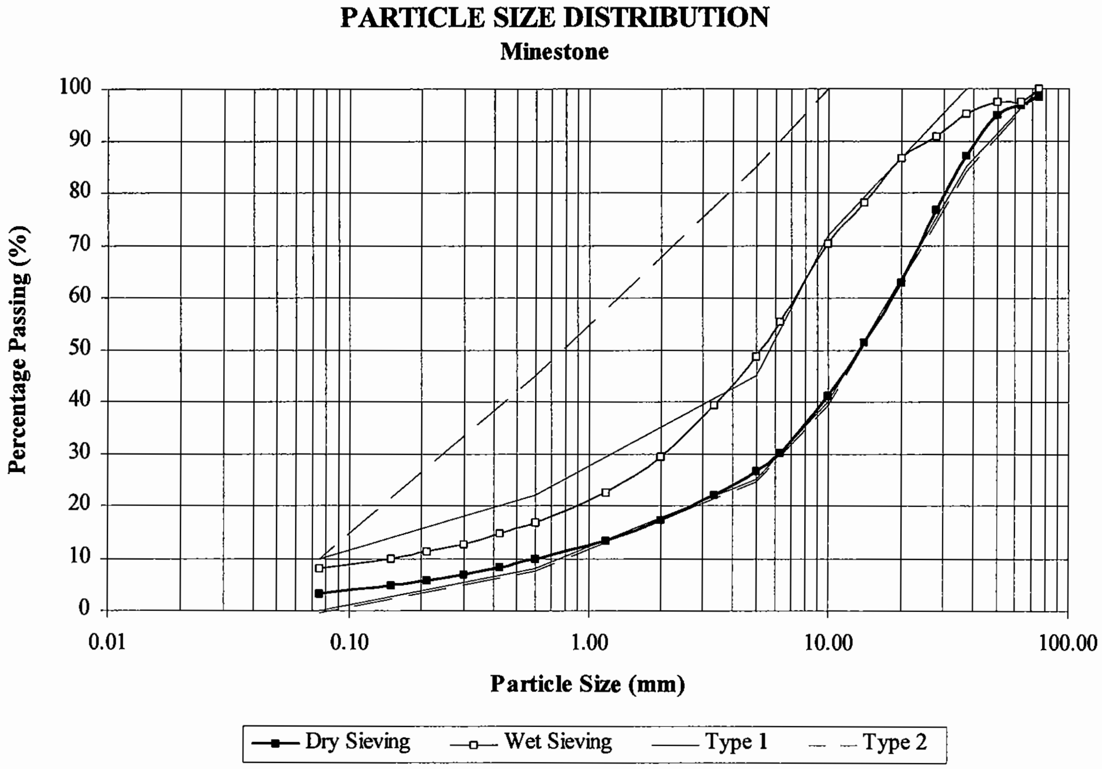

Figure 3.3 Particle size distribution (minestone).

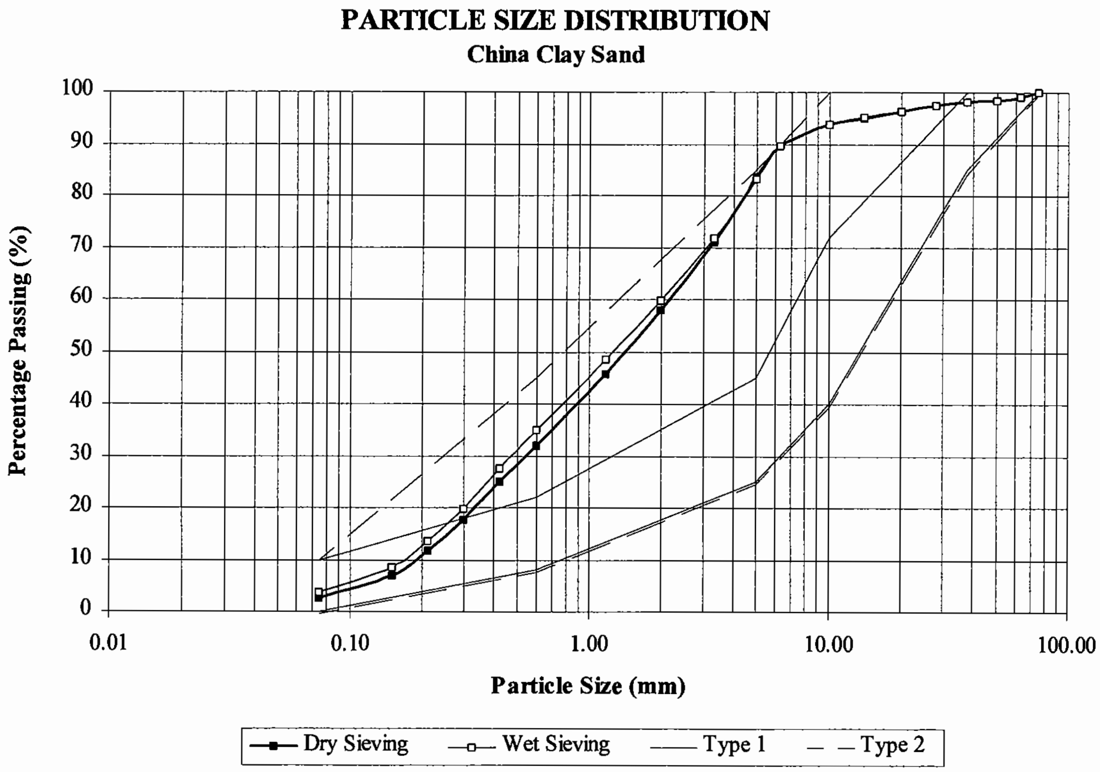

Figure 3.4 Particle size distribution (china clay sand).

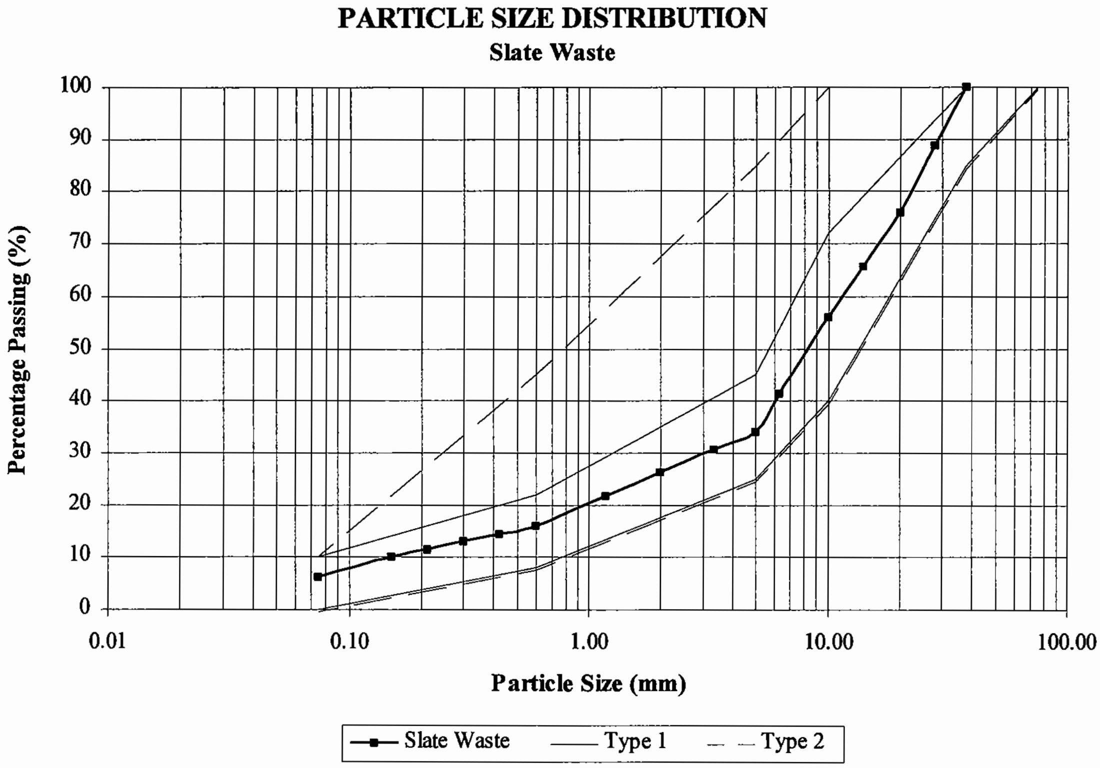

Figure 3.5 Particle size distribution (slate waste).

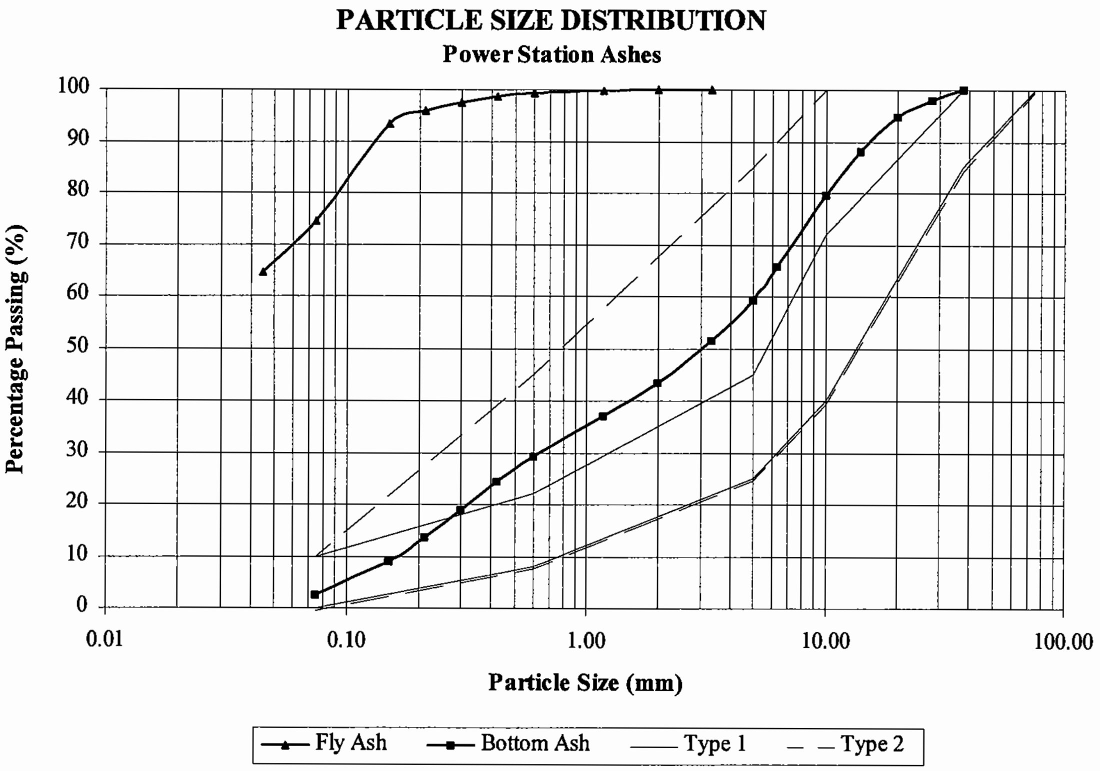

Figure 3.6 Particle size distribution (power station ashes).

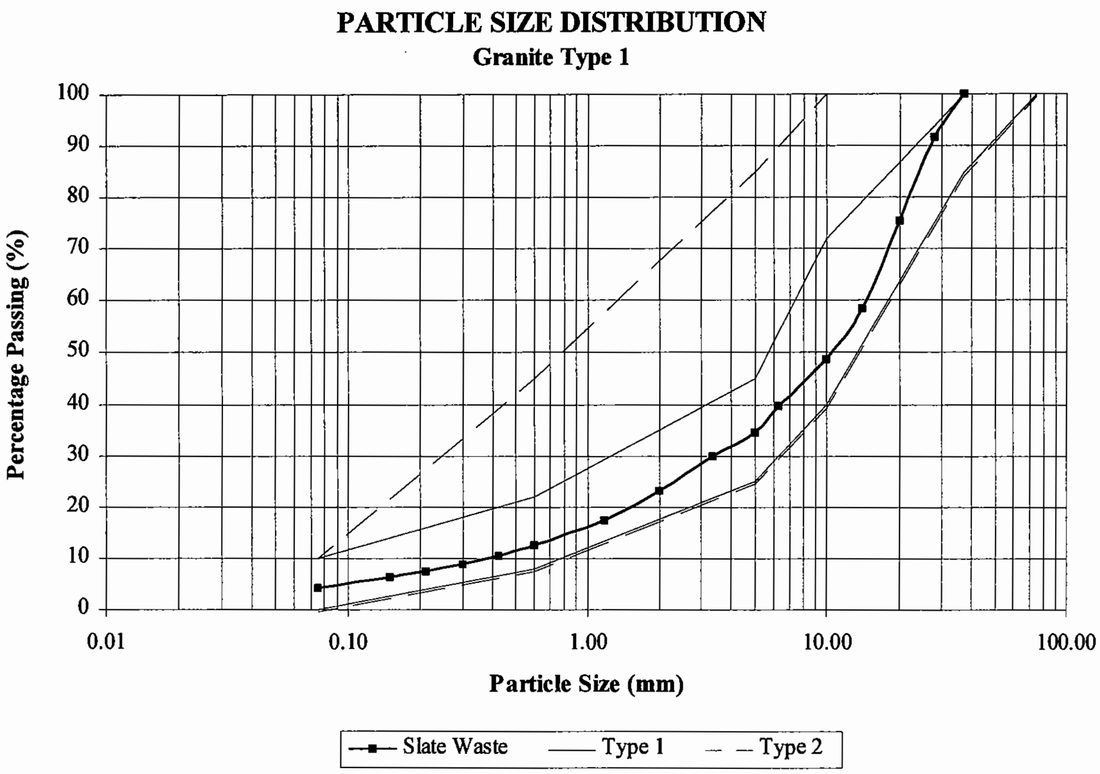

Figure 3.7 Particle size distribution (granite Type 1).

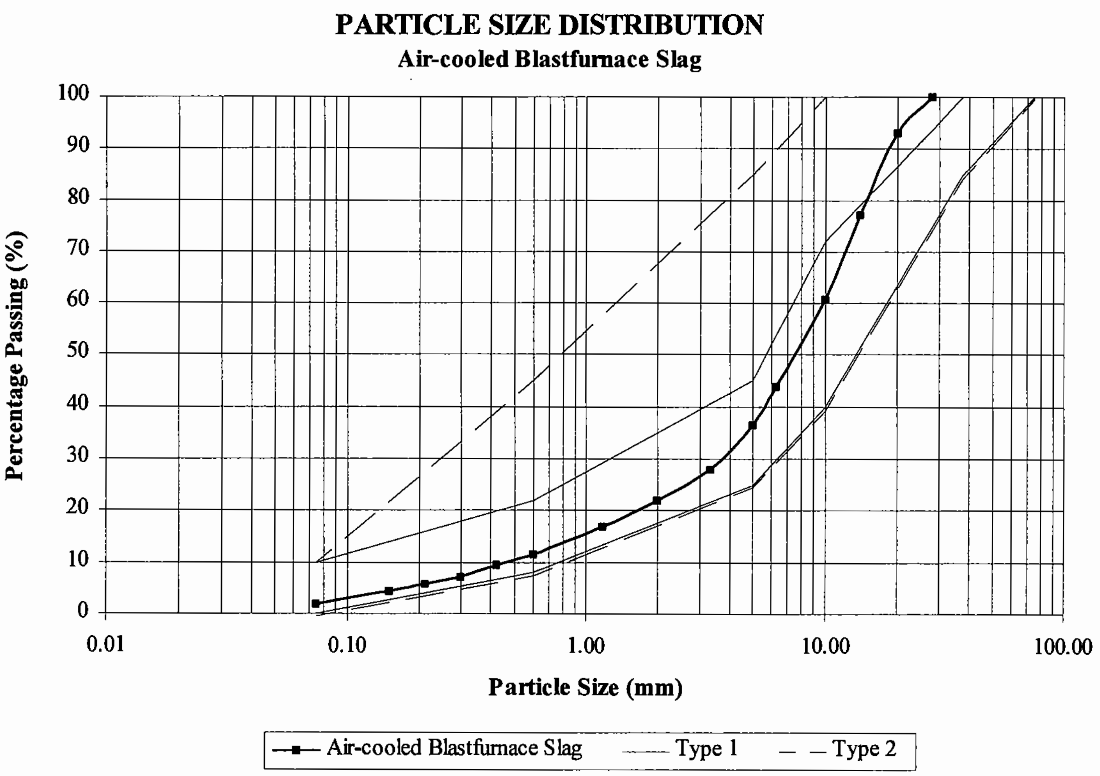

Figure 3.8 Particle size distribution (blastfurnace slag).

Table 3.2 Summary of conventional test results.

Property	Minestone	China Clay Sand	Slate Waste	Furnace Bottom Ash
Flakiness Index	41	12	89	----
Elongation Index	60	20	66	----
Plasticity of < 425 μm fraction	$I_p = 15$	Non Plastic	Non Plastic	Non Plastic
Water Absorption (%)	5.7	1.9	0.5	10.4
Particle Density ρ_s (Mg/m^3)	2.43	2.53	2.80	1.90
Particle Strength Tests: Size fraction	5.00-6.30mm	5.00-6.30 mm	5.00-6.30 mm	5.00-6.30 mm
Aggregate Crushing Value, ACV (%)	33 (1)	27	22	51 (1)
Aggregate Impact Value, AIV (%)	24	22	20	43
Ten Per Cent Fines Value, TFV_{dry} (kN)	20	40	50	10
Ten Per Cent Fines Value, TFV_{soaked} (kN)	0 - 5	35	30	7
Magnesium Sulphate Soundness, S (%)	4	81 (2) 70 (3)	96	77
Sulphate Content (g/litre)	0.33	0.03	0.01	0.17
Optimum Moisture Content, OMC (%)	7.2	10.0	6.6	24.0
Maximum Dry Density MDD (Mg/m^3)	2.12	1.93	2.23	1.22
California Bearing Ratio (%) CBR	5	87	73	69
MC	7.2	10.0	6.6	24.0
DD	2.12	1.93	2.23	1.22
Permeability (m/s)	----	----	2.4×10^{-3} (4)	2.4×10^{-2} (5)
Frost Heave (mm)	----	8.2	8.0	0.2

Key: ---- = Not measured MC = Moisture Content DD = Dry Density

(1) The test is not suitable for testing aggregates with an ACV higher than 30 [DoT, 1993]

(2) Size fraction: 3.35 - 5.00 mm

(3) Size fraction: 5.00 - 6.30 mm

(4) Constant head vertical permeameter (100 mm diameter) [Goulden, 1992]

(5) Large horizontal permeameter (specimen size, 1000 x 300 x 300 mm) [Dawson, Brown & Thom, 1989]

Table 3.3 Summary of conventional test results for the pulverised fuel ash.

Property		Pulverised Fuel Ash
Typical Chemical Composition (1) (%)	S_iO_2	48.5
	Al_2O_3	25.5
	Fe_2O_3	12.1
	CaO	2.5
	MgO	1.5
	Na_2O	1.0
	K_2O	3.0
	TiO_2	1.1
	SO_3	1.0
	Cl^-	0.06
Moisture Content, MC (%)		3.7 (as supplied)
Loss On Ignition (%)		4.1
Fineness: Retained # 45 µm		35.3
Plasticity of < 425 µm fraction		$I_p = 17$
Relative Density (Mg/m^3)		2.11
Sulphate Content (g/litre)		0.83
Optimum Moisture Content, OMC (%)		20.0
Maximum Dry Density, MDD (Mg/m^3)		1.40
California Bearing Ratio (%)	CBR	45
	MC	20.0
	DD	1.40
Frost Heave (mm)		15.3

Key: MC = Moisture Content DD = Dry Density

(1) Data supplied by Powergen, Ratcliffe-on-Soar Power Station, Ratcliffe, Nottingham.

3.5. Discussion of results

3.5.1. Minestone

According to existing specifications the minestone tested would have been rejected for sub-base material due to:

- its high plasticity index value;
- its high water absorption and the remarkably low magnesium sulphate ($MgSO_4$) soundness value, indicating low durability when used in pavement layers;
- the very low Ten per cent Fines Value obtained, especially when soaked, showing low particle strength according to this test. The Aggregate Crushing Value was too high (above 30) confirming the weakness of the particles; and
- the very low CBR value.

Values obtained for TFV and $MgSO_4$ soundness tests were so low that it would have been rejected even as capping material. The sulphate content is also above the limit specified for materials placed within 500 mm of metallic items. Apart from the poor performance in the tests conducted, minestone is specifically excluded for unbound applications at capping and sub-base levels.

TFV in soaked conditions was in the range between 0 and 5 kN. As a matter of fact, the soaking process alone, without load application, was responsible for the generation of 4 % fines and more important deeply weakened the particles. TFV was not determined precisely due to the difficulty in using the compression machine for maximum forces less than 10 kN, and the low relevance of the precision of any result that could have been obtained in that range.

Frost susceptibility was not determined due to the very poor performance demonstrated in the other tests, effectively excluding the material from unbound pavement applications. The main perceived problem with this material is its low particle stability when in contact with water, resulting in deep particle degradation. This is also illustrated by the difference in grading curves when determined in dry and wet conditions (Figure 3.3). Consequently, this minestone cannot be recommended for unbound use in pavements principally because it is likely to come in contact with water. Minestone is very variable and these findings may not apply to other minestones. Further testing on this material should be conducted bearing in mind the aim of improving its particle and bulk stability by treatment with appropriate stabilisers. This is considered in Chapter 4.

3.5.2. China clay sand

As far as china clay sand is concerned it fails to meet the following requirements:

- sub-base Type 1 range of grading. Nevertheless, it still meets sub-base Type 2 requirements;
- TFV requirements for sub-base materials, although still complying with DoT limits for capping materials.

Water absorption and magnesium sulphate soundness values are just within the allowed limits. In the $MgSO_4$ test, the soundness value for the fraction between 5.00 and 6.30 mm (70 %) was smaller than that value for the fraction 3.35 - 5.00 mm (81 %). Taking into account that the latter represents a larger percentage of the whole material, it can be considered as a more representative value. Sherwood [1995] analysing critically this durability requirement now imposed by the DoT Specification pointed out that the value specified is a result of a research work conducted by the Transport Research Laboratory (TRL) specifically applied to bitumen macadam roadbase. Despite this, the durability criterion was adopted for unbound sub-base materials. In the 1993 amendments of the DoT Specification the limit was reduced from 75 % to 65 %.

This material has higher potential for unbound applications than perceived by the direct comparison with the specification requirements. No problems are anticipated either chemical or of durability owing to the low sulphate content, the reasonable soundness value and the non-plasticity of the fines. In addition, this material is non-frost susceptible and produced high CBR values which, despite the low credibility of the test, seem to indicate a high bearing capacity.

3.5.3. Slate waste

Slate waste is processed by the supplier in order to meet all DoT requirements for granular sub-base Type 1. This was confirmed by the results obtained and represents an exception within the set of materials studied. The soaked Ten per cent Fines Value shown in Table 3.2 does not meet the DoT requirements. Nonetheless, this test was conducted using a non-standard size fraction (5.0 - 6.3 mm instead of the standard 10.0 - 14.0 mm) because the standard fraction was not available in adequate quantities in some of the secondary materials studied and for comparative purposes it was decided to perform the test using the same size fraction for all materials. A change in the size fraction tested affects the result but it is not known in what way due to the flaky nature of the slate. Data available on slate waste of the same type and source, and obtained with the standard fraction, show adequate particle strength of the material [Goulden, 1992]:

- TFV in dry conditions 160 kN;
- TFV in soaked conditions 110 kN.

Good performance was also demonstrated in other tests. The water absorption was very low, 0.5 %, suggesting a high durability. This was also confirmed by the results obtained in the soundness test, in which very high percentages were obtained. The slate showed an exceptional performance in this test, better than some conventional aggregates. In terms of particle crushing resistance, the values obtained from the ten per cent fines test on the standard size fraction are fairly good. The slate has a high particle density, even higher than for most primary aggregates. The fraction passing 425 μm sieve was found to be non-plastic and the sulphate content very low. Inevitably, a very high flakiness index was measured, 89, and a high elongation index of 66. The permeability, measured vertically in the falling head permeameter, is higher than for most conventional aggregates. The horizontal permeability of the slate waste may be expected to be much higher than its vertical permeability. Good CBR values were obtained.

For slate waste, acceptance on the grounds of existing specifications for conventional materials seems evident. However, since present knowledge of slate waste results essentially from conventional laboratory testing and very limited data are available from field performance, it is considered important to extend previous testing with performance tests such as repeated load triaxial tests.

3.5.4. Pulverised fuel ash

Based on the results obtained, pulverised fuel ash in an unbound form does not seem adequate for the applications envisaged here in capping and sub-base layers. This is essentially due to the following factors:

- its particle size distribution, principally formed by silt and clay-sized particles, is totally outside the limits specified for unbound sub-bases;
- its high plasticity. When mixed with water, fly ash appearance is very similar to a moistened clay. The plasticity index value is too high for unbound use as a sub-base material;
- possible loss of bearing capacity when saturated;
- frost susceptibility.

Fly ash is not strictly an aggregate although it may be used as a pavement material and as a major component of stabilised mixtures. Despite the reasonable CBR value obtained, its potential for unbound applications in pavement foundations is limited. However, it has been used successfully as an embankment fill.

3.5.5. Furnace bottom ash

A similar situation happens with furnace bottom ash in terms of conventional specification for sub-base and capping materials. The reasons for exclusion in this case are:

- failure to fit into sub-base Type 1 range of grading, but still within Type 2 limits;
- low TFV, under acceptable limits for both sub-base and capping layers (although there is difficulty carrying out the test according to the specification because of insufficient material of the correct size). This is also confirmed by the ACV obtained of 51, although this test is not suitable for weak aggregates.

Note that water absorption requirements were not included in the 1993 amendments [DoT, 1993] and so, despite the high value obtained, this would not be a reason for exclusion.

Bottom ash possesses some properties indicating a higher potential for pavement construction than perceived from the present assessment techniques and the resulting exclusion. For example, the material possesses a slightly more uniform grading curve than conventional Type 1 aggregates giving rise to high permeabilities, non-frost susceptibility and likely to result in a high toleration to an increase in (non-plastic) fines resulting from the material's degradation. It also has a remarkably low particle density, important for construction on soft soils, and reasonable CBR values.

Another particular aspect of bottom ash is that despite having high water absorption values it did not give low soundness values. Thus, it seems that these two values are not inter-related for bottom ash and that good durability may be expected. This is because of the vesicular nature of the material and it is contrary to what generally happens with conventional aggregates which are assumed by the the DoT Specification requiring water absorption values under 2 % [DoT, 1992]. It reinforces the need to assess and control materials' behaviour using performance-related tests. The determination of index properties, indirectly related with performance is not totally reliable because exceptions may occur, especially when using secondary materials.

3.5.6. General discussion

The exclusion of china clay sand and furnace bottom ash based on the results of particle size distribution and TFV test does not seem to be reasonable without further testing to determine their mechanical properties by using performance tests such as the repeated load triaxial test, which simulates the repeated loading effect due to traffic and allows the evaluation of those properties using correct stress levels. Some form of treatment may also be beneficial and may strengthen these materials to a sufficient degree to meet the DoT requirements for cement bound materials.

Grading envelopes are generally based on Fuller's law [Fuller, 1905] with some adaptations resulting from past experience. Materials failing to meet grading requirements are not necessarily materials to reject. In some cases, the use of finer than standard gradings may have beneficial effects in decreasing inter-particle contact stresses and, hence, material degradation. However, their use introduces another problem due to the lack of previous experience of testing non-standard size fractions. This results in an increased difficulty in comparing the results with those obtained from standard gradings and in comparing results of different materials for which the relationship between standard and non-standard fractions will vary.

With regard to particle strength, it has been demonstrated [Dawson & Nunes, 1993] that furnace bottom ash and slate waste are characterised as having low resistance to particle crushing according to the particle strength tests currently used in the UK for this purpose (Ten per cent Fines Value and/or Aggregate Crushing Value) particularly the bottom ash. Nevertheless, both materials showed satisfactory performance during compaction, with neither significant degradation nor mechanical instability. Dawson [1989] discussed in depth the possible reasons for this apparent contradictory behaviour. In brief, those tests seem to be unnecessarily intense and are performed using unrepresentative fractions of the aggregate full grading, not being able to predict the performance of the material *in situ* as an unbound granular layer, neither under compaction nor in service.

In terms of the test methods used here some changes were necessary in relation to normal procedures described for conventional materials. First of all, particle strength tests should have been performed on a standard fraction of the aggregate between 10 and 14 mm. It was considered adequate to follow the standard method when the percentage of material within that range was at least 15 % approximately. In practice, it was not possible to obtain this size fraction for some of the materials tested due to the small percentage available in the bulk material and, because of this, all materials were tested using non-standard particle fractions to enable a direct comparison of results obtained. Smaller particles usually produce higher TFV [Dawson, 1989] and so, if the materials were tested with the standard particle fraction it may have resulted in lower TFV values. However, this is not always true and in the case of slate waste the use of a fraction with smaller particles resulted in a lower Ten per cent Fines Value, probably due to the flakiness of the particles.

The Specification for Highway Works requires minimum values of CBR test for granular sub-base Type 2. The specimen should be tested unsoaked, with surcharge discs, and the specimen compacted at the optimum moisture content and a density corresponding to a uniform air voids content of 5 % [DoT, 1993]. From the compactibility tests, it was perceived that most of the materials had an air voids content higher than 5 % when at the optimum point. When trying to compact specimens above the optimum levels, using a vibrating hammer, a great deal of effort was necessary in order to reach the point relating to 5 % air voids. This did not seem to be comparable to the situation in the field. Consequently, further specimens were compacted using the conventional effort at optimum moisture content and maximum dry density values, since they are always values of reference for the compaction during construction.

The tests indicated focus essentially on the properties of the aggregates and particles rather than on those of the compacted material. Furthermore, the determination of the necessary mechanical properties to be used in mechanistic methods of pavement design is not possible using these tests. They may be indirectly estimated from these results, for example from CBR tests, using empirical equations, but the reliability of the estimated values will be reduced. This aspect is further discussed at the end of next chapter.

3.6. Summary

- The present methodology for the assessment of unbound aggregates for pavement foundations was presented according to the Specification for Highway Works. The requirements of this specification are based on past experience obtained essentially with conventional aggregates and do not necessarily apply directly to secondary materials.

- The testing programme set up for evaluating the secondary materials studied against current specifications was described and the results obtained were presented.

- Slate waste is processed at source in order to meet the requirement for Type 1 sub-base material. All the other materials failed to meet one or more requirements for capping and sub-base layers. Minestone and pulverised fuel ash are believed not to be suitable for unbound applications in pavements (unless very special precautions are taken such as their total isolation in relation to the environment). China clay sand and furnace bottom ash possess properties demonstrating some potential for unbound applications that should be verified through the performance of more fundamental tests such as the repeated load triaxial test.

- All materials should be considered for stabilisation (again with the exception of slate waste)
    - to improve their performance as modified 'unbound material' and
    - to verify their properties against the requirements for cement bound materials.

- Several aspects discussed stress the desirability of developing performance-based specifications for the evaluation of pavement materials. This would enable a wider use of alternative materials.

CHAPTER 4

LIGHT STABILISATION OF SECONDARY MATERIALS

4.1. Introduction

Apart from the potential for the replacement of conventional aggregate in pavements, either in unbound or bound layers, some of the secondary materials included in this research have cementitious / pozzolanic properties. Hence, they can be used for soil and aggregate stabilisation as secondary binders, alone or combined with conventional binders. Pulverised fuel ash, blastfurnace slag (granulated and pelletised), gypsum and cement kiln dust, alone or in the presence of lime or cement, are examples of secondary binders. Indicative values of light, moderate and heavy stabilisation will be given later in this Chapter (Table 4.4).

These binding properties are an important and attractive aspect of these materials which can help to improve their competitiveness against primary aggregates by reducing layer thicknesses or enhancing mechanical properties, general performance and lives of the pavement layers. Improvements in the quality of sub-base materials may also lead to the reduction in layer thicknesses of overlaying materials.

In the United Kingdom, rigid pavements are not used routinely for the construction of motorways or heavily trafficked roads [DoT, 1992 i], and consequently composite pavements will probably become more important in the future. Furthermore, the growth of heavy traffic and the desirability of using secondary materials are likely to contribute to this tendency.

In composite structures, the use of bound materials under the surfacing, which have a good tensile strength and high resilient modulus, gives rise to some particular aspects of performance of this type of pavement in relation to conventional flexible pavements [Quaresma, 1990]:

- better load spreading to underlying materials;
- development of tensile stresses in the base of the concrete or cement bound layer, which may cause the appearance of cracking, essentially longitudinal in the initial phase;
- development of transverse cracking, due to thermal shrinkage;
- good quality support to the bituminous layer, decreasing substantially the tensile stresses in its base, except when in the proximity of fissures in the roadbase;
- where these fissures exist, there will be concentration of tensile stresses in the base of the bituminous layer, originating new cracks in this layer, which may propagate to the surface of the pavement. This behaviour is known as reflective cracking.

The use of bound sub-bases, generally accepted to result in better performances of rigid and rigid composite pavement structures and currently compulsory for this type of pavement [DoT, 1994], should enhance the quality of flexible and flexible composite types of structure, emphasising the advantages already mentioned. Indeed, when the sub-base is constructed with unbound granular material, under a bound roadbase, its contribution to the overall performance is limited. On the other hand, if the sub-base is made of bound materials it has an important effect in reducing tensile stresses in the roadbase, which can be translated into cost savings, by reducing thicknesses or cement contents, or by increasing pavement lives.

4.2. Hydraulic road binders

4.2.1. Conventional binders

The primary binders that are most commonly used for the stabilisation of soils and granular materials are cement and lime which are either used as a conventional binder or as an activator of a secondary binder. Figure 4.1 summarises guidelines presented in Terrel *et al.* [1979] for binder selection, according to the grading and plasticity of the material for stabilisation (in that figure Ip is the plasticity index).

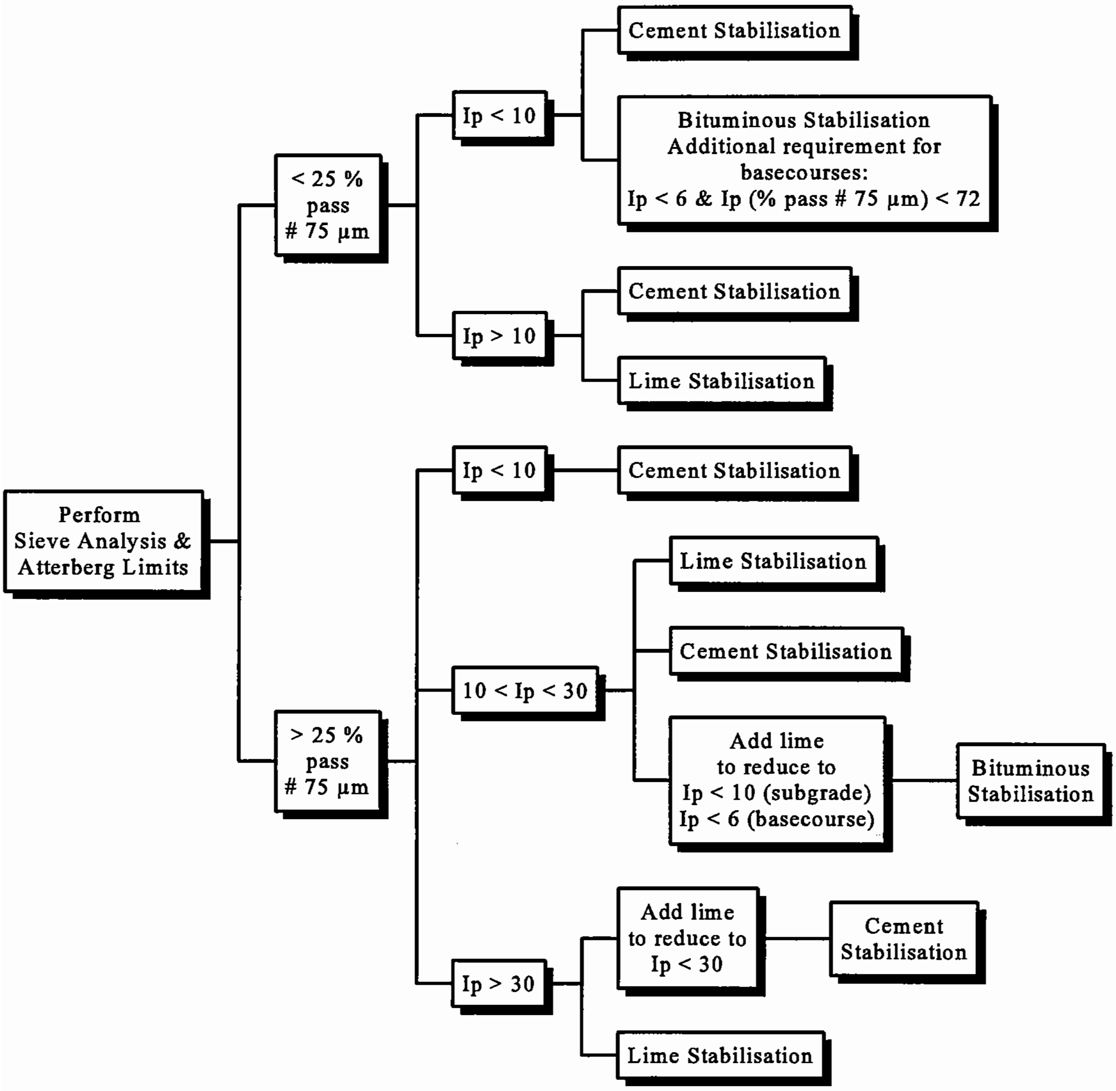

Figure 4.1 Guidelines for the selection of stabiliser.

4.2.1.1. Cement

Cement is manufactured by calcining, at about 1450 °C, a mixture of calcarium (70 to 80 % approximately), clay and eventually other materials rich in silica, alumina or iron [Coutinho, 1988]. From this operation, conducted in rotary kilns, results Portland clinker which is then cooled and subsequently ground. At this stage additives are introduced to produce various types of cement such as Portland cement, blastfurnace cement, Portland-pozzolana cement and pozzolana cement. The typical composition of the cements used in this research is presented in Table 4.1.

Table 4.1 Typical chemical composition of cements used.

Component	**Cement Composition (%)** (1)	
	Hope Works	**Weardale** (2)
$Si\ O_2$	20.4	20.8
Insoluble Residue	0.4	0.38
$Al_2\ O_3$	5.8	4.0
$Fe_2\ O_3$	2.2	4.9
$Ca\ O$	65.0	63.9
$Mg\ O$	1.1	2.13
$S\ O_3$	3.2	2.0
$Na_2\ O$	0.2	0.11
$K_2\ O$	0.6	0.57
Loss on Ignition	1.1	1.1
Fluorine	0.15	0.09
Free Lime	1.7	1.6
Tricalcium Aluminate	11.7	2.3

(1) Data supplied by Blue Circle Industries.
(2) Cement with low tricalcium aluminate content.

Cement possesses hydraulic properties and so it can react with water forming cementitious compounds. These hydration reactions give origin to hydrated calcium silicates and aluminates which are responsible for the development of strength in the stabilised material. Another product of hydration is calcium hydroxide, which may further react with the clay minerals of the soil to produce further cementitious compounds. These secondary reactions are of the lime-clay type and will be discussed in the next section.

Due to its hydraulic characteristics, in principle, cement can be used for the stabilisation of any type of soil or granular material. Although the range of materials that can be stabilised with cement is wider than with lime, it is believed that the best results are obtained with well-graded granular materials with enough fines to create a floating matrix. Its use with clays may be more problematic than the use of lime due to the difficulties in mixing them.

4.2.1.2. Lime

For the production of quicklime, high quality limestone is calcined at high temperatures (1150 or 1300 °C depending on the size of the stones), generating calcium oxide, or quicklime, and carbon dioxide. Hydrated or slaked lime results from the hydration of quicklime when in contact with controlled amounts of water. The typical composition of the lime used in this research is presented in Table 4.2.

In general terms, the most important characteristics of lime used for stabilisation, to assure an effective treatment, are [PIARC, 1991]:

quicklime

- free CaO ≥ 80 %;
- fineness of grading - 0 / 2 mm with passing # 80 μm ≥30 %;
- rate of hydration - 1 : 4 mixture of lime and water to reach a temperature of at least 60 °C after 10 minutes;

hydrated lime

- free CaO ≥ 60 %;
- fineness of grading - passing # 80 μm ≥85 %.

Table 4.2 Typical chemical composition of lime used.

Component		**Lime Composition (%)** [1]
Calcium Oxide	CaO	95.63
Calcium Carbonate	$CaCO_3$	2.23
Magnesium Oxide	MgO	0.47
Calcium Sulphate	$CaSO_4$	0.12
Ferric Oxide	Fe_2O_3	0.07
Aluminium Oxide	Al_2O_3	0.11
Silica	SiO_2	0.74
Combined Moisture	H_2O	0.63
Mn		160 ppm
F		70 ppm
Pb		2 ppm
As		0.4 ppm
Neutralising Value	as CaO	96.75

[1] Data supplied by Buxton Lime Industries.

There is a wide range of domains in which lime is used: as a binder for soil stabilisation, in the building industry, in steel manufacturing industry, in non-ferrous metallurgy, in the chemical industry, in paper manufacturing and for water treatment [BLA, 1990]. Lime does not exhibit any hydraulic property by itself and for this reason it can only be used for the stabilisation of materials having pozzolanic properties and the strength development process is slower than when cement is used.

The selection of a particular stabiliser should be based on an adequate laboratory study of the engineering properties of the individual materials for stabilisation and those of the stabilised materials. In general terms, lime reacts better with soils having particle size distributions ranging from fine to medium. The effect of lime is observed in two stages:

Lime modification

- a short term reaction with the clay minerals from which results a reduced plasticity and shrinkage, increased workability and trafficability, higher compactibility and bearing capacity. These improvements result from cation exchange of the sodium or potassium ions from the clay surfaces by calcium ions from the lime, for which the clay minerals have a greater affinity [Sherwood, 1993; Thomas, 1986];

Lime stabilisation

- a long term pozzolanic reaction that results in the improvement of the mixture's performance in terms of swelling and strength over the time. This results from the reaction of the lime, in the presence of water, with pozzolanic components of the materials to be stabilised such as clay minerals (aluminium silicates).

The pozzolanic reactions mentioned are extremely affected by curing temperature [PIARC, 1991]. At temperatures below 5 to 10 °C the reactions are practically suspended, while at 40 °C they develop 10 to 20 times quicker than at 20 °C. Hence, in tropical climates lime and cement frequently produce similar results both in terms of rate of strength development and of ultimate strength. Besides, the presence of particular substances have various effects on the stabilised materials such as their inhibition or retardation of the reactions (e.g. organic matter, nitrates), their acceleration (e.g. sulphate, calcium chloride when at low content), and their swelling risks (e.g. sulphate, sulphides).

Lime is used in the same way as cement for the activation of pozzolanic materials, which may, in turn, be used for the stabilisation of other aggregates. Lime / pulverised fuel ash systems may present some advantages in relation to cement / fly ash systems, such as the greater flexibility in the organisation of the construction works and reduced shrinkage due to the slower strength development. The latter may also be a problem for trafficability and solutions to accelerate the pozzolanic reactions have been attempted. The inclusion of calcium carbonate as an additive was shown to accelerate strength development and improve the performance of the mixtures in laboratory studies using routine tests, such as California bearing ratio (CBR), frost heave and conventional static triaxial tests [Pritchard, 1991; Littleton & Willavise, 1992]. Stabilised fly ash with lime contents as low as 2 % were reported to have satisfactory performances for embankments and capping layers, both under soaked and unsoaked conditions, and the addition of calcium carbonate to the mixture produced a non-frost-susceptible material.

When using lime to activate a secondary binder for the stabilisation of soil or granular materials containing clay, care should be taken since the lime reacts preferentially with the clay which also has pozzolanic properties, but originates compounds with lower cementitious characteristics [Sherwood, 1995]. A pre-treatment of the material with lime may be necessary prior to the addition of binder.

4.2.2. Secondary binders

Hydraulic road binders are here considered in a general way, including binders with hydraulic properties but also the ones with pozzolanic properties, in a similar approach as defined in the CEN draft on this issue [CEN, 1995 iii]. Hence, the secondary materials with properties which enable them to be used as secondary binders are presented below.

Hydraulic binders

- hydraulic or sulphocalcic fly ash
- granulated blastfurnace slag
- pelletised blastfurnace slag (fine fraction, 0 to 3 mm)

The slags need to be combined with a basic activator such as lime, cement, hydraulic fly ash, calcium or sodium hydroxide.

Pozzolanic binders

- silicoaluminous fly ash
- natural pozzolans and volcanic ash
- thermally activated clays and shales.

Pozzolanic binders need to be activated with materials having high percentages of calcium oxide, for example, cement or lime.

From these materials, the ones with particular interest for this project have already been described in Chapter 2. Comparatively, pozzolanic binders are characterised by having lower calcium oxide contents (< 5 %) and, when used for stabilisation, the resulting materials develop strength more slowly. However, all these binders set slowly when compared with conventional cement, which is why they are more appropriate for road construction.

Secondary binders may be used with conventional or secondary aggregates and in combination with conventional or alternative binders. The use of a secondary binder is beneficial, since it permits the application of higher percentages of materials with reduced market value when compared with conventional binders. The resulting mixtures will benefit from a more uniform and abundant distribution of binder. As a result, unreacted binder will be available in the long term to re-cement cracks in the presence of moisture.

4.3. Hydraulic stabilisation of pavement materials

4.3.1. Mechanical stabilisation

A complete laboratory study of hydraulic mixtures should include an initial mechanical stabilisation of the aggregates. Grading characteristics are very important for stabilisation and affect the type and amount of binder required. One of those characteristics is the coefficient of uniformity (Cu) which is defined as follows:

$$C_u = D_{60} / D_{10} \tag{4.1}$$

where: D_{60} particle diameter at which 60 % of the material by weight is finer;

D_{10} particle diameter at which 10 % of the material by weight is finer.

The bonding action resulting from stabilisation develops essentially at the contact points between particles. The lower the coefficient of uniformity is (i.e. the more uniform the particles are), the higher percentage of binder necessary for its stabilisation. Well graded aggregates have the advantage of producing highly compacted materials and due to the increase in inter-particle contact area they are more adequate for stabilisation producing higher strength results.

The mechanical stabilisation process consists of the combination of the aggregate to be stabilised with another inert material with different particle size distribution (generally from 0 to 5 mm) so that the grading of the mixture is more adequate for hydraulic stabilisation. Ranges of grading are recommended in the specifications [DoT, 1993], and are generally based on the equations originally proposed by Fuller [Fuller, 1905].

Hence, the main objectives of a grading correction are the production of a mechanically stable material, reaching higher densities after compaction (and consequently higher bearing capacities), which will optimise the use of binder and facilitate the laying and compacting operations. The materials to consider for grading correction may be some of the secondary binders mentioned above and in this way they have a double action of grading corrector and binder.

4.3.2. Mix design procedure

Once the type of binder has been chosen, the mix design of hydraulic bound materials involves the following main criteria:

- optimisation of compaction moisture content to maximise the density;
- optimisation of binder content to meet the requirements of the layer where it is going to be used.

The first is determined using the compactibility test. Note that for determining the optimum moisture content the binder has to be included in the mixture tested, since it affects the water demand for compaction. In this procedure it is assumed that the amount of water corresponding to the optimum moisture content should be sufficient to hydrate all the binder used and give origin to the best mechanical properties. This is not always the case and the consideration of more than one level of moisture during a stabilisation study in the laboratory may be necessary to fully optimise the mixture. This aspect will be further discussed in Chapter 8.

The binder content for stabilisation is determined in order to meet the minimum target requirements, generally specified in terms of compressive strength [DoT, 1993]. Specimens are compacted over a range of binder contents, then cured and tested. The strength values obtained are plotted against the percentages of binder used. Finally, the binder content to meet the specified target strength may be chosen from this type of chart. The differences occurring between laboratory and site conditions, in terms of mixture preparation and compaction, should be taken into account. It is also necessary to bear in mind that the strength results reported are average values (instead of characteristic values usually adopted for concrete). For this reason a safety margin is generally considered when selecting the binder content.

4.3.3. Specification requirements

Presently, the Specification for Highway Works [DoT, 1993] includes three classes of stabilised capping materials:

- class 9A cement stabilised well graded granular material;
- class 9B cement stabilised silty cohesive material;
- class 9C cement stabilised conditioned pulverised fuel ash cohesive material;
- class 9D lime stabilised cohesive material.

For sub-base and base layers it specifies the requirements of four categories of cement bound materials, CBM1, CBM2, CBM3 and CBM4. All categories can be used at sub-base levels but those of better quality, CBM3 and CBM4 are generally used for base layers, with the exception of rigid pavements where they are applied in sub-bases, while CBM1 and CBM2 are preferred for sub-base construction due to that layer's lesser requirements.

A summary of the requirements for both stabilised capping and cement bound materials is presented in Figures 4.2 and 4.3 and in Table 4.3 [DoT, 1993 & 1993 i].

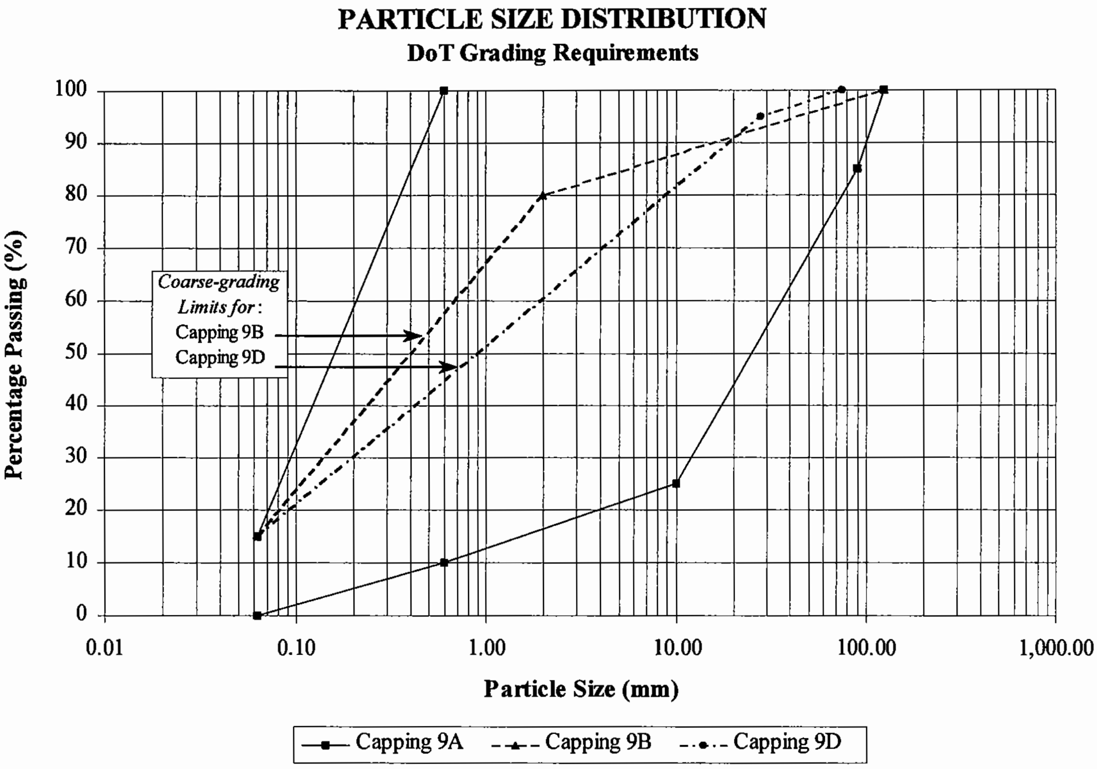

Figure 4.2 Summary of grading requirements for stabilised capping layers.

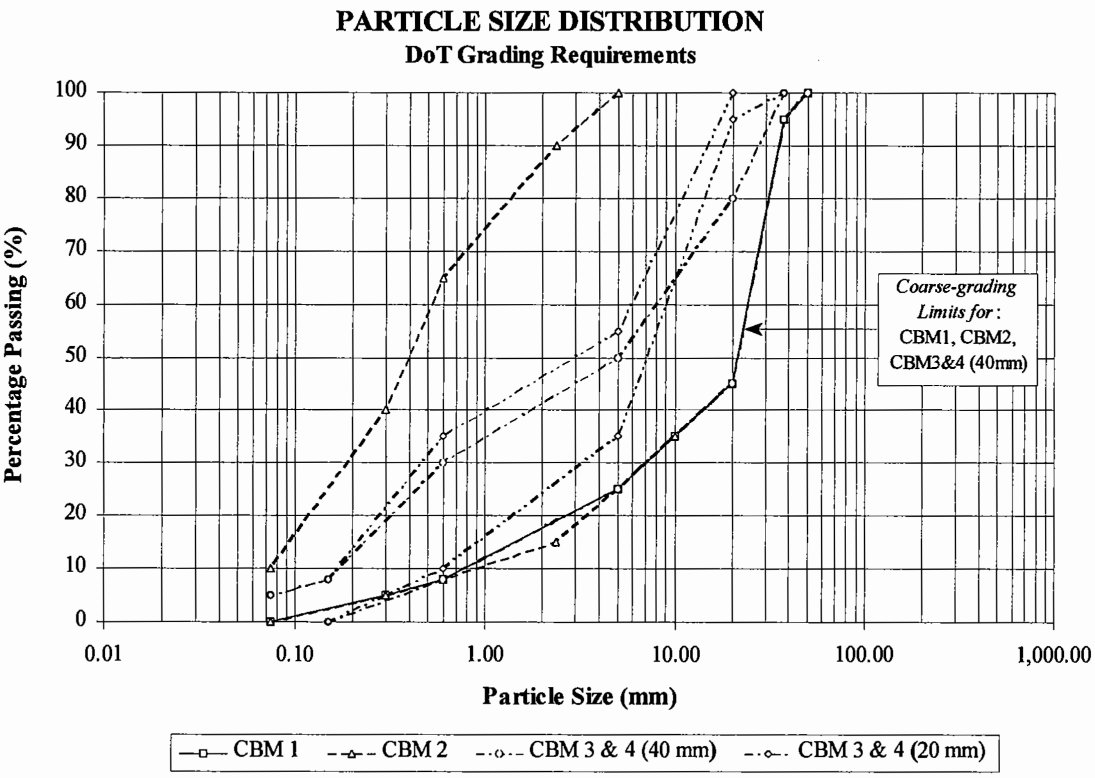

Figure 4.3 Summary of grading requirements for cement bound materials.

Table 4.3 Summary of DoT requirements for stabilised capping layers and cement bound materials.

Property		Class of Material			
Stabilised Capping Materials [1]					
		Class 9A	Class 9B	Class 9C	Class 9D
Coefficient of Uniformity		NS	≥ 5	NS	NS
Liquid Limit		≤ 45	≤ 45	NS	NS
Plasticity Index		≤ 20	≤ 20	NS	≥ 10
Organic Matter (%)		≤ 2	≤ 2	NS	≤ 2
Total Sulphate Content (%)		≤ 1	NS	≤ 1	NS
Pulverisation (%)		≥ 60	NS	≥ 60	≥ 30
Moisture Condition Value (%)		NS	≤ 12	NS	NS
Binder Content (%)		cement ≥ 2	cement ≥ 2	cement ≥ 2	lime ≥ 2.5
Bearing Ratio (%)		≥ 15	≥ 15	≥ 15	≥ 15
Cement Bound Materials [2]					
		CBM 1	CBM 2	CBM 3	CBM 4
Maximum Nominal Size (mm)		NS	NS	40 or 20 [3]	40 or 20 [3]
Water Absorption (%)		NS	NS	≤ 2 [4]	≤ 2 [4]
Soundness Value (%)		NS	NS	> 75	> 75
TFV $_{soaked}$ (kN)		NS	≥ 50	NS	NS
7-day Compressive Strength (MPa)	Average	4.5	7.0	10.0	15.0
	Minimum	2.5	4.5	6.5	10.0
Immersion Test (%) [5]		≥ 80	≥ 80	NS	NS

Key: NS = Not Specified

[1] ***Permitted Materials for Capping***: Class 9A - any material, or combination of materials, other than unburnt colliery spoil and argillaceous rock; Class 9B - any material, or combination of materials, other than chalk, unburnt colliery spoil and argillaceous rock; Class 9C - conditioned pulverised fuel ash; Class 9D - any material, or combination of materials, other than unburnt colliery spoil.

[2] ***Permitted Materials for CBMs***: CBM1 - no restrictions; CBM2 - gravel-sand, washed or processed granular material, crushed rock, all-in aggregate, blastfurnace slag or any combination of these; CBM3 & 4 - natural aggregate, crushed concrete, crushed air-cooled blastfurnace slag, pulverised fuel ash as part of the aggregate.

[3] When the spacing between longitudinal reinforcement is less than 90 mm, the nominal size of coarse aggregate should not exceed 20 mm.

[4] If greater than 2 % then the soundness test should be carried out.

[5] After the 7 days immersion period the specimens should not show any signs of cracking or swelling.

4.4. Testing programme

From the information collected and results obtained in Chapter 3, it seems that all materials studied have some potential for use in pavement foundations. Table 4.4 summarises the level of treatment thought necessary for this purpose based on the results obtained with the unbound material. In terms of economical viability, cement contents up to 15 % may usually be employed [Kennedy, 1980]. China clay sand and bottom ash have an aggregate matrix and their properties are close to a suitable sub-base material. In fact, their unbound application in low-trafficked roads should be considered. For fly ash, light to moderate stabilisation was proposed to compensate for the lack of an aggregate as a main component of the mixtures which reduces the short-term bearing capacity of the material. For minestone the highest levels were considered for this study due to the very poor results obtained when unbound, in particular the important particle instability when in contact with water.

Table 4.4 Estimation of necessary treatment levels.

Material	**Proposed Levels of Treatment**			
	None (c = 0 %)	**Light** (c = 2 - 4 %)	**Moderate** (c = 4 - 7 %)	**Heavy** (c ≥ 7 %)
Minestone			✓	✓
China Clay Sand	✓	✓		
Slate Waste	✓			
Furnace Bottom Ash	✓	✓		
Pulverised Fuel Ash		✓	✓	

Key:
- c — indicative values in terms of cement content.
- Light — Treatment Level I.
- Moderate — Treatment Level II.
- Heavy — Treatment Level III (considered in Chapter 8).

For the formulation of the mixes used in this project the technical guidelines presented in Chapter 2 were taken into account together with the technique used in France for this purpose [SETRA & LCPC, 1980]. Although those formulations were developed specifically for the French traffic and climate conditions, they give a good indication of the levels of treatment necessary for the stabilisation of the same materials in other countries with similar conditions. The proposed formulas are indicated in Table 4.5 and the main objectives of those mixes are to obtain:

- slow setting time to increase the flexibility in laying and compacting the layers;
- immediate stability to enable initial trafficking after short periods of time;
- high stiffnesses and resistance to fatigue for the long term.

Table 4.5 Formulation of mixes proposed in France [SETRA & LCPC, 1980].

Components	**Stabilised Materials**			
(by %)	cement-bound courses	slag-bound courses	fly ash-lime-bound courses	pozzolana-lime-bound courses
aggregate	95.5 to 96.5	80 to 85	85 to 90	85
cement or hydraulic fly ash	3.5 to 4.5			
granulated or pelletised slag		15 to 20		
fly ash			8 to 12	
pozzolans				12
lime		1	2 to 3	3
water	4 to 7	6 to 9	4 to 9	6 to 9

Two levels of treatment were proposed for this project, corresponding to the lower end of these formulations (Tables 4.4 and 4.5), with the objective of achieving a light stabilisation of these materials. Table 4.6 summarises the materials and binders considered for each treatment level. A testing programme was established to assess the performance of the selected mixes. The aim of the programme was to describe performance according to existing procedures, in particular the DoT Specification, and the appropriate British Standards as follows:

- compactibility tests [BS 5835, 1980];
- compressive strength tests performed on 100 mm-cubic specimens [BS 1924, 1990];

- immersion tests [BS 1924, 1990]; and
- determination of frost heave [BS 812, 1989].

It also included repeated load triaxial tests which will be the subject of Chapter 6.

All specimens were compacted using the optimum values of moisture content (OMC) and dry density determined in the compactibility test, with the exception of minestone. Stabilised minestone specimens were compacted throughout this research using OMC + 2 % in accordance with the technical guidelines presented in Chapter 2. Initially, this material was mixed with lime in dry conditions. Then, the amount of water corresponding to the OMC was added and further mixing carried out, after which the mixture was allowed to stand for 24 hours. After this period, the extra 2 % of water was added, the components thoroughly mixed and then compacted. This procedure was found to facilitate the compaction process.

Lime instead of cement was used as a binder for the stabilisation of minestone (MI1) or as an activator for the fly ash (MI4). Minestone's stabilisation with cement is already the subject of several research studies and has been widely covered in the bibliography [Rainbow *et al.*, 1984; British Coal, 1986; Thomas, 1986; Carr & Withers, 1987; Kettle, 1990]. On the other hand, lime stabilisation has not been fully investigated. In principle, the use of this stabiliser should be possible due to the reported pozzolanic properties of some minestones [Chi, 1992; Sherwood, 1995] and because they contain clay minerals that can react with lime. In 1977, a report of the Organisation for Economic Co-operation and Development made reference to French studies centred on the pozzolanic properties of colliery shales with possible use of the fine particles as a binder, combined with lime, for subgrade stabilisation. These properties have not been fully exploited in pavement construction.

Table 4.6 Testing programme and summary of conventional test results.

Material	Material Code	Binder (%) None	Binder (%) Cement	Binder (%) Lime	Binder (%) Fly ash	Compactibility Tests OMC	Compactibility Tests MDD	Compressive Strength (1) 7 days	Compressive Strength (1) 14 days	Immersion Tests	Frost Heave (1)	Repeated Load Triaxial Tests
			%	%	%	%	Mg/m^3	MPa	MPa	%	mm	@ 24h / 28d (2)
UNBOUND												
China clay sand	CC6	✓				10.0	1.93	Not Applicable			8.2	Results in Chapter 6
Granite Type 1	T1	✓				6.5	2.19				Not Tested	
Slate waste	SW1	✓				6.6	2.23				8.0	
TREATMENT LEVEL I												
China clay sand	CC4		2			8.7	1.97	1.56	1.80	114	3.0	
Furnace bottom ash	BA4		2			22.2	1.24	0.27	0.24	77	0.0	
Minestone	MI1			5		8.7	2.07	0.54	0.74	63	4.3	
Pulverised fuel ash	FA2		2			19.5	1.41	0.99	1.60	97	6.5	
TREATMENT LEVEL II												
China clay sand	CC2		2		8	8.3	2.04	Not Tested			Non-susceptible	
Furnace bottom ash	BA3		2		8	19.4	1.29				Non-susceptible	
Minestone	MI4			3	12	8.0	2.04				Non-susceptible	
Pulverised fuel ash	FA3		5			19.0	1.42				Non-susceptible	

Key: OMC = Optimum Moisture Content MDD = Maximum Dry Density

(1) Cement stabilised specimens tested at 7 days and lime stabilised specimens tested at 28 days, according to BS 1924 [1990].

(2) Unbound specimens tested after 24 hours and treated specimens tested after 28 days curing.

4.5. Results

A summary of the conventional test results obtained is presented in Table 4.6. The characterisation focused on the utilisation of the mixtures at sub-base level or above. For this reason, the requirements for stabilised capping were not verified exhaustively. Nevertheless, with the exception of minestone which is explicitly excluded from all classes, the other materials may be used according to the UK DoT Specification for stabilised capping layers. In fact, for this layer the requirements are quite broad enabling the use of a wide range of materials. Taking into account the results when unbound, the following factors may be pointed out:

- china clay sand was found to meet the DoT requirements for unbound capping;
- the main problem with furnace bottom ash was the low Ten per cent Fines Value which is not a requirement for capping; and
- pulverised fuel ash is specifically considered for Class 9C.

On the other hand, all stabilised materials studied failed to meet the limits specified for CBM1 by a large margin, with furnace bottom ash (BA4) and minestone (MI1) resulting in particularly poor results. The assessment of the durability of these last two materials is also of concern due to a greater than 20 % drop in strength in the immersion test. A positive result is the substantial reduction in frost heave due to stabilisation, even with percentages of binder as low as 2 %. In particular, the fly ash was frost susceptible when tested in an unbound form and its stabilisation with 2 % cement reduced the problem to a level that may be expected with conventional crushed aggregates.

The materials with the lowest levels of stabilisation (treatment level I) are clearly excluded for sub-base construction and other applications at higher levels in the pavement structure on the basis of their strength. The evidence collected in the bibliographic review indicates that some of these materials have been successfully used in road construction either in the UK or abroad reinforcing their potential.

Studied more closely, the strength requirements in the DoT Specification for cement bound materials appear to be too harsh and restrictive in relation to stabilised secondary materials. In the USA, the mix design procedure is based on the strength and durability of the materials. The former is assessed by compressive strength tests [ASTM D1633 & D5102, 1990] and the latter using wetting-drying and freeze-thaw durability tests [ASTM D559 & D560, 1989]. It has been demonstrated that soil-cement mixtures with compressive strengths of 4.5 MPa (identical to the limit imposed in the UK for CBM1) have a 95 % sample survivability under the severe testing conditions imposed by those two durability tests [Saylak, Taha & Little, 1988]. Hence, the limits imposed for the compressive strength of stabilised materials are related to the need for passing through durability tests that have been recognised as being too severe and not because those materials are subject to similar levels of stresses when used in pavements. Different limits have been proposed in the bibliography, in particular the recent draft for European Standard [CEN, 1995 i] which proposes five grades of materials, the first of which has no minimum requirement for compressive strength (Table 4.7 gives some examples of the strength requirements proposed). Apart from this exception, however, the mixtures tested are too weak to be accepted either for cement bound material or soil-cement according to the sources listed in Table 4.7.

Table 4.7 Compressive strength requirements proposed by several sources.

Material	**Compressive Strength**		**Notes**	**Source**
	Limit (MPa)	Age (days)		
Fly ash bound mixtures	≤ 2	360	Grade 1	[CEN, 1995 i]
	2	360	Grade 2	
	5	360	Grade 3	
	8	360	Grade 4	
	11	360	Grade 5	
Soil-cement (1)	2.4	7	Light to medium traffic	[Saylak *et al.*, 1993]
	3.1 to 4.5	7	Medium to heavy traffic	
Cement bound fly ash	5	28	For roadbases	[Bolt, 1987]
Cement bound fly ash	2.8 to 3.1	7	Cylindrical specimens	[DiGioia *et al.*, 1986]
	3.5	7	Cubic specimens	
	5.5	7	Maximum value (2)	
Lime bound fly ash	3.8 to 4.1	28	Cylindrical specimens	

Key: (1) Material corresponding to CBM1.
(2) Maximum strength recommended to avoid distinct cracking which may reflect through the bituminous surface layer.

4.6. Inadequacy of present approach

It is important to relate the requirements to be imposed on road materials to the demands of the layer in which they are going to be used, namely the level in the pavement, the type of structure and traffic. In addition, the properties determined should be those the material needs to possess to fulfill its functions and assure an adequate performance in the pavement structure. The fundamental properties required, directly related to performance in pavement structures, are:

- resilient modulus, to spread the traffic load and prevent overstressing of underlying soils, in particular the subgrade;
- resistance to permanent deformation, in particular for unbound materials to prevent rutting;
- tensile strength and resistance to fatigue of stabilised materials, to prevent cracking;
- durability of both unbound and stabilised materials to assure an adequate performance throughout the life of the pavement.

These properties are not included for material characterisation in the present approach to the use of secondary materials that has been covered up to this chapter, with the exception of durability whose evaluation is considered through the immersion test. Instead, other requirements are specified emphasising properties of the particles rather than those of the bulk compacted material. For stabilised mixtures, the property requirements are in terms of unconfined compressive strength which are not the most important to characterise cement bound materials, as compressive strength is not a determinant factor in pavement failure mechanisms. Indeed, the mechanisms generally considered for design purposes and which will be described in more detail in Chapter 9 are as follows:

- cracking of bound layers which involves mechanical properties such as resilient modulus, Poisson's ratio, tensile strength and fatigue resistance;
- rutting resulting from the accumulation of vertical permanent strains in all layers

of the pavement, but especially in the unbound layers. The main properties involved in rutting are resilient modulus, Poisson's ratio and resistance to permanent deformations.

Furthermore, the development of properties with the time is not taken into account in the present specifications and the ages considered (generally 7 days and, in particular cases, 28 days) will normally be restrictive as far as the use of slow cementing secondary materials is concerned because intense trafficking may not occur until several months after placement.

Overall, the conventional type of specification, of a recipe nature, leads to an inadequate assessment of alternative materials for road construction both in unbound and stabilised forms. For this reason, it is desirable to advance towards the development of end-product specifications. There are already examples of specifications moving in this direction, such as the ones for cement bound materials, though 'recipe' restrictions concerning the type of material, grading and properties of the aggregate particles still remain.

In conclusion, standards specifying the relevant mechanical properties of end-products and requiring their assessment through performance-based tests are the way forward in pavement materials' engineering. For this to become a reality, it is necessary to consider other types of tests, capable of determining the fundamental mechanical properties of the materials that may be used in mechanistic methods of pavement design and structural assessment for maintenance purposes. These aspects, together with the essential developments of methodologies for material evaluation, testing equipment and techniques to enable the utilisation of new materials in road construction, will be the main subject of the subsequent chapters.

4.7. Summary

- Some of the secondary materials investigated possess hydraulic or pozzolanic properties enabling their use as secondary binders. These materials are identified and the process for stabilisation is described in general terms, including mechanical stabilisation, mix design and the present specification requirements to be met by stabilised materials. The action of the main stabilisers that results in the stabilisation of pavement materials is described.

- The testing programme set up to assess treated secondary materials against the present specifications included, essentially, compressive strength and immersion tests and the determination of frost heave.

- All materials, as treated, failed to meet the lowest strength requirement for cement bound materials and some questions were raised as to the durability of lime stabilised minestone and cement stabilised bottom ash, for the levels of treatment considered. A significant reduction in frost heave after stabilisation was observed for all materials studied.

- The present DoT strength requirements appear to be too harsh and restrictive in relation to stabilised secondary materials. Alternative limits proposed by several sources were presented but these may also be high.

- Overall, the assessment of secondary materials using existing specifications is not satisfactory because their assessment is not based on fundamental mechanical properties, directly related to material performance in pavements or involved in pavement failure mechanisms. The development of new performance-based specifications is desirable but these will require new tests, testing equipment and techniques to be considered.

CHAPTER 5

TESTING EQUIPMENT DEVELOPMENT

5.1. Introduction

The need to consider new tests and testing equipment for a rational characterisation of pavement materials was demonstrated in the last chapter. Here, two equipments capable of determining performance-related properties will be presented:

- first, the repeated load triaxial apparatus mainly used for unbound and lightly treated materials; and
- second, the repeated load indirect tensile apparatus for the characterisation of stabilised materials, for which the first equipment cannot provide the necessary characterisation in tension.

These equipments meet the requirements for a mechanical characterisation of pavement materials. In particular the determination of resilient modulus, permanent deformation and triaxial strength in the triaxial apparatus and tensile strength, stiffness modulus and fatigue characteristics in the indirect tensile apparatus. It is the objective of this chapter to describe them together with the developments made throughout this research. The issues facing the triaxial and indirect tensile tests will be examined in Chapters 6 and 7 respectively.

5.2. The repeated load triaxial apparatus

5.2.1. Introduction

The repeated load triaxial apparatus (150 x 300 mm) was first developed by Boyce [1976] providing the University of Nottingham with an important test facility to study several aspects of the performance of unbound granular materials under repeated loading. Since then, this equipment has been modified and upgraded by several researchers and fully described [Boyce, 1976; Pappin, 1979; Brown, O'Reilly & Pappin, 1989; Karasahin, 1993].

The repeated load triaxial tests are performed on specimens of granular materials of 150 mm in diameter and 300 mm high. The equipment is capable of cycling both deviatoric and confining stresses and permits on-sample measurement of axial and radial strains leading to more accurate measurements of behaviour. A general view of the triaxial rig and some aspects of the triaxial specimen and instrumentation used for the measurement of deformations are shown in Figures 5.1 to 5.3. A brief description of the present configuration is presented below and the modifications made during this research identified.

5.2.2. Loading system

The loading of the triaxial specimens is carried out inside the triaxial cell through hydraulic actuators controlled by servovalves. The maximum working load for the axial actuator is 20 kN which corresponds to axial stresses on a 150 mm specimen of up to 1150 kPa approximately. Frequencies of loading up to 16 Hz may be applied [Boyce, 1976].

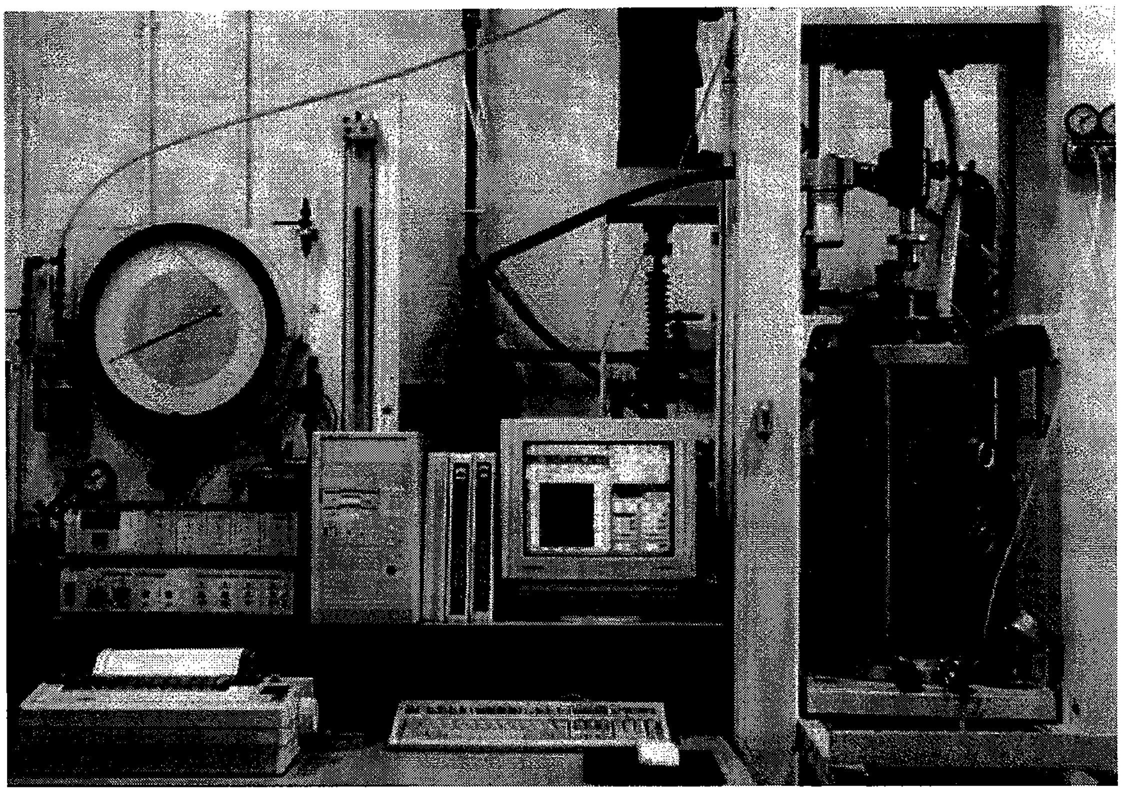

Figure 5.1 View of the repeated load triaxial apparatus.

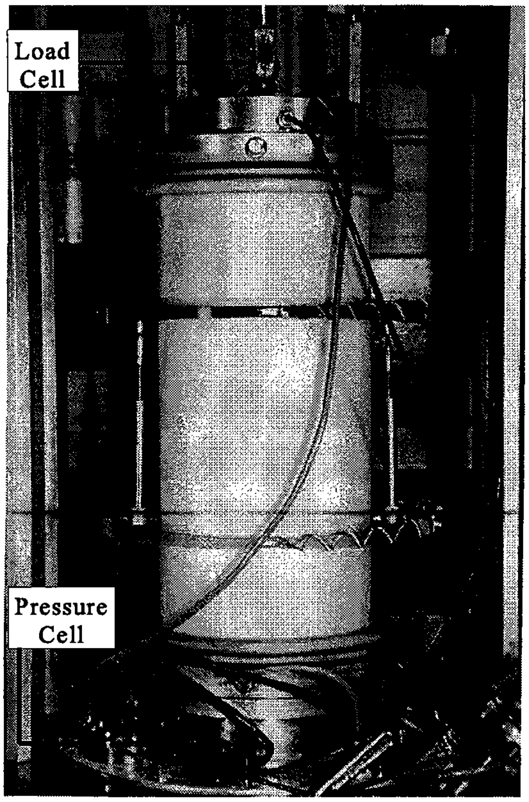

Figure 5.2 Triaxial specimen.

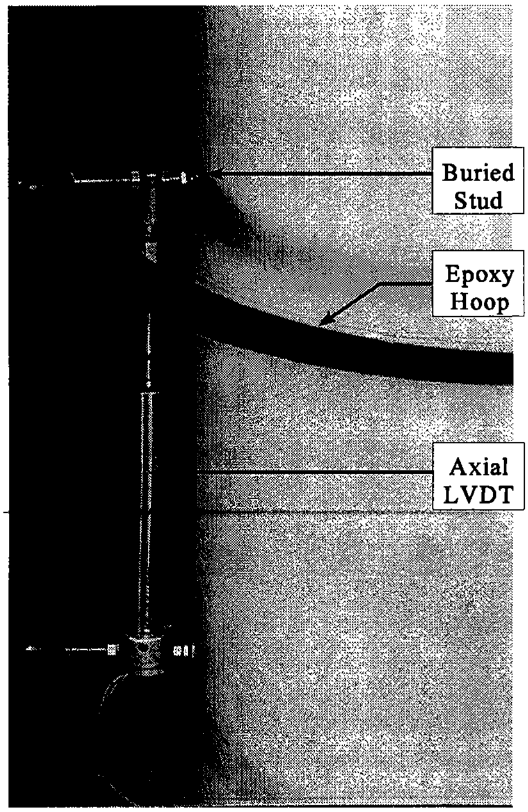

Figure 5.3 Detail of triaxial instrumentation.

For the application of confining pressure to the specimen a medium has to be used inside the triaxial cell. This is generally air for constant confining pressure tests while silicone oil is preferred for variable confining pressure tests. Silicone oil has a relatively low viscosity, is chemically inert and has non-conductive characteristics that makes it especially suitable for this application. Confining stresses up to 400 kPa at frequencies up to 2 Hz may be applied [Boyce, 1976].

The applied axial load and confining pressures are continuously monitored by the load cell and pressure cell. The original load cell [Boyce, 1976] was located in the lower part of the loading rod, which was milled down from a 19.05 mm (3/4 inch) circular section to a 12 mm square section (Figure 5.2). Foil strain gauges were fixed to all faces of the squared section. This load cell possessed an adequate sensitivity and linearity and as it was placed inside the triaxial cell the readings were not affected by friction between the loading rod and the top of the triaxial cell. However, the uncovered strain gauges were found to be particularly susceptible to electrical noise especially when air was used as the cell fluid for constant confining pressure tests. In addition, due to the rigidity of the loading rod, the top platen was unable to perform small rotations to adapt to the top of the specimen, giving rise to the application of non-uniform axial stresses. For these reasons, a new load cell was designed, the most particular features being the inclusion of a shear pin and an integral radial spherical bearing (Figure 5.4) to allow small rotations in any vertical plane while maintaining the capability of measuring compression as well as tension essential for the performance of triaxial extension tests. The strain gauges are located inside the metallic base of the cell, which is in direct contact with the top platen, solving the problems related with electrical noise.

5.2.3. System for measuring deformations

Axial deformations are measured using two Linear Variable Differential Transformers (LVDT) placed in the central 150 mm of the specimen and diametrically opposite to each other (Figures 5.2 and 5.3).

For the measurement of radial deformations two epoxy hoops are used at two different levels, 1/4 and 3/4 of the specimen height (Figures 5.2 and 5.3). These have narrower sections where the strain gauges are fixed to achieve higher sensitivities [Boyce, 1976].

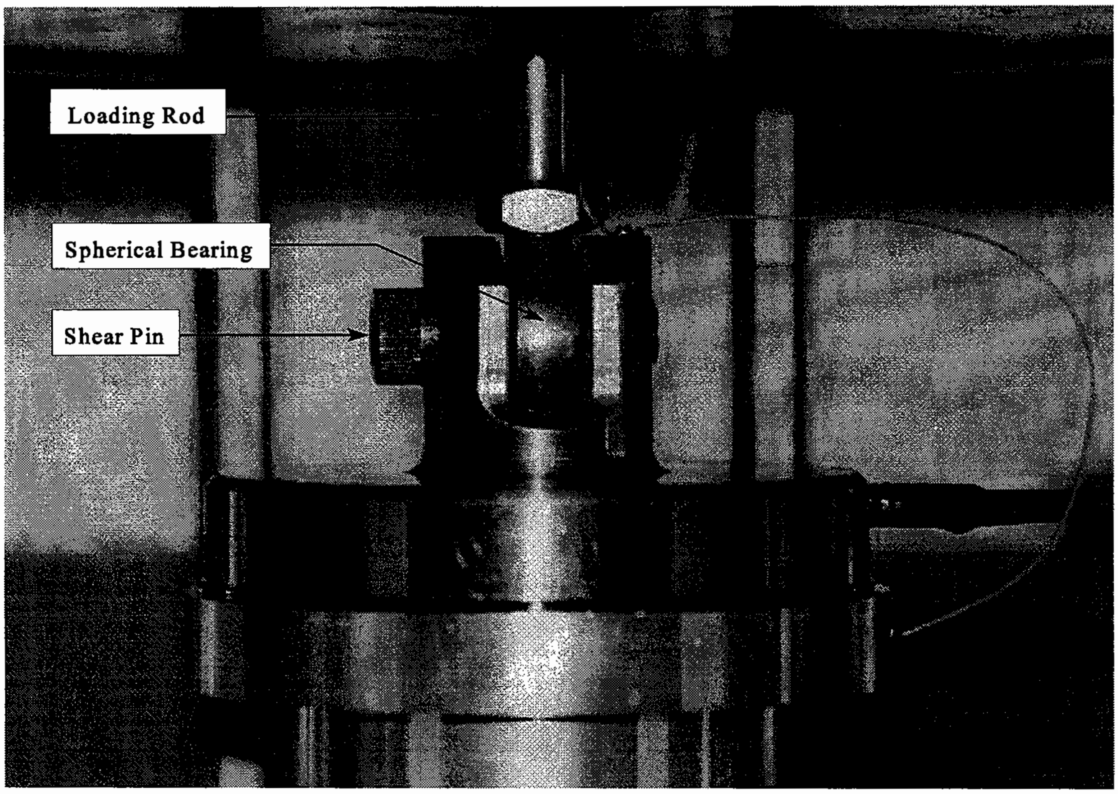

Figure 5.4 New load cell for the triaxial equipment.

The deformation measuring points are materialised in the specimen by four location studs placed before the compaction of the material. In some cases, the original studs [Boyce, 1976] were found not to produce a firm embedment in the specimen due to the difficulties of material compaction around the studs and superficial location of the studs. For this reason, a modified version was manufactured incorporating a necked extension that would enable a deeper embedment of the studs in the compacted material. Similar studs have been used in France [Paute, Lefort & Benaben, 1990]. A diagram of the modified stud is shown in Figure 5.5.

5.2.4. Digital control system

The complete computer-controlled system to perform repeated load triaxial tests and automatically conduct the data acquisition is represented schematically in Figure 5.6. It comprises three major components [Chan & Sousa, 1991]: testing apparatus, interface hardware and software.

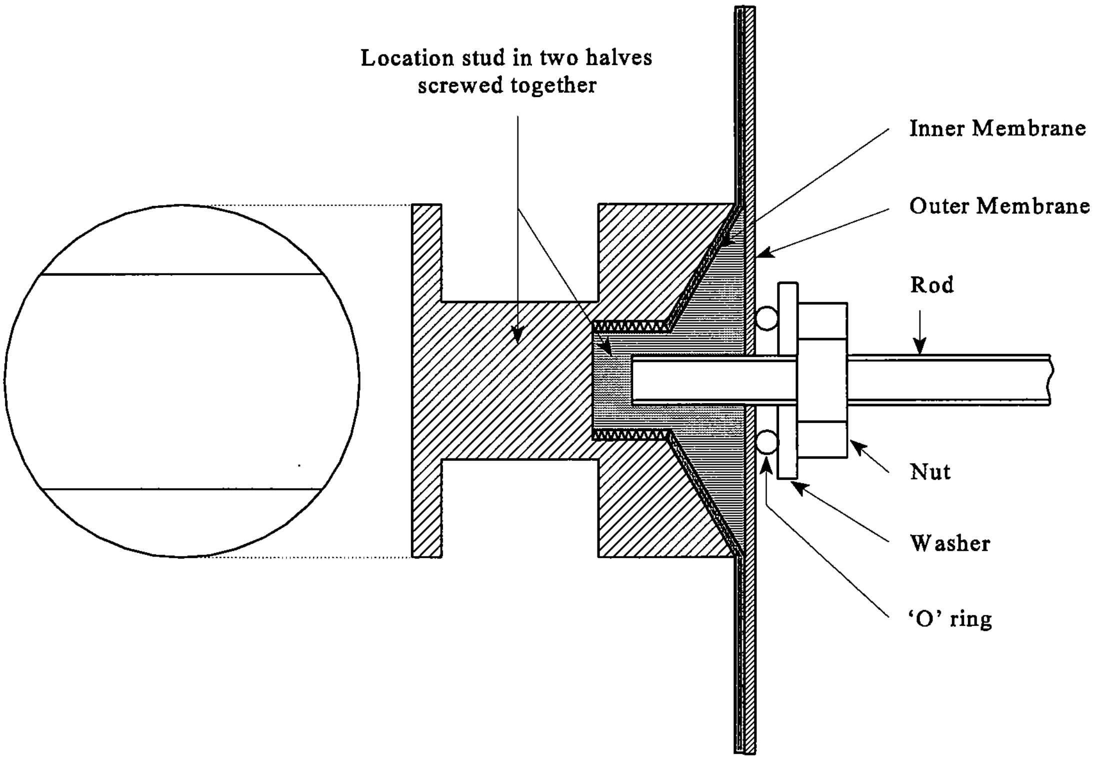

Figure 5.5 Location stud.

The feedback closed-loop control system shown in the figure operates as follows:

- according to the stresses one wants to apply to the specimen, which are entered into the control program, a signal is sent through the D/A converter and through the controller to the servovalve which in turn activates the hydraulic actuator to apply the desired stress;
- the response of the specimen is monitored by output transducers, in particular to measure the axial load and confining pressure, the load and pressure cells. Their signals are sent back to the computer, i.e. back to the control algorithm, through the transducer signal conditioner unit and the A/D converter;
- the control software compares the output signals with the command signals. If the amplitude differs by more than the control tolerance specified by the user the

equipment sends an adjustment signal to the servovalve so as to adjust to the desired levels the load / stress applied by the hydraulic actuators.

The computer program currently used at the University of Nottingham is ATS (Automated Testing System) which has been described by Sousa & Chan [1991]. Apart from the control algorithms and data acquisition, this software includes monitoring, data analysis and reporting capabilities. However, the direct use of the data analysis and report programs is not advisable because the calculations are based on maximum and minimum values obtained directly from the raw data without filtering to account for the effects of electrical noise on the readings.

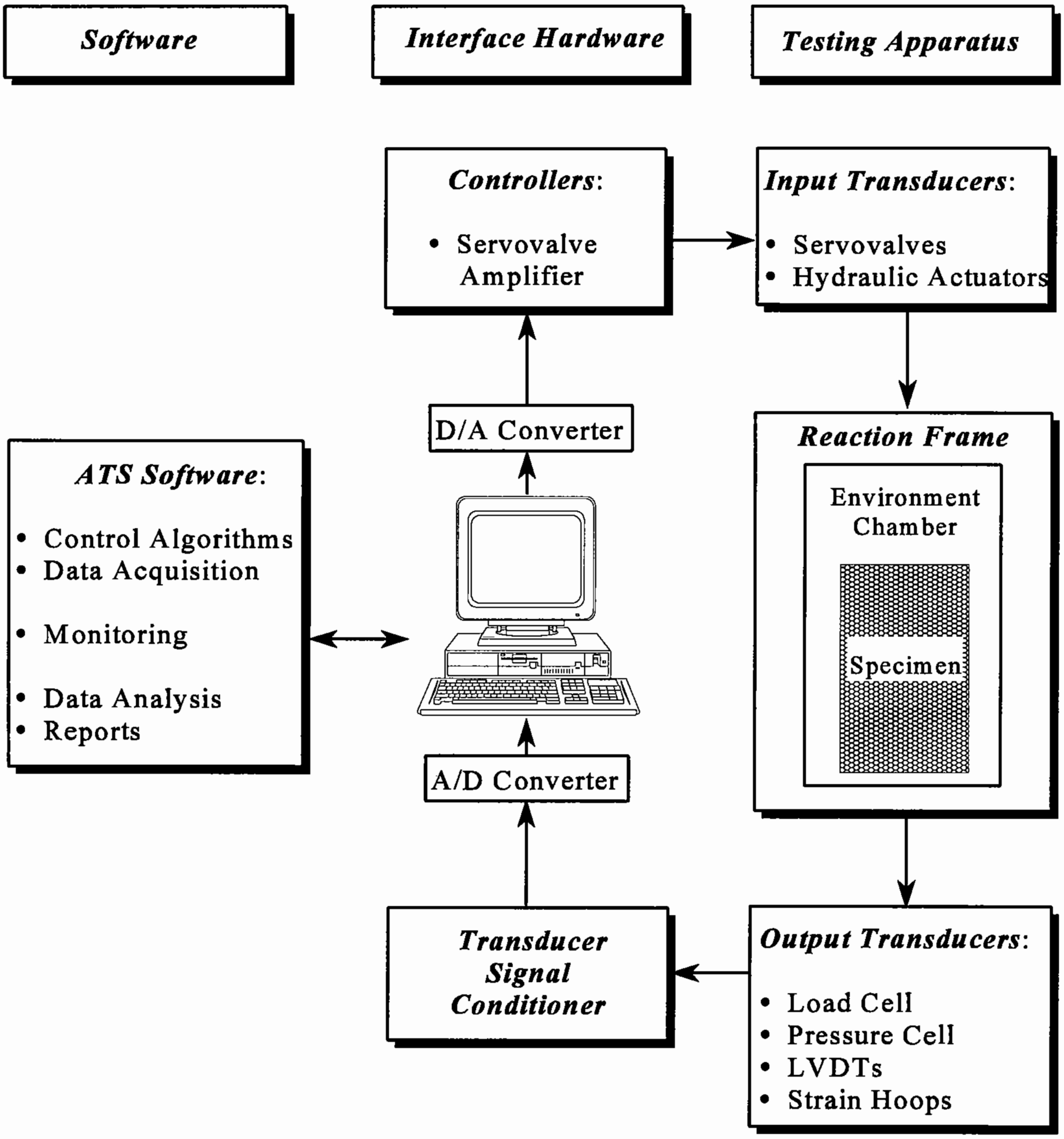

Figure 5.6 Digital control system in the repeated load triaxial apparatus.

5.3. The Nottingham Asphalt Tester (NAT)

The Nottingham Asphalt Tester was developed in the University of Nottingham and evolved from a rig for static creep on bituminous materials [Brown et al., 1995]. It is a commercially available computer-controlled pneumatic equipment for determining the fundamental properties of bituminous materials on a routine basis. The system comprises a pneumatic actuator that applies repeated loading to the specimen and LVDTs to measure the corresponding deformations. In Figures 5.7 and 5.8 the conventional NAT and the sub-system used for stiffness modulus determination are presented. Stiffness modulus is here used in accordance with the terminology currently adopted in the field of bituminous and hydraulically bound materials and compares with the term resilient modulus referred for unbound and lightly treated materials.

The NAT equipment is now reasonably established for testing bituminous materials, and it is already the subject of Drafts for Development of the British Standard Institution. There are four types of test that can be performed on bituminous materials in the NAT equipment:

- indirect tensile stiffness modulus test [BS DD 213, 1993];
- resistance to permanent deformation in unconfined uniaxial loading [BS DD 185, 1993]. This test may be conducted under static or repeated loading (static / dynamic creep test);
- fatigue characteristics in indirect tensile mode [BS DD ABF, 1996].

The tests can be carried out on specimens cored from the pavement or compacted in the laboratory, being able to accommodate specimens with diameters ranging from 100 to 200 mm, 100 mm and 150 mm being the most commonly used. Their thickness may vary from 20 to 80 mm [Cooper, 1994].

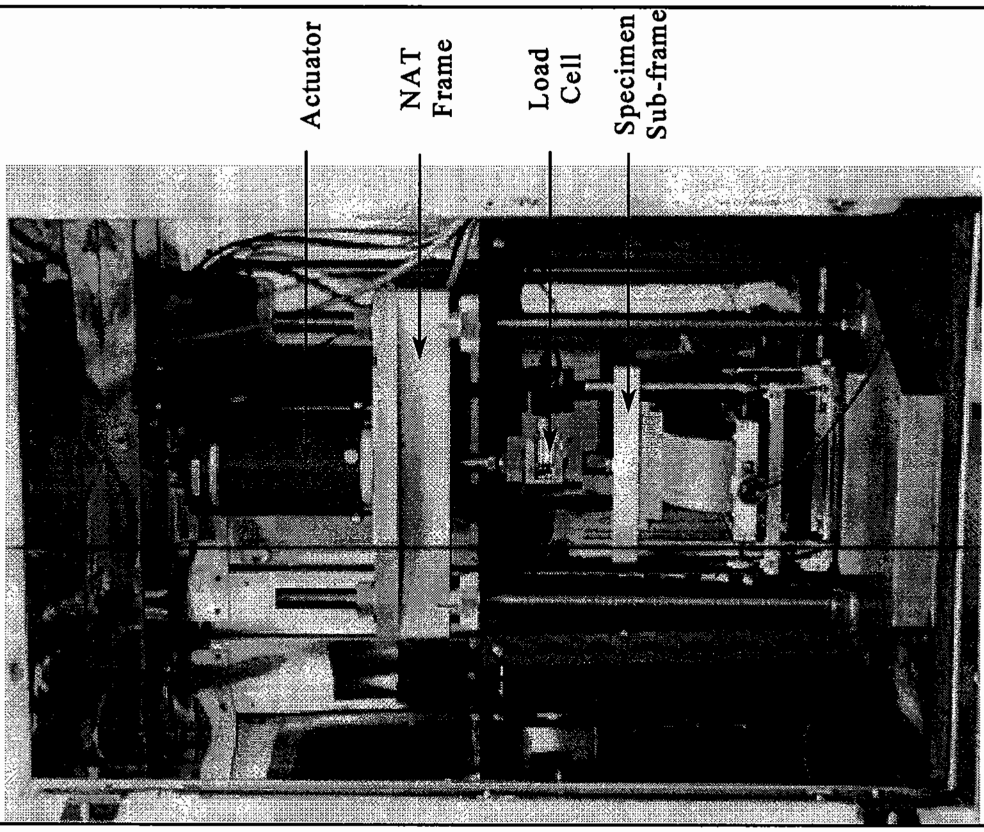

Figure 5.7 Nottingham Asphalt Tester.

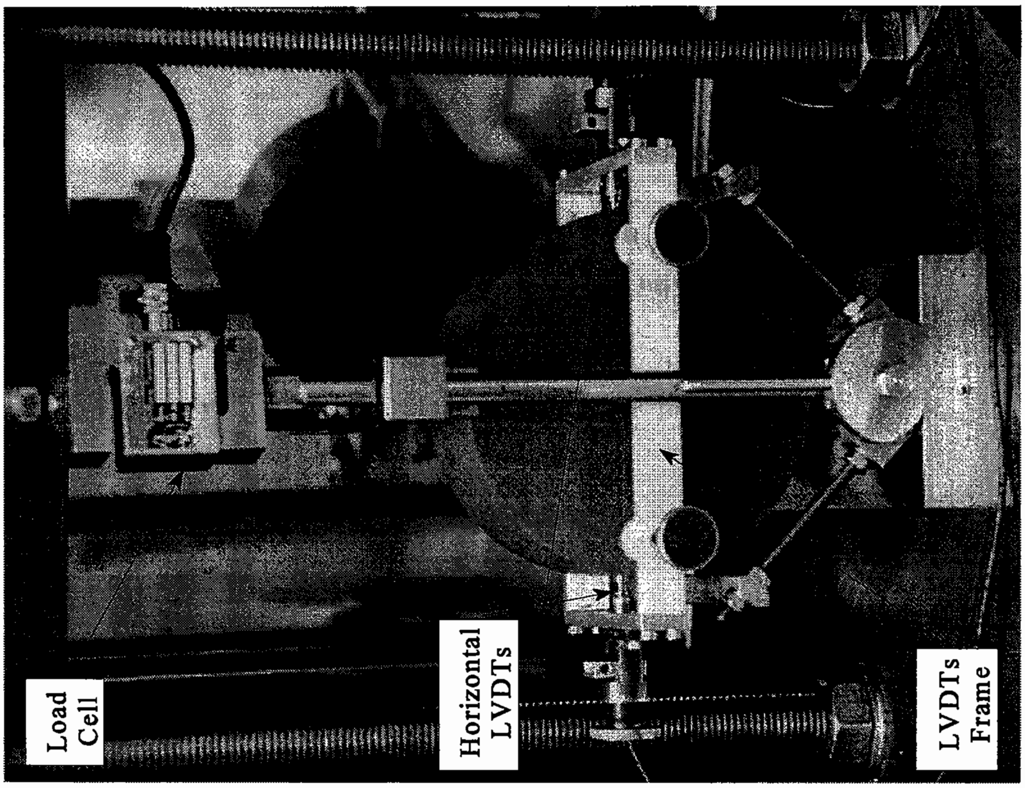

Figure 5.8 Detail of the arrangement for stiffness modulus tests.

5.4. The repeated load indirect tensile apparatus

5.4.1. Introduction

With the development and continuous improvement of the NAT equipment and its growing widespread use for the characterisation of bituminous materials, the indirect tensile test is becoming more popular for material evaluation. There are three main tests, relevant for the assessment of bound materials, that can be performed using indirect tensile equipment: strength test, stiffness modulus test and fatigue test, all in indirect tensile mode.

The adaptation of the NAT for the study of stabilised materials is seen as an attractive proposal resulting in some advantages for the characterisation of pavement materials:

- advance in relation to the present situation in which the main standardised test for stabilised materials is the compressive strength test.
- characterisation of materials using more performance-related tests (as tensile stresses may be developed);
- possibility of testing cores directly obtained from the pavement, using special techniques of coring and sawing;
- the use of the same equipment for the characterisation of both stabilised and bituminous materials, bringing economic advantages to testing laboratories;
- determination of stiffness, fatigue and strength in the same apparatus;
- uniformity in the testing of pavement materials;

5.4.2. Limitations of NAT for testing hydraulic bound materials

The conventional NAT equipment for bituminous materials has important limitations for testing stabilised materials that are outlined below.

Equipment (general)

- Low maximum loading capacity, up to 4.3 kN. Many hydraulically bound materials may be too strong for testing in such an apparatus.
- General drawbacks associated with pneumatic equipment, especially in terms of control of the load pulse.
- Limitations of the commercial software controlling the test which reduces the flexibility for research purposes.

Tensile strength

- Indirect tensile strength tests are not generally performed in the NAT.

Stiffness modulus

- Stiffness modulus tests are presently performed in deformation-controlled rather than load-controlled mode (which is more useful for hydraulic materials).

Fatigue properties

- Inadequate design details of the NAT sub-frame for fatigue testing.

Some problems related to the software controlling the tests in terms of deformation rather than load, and the control of the load by a closed-loop system converging slowly to the desired value in practice meant that specimens were initially overloaded.

A repeated load indirect tensile apparatus was developed answering to the limitations presented above and was used throughout this research study.

5.4.3. General description of the equipment

The repeated load indirect tensile apparatus was developed using, whenever possible, existing equipment within the Department of Civil Engineering of the University of Nottingham, in particular components of the triaxial system were used. The major components of modern laboratory testing systems are not specific for a particular type of test and so they may be modified into multi-purpose systems that may be used for the performance of different types of tests on different materials. Figure 5.9 presents a picture of the equipment. Special emphasis was given to the design of an indirect tensile sub-frame that could be used in different test rigs.

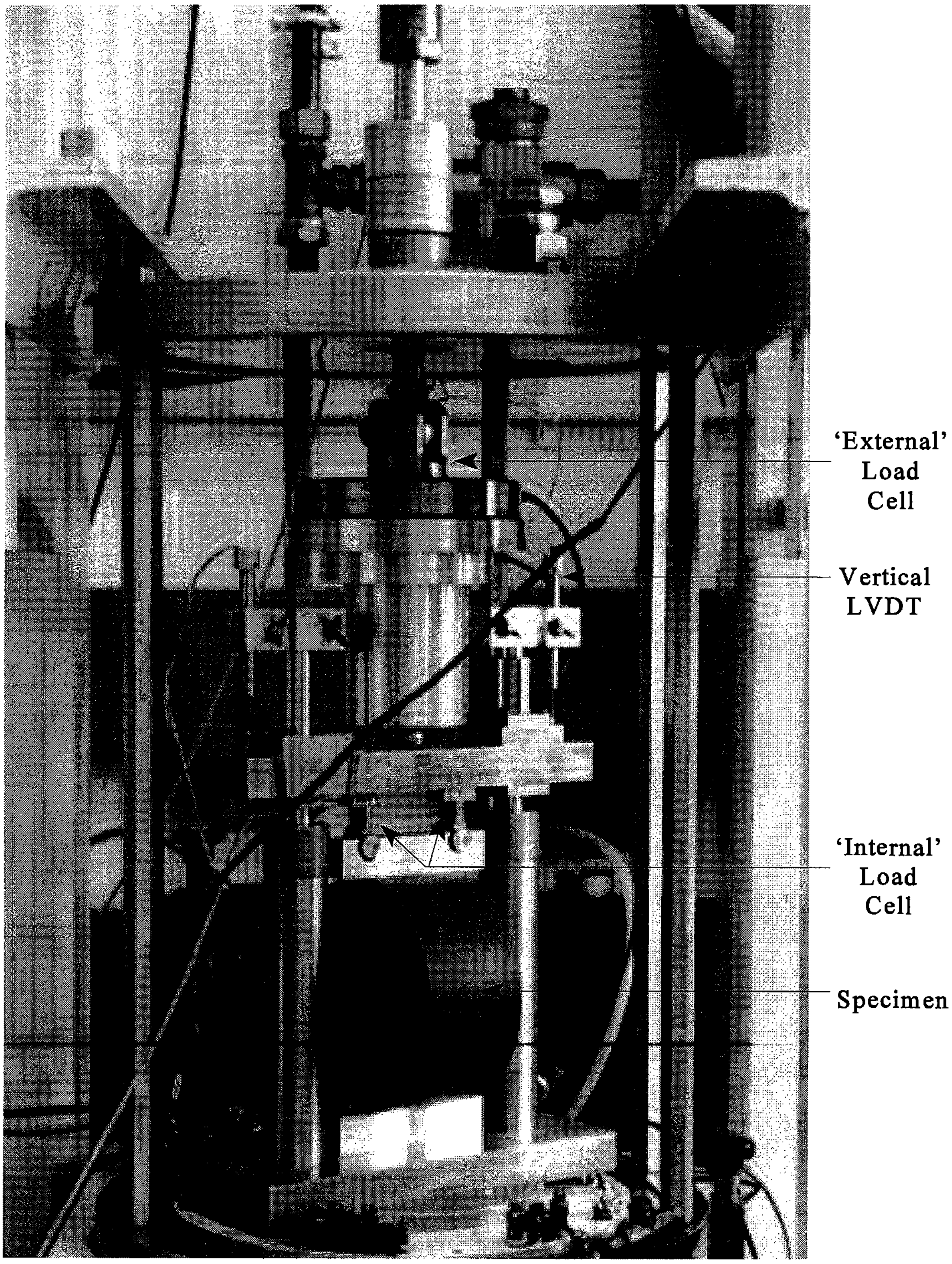

Figure 5.9 Repeated load indirect tensile apparatus.

In terms of loading and deformation measurement systems the equipment incorporates the following components:

- servo-hydraulic loading system capable of applying a maximum vertical load of 20 kN at frequencies up to 16 Hz;
- three pairs of LVDTs are used for the measurement of
  - vertical displacements of the loading cross-head. These are used, essentially, to stop the test after specimen failure;
  - horizontal deformation of the indirect tensile specimen during stiffness modulus determination;
  - vertical and horizontal deformation using miniaturised LVDTs in the central part of the specimens. This system is essentially used for Poisson ratio evaluation and is described in more detail in Chapter 7.

With regards to control, the components form a feedback closed-loop system as already described for the repeated load triaxial apparatus.

5.4.4. Control software modifications

Initial tests in the conventional NAT for stiffness modulus determination were conducted in deformation-controlled mode as this was the available version. This was found not to be a feasible method for hydraulic bound materials due to the wide variation of the values of stiffness modulus for different materials. In order to know in advance the level of deformation (or strain) that one should impose onto the specimen, safeguarding its integrity, it is necessary to know or estimate stiffness which is ultimately the object of the test. On the other hand, for load-controlled tests the stress levels to be used can easily be related to the monotonic tensile strength of the material. Taking into account these considerations the control software was modified to enable the performance of tests in load-controlled mode.

For research purposes more flexibility was needed, in particular for a better control of the load wave form and to increase the range of frequencies that could be used. Currently, the frequencies in the NAT may range from 60 to 160 ms. This led to the use of the servo-hydraulic system and the ATS software to control the repeated load triaxial test since it permitted the required flexibility.

5.4.5. Development of sub-frame for fatigue testing

In the initial stages of this research a fatigue frame similar to the one used for bituminous materials was considered [BS DD ABF, 1996; Read, 1996]. The original design (Figure 5.10) was maintained and the only modifications were in relation to the dimensions of the frame to incorporate the bigger 150 mm specimens used in this research.

From the initial tests it was evident that, for the stiffer hydraulic bound materials, small unevenness of the specimens was causing the loading head to rotate and transmit part of the load to the vertical guiding rods. Even when this was not the case in the beginning of the test, it would occur during the deformation process. Consequently, the vertical line loading transmitted to the specimen was significantly altered to a non-uniform load which in turn resulted in further aggravation of the problem.

To solve this issue it was necessary:

- to re-design the sub-frame to improve its rigidity and extend the linear bearings to reduce the possibility of rotation of the cross-head. The developed sub-frame is shown in Figure 5.11;
- to develop a new load cell adequately positioned in the testing apparatus for an accurate measurement of the load actually transmitted to the specimen. This is presented in the next section.

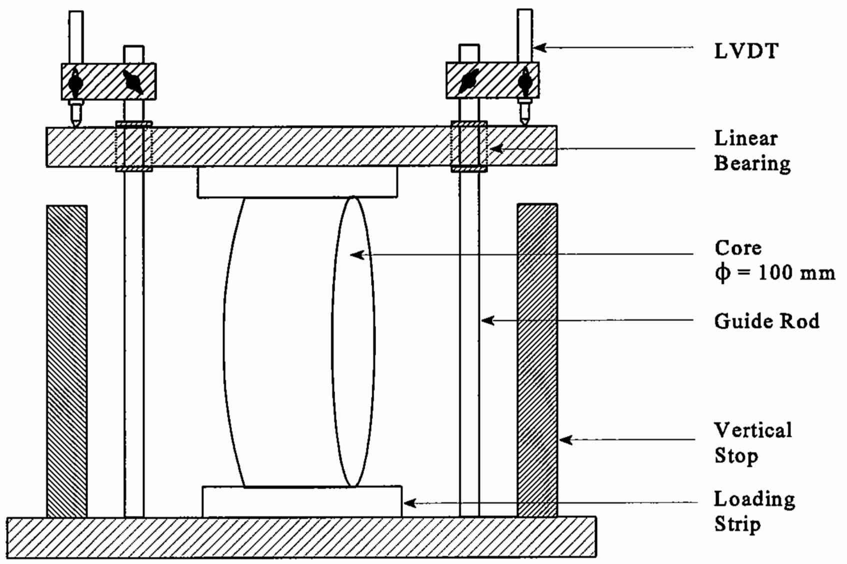

Figure 5.10 Sub-frame for fatigue testing of bituminous materials [Read, 1996].

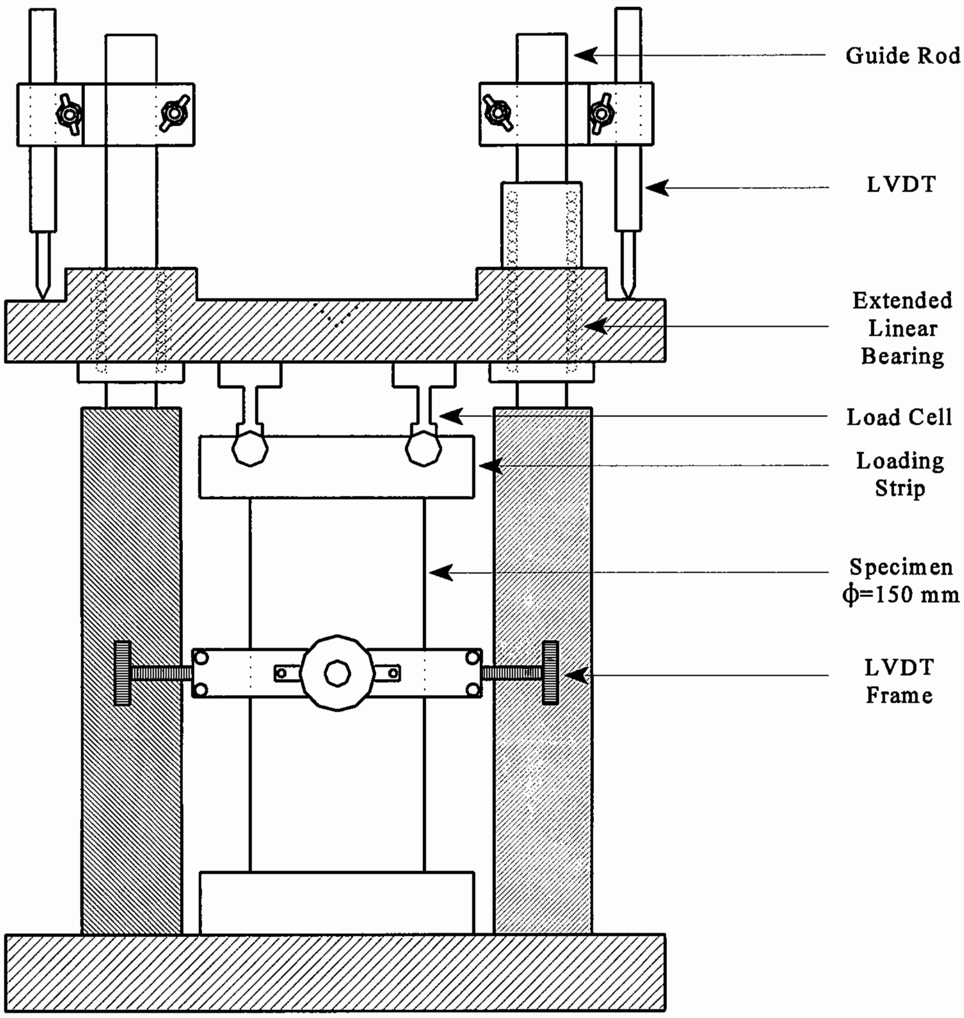

Figure 5.11 New sub-frame of the repeated load indirect tensile apparatus.

5.4.6. Development of 'internal' load cell

The location chosen for the load cell was below the cross-head to account for any reduction of the load transmitted to the specimen. This is likely to result from the friction in the linear bearing, an effect that is aggravated with deviations of the loading head from the strictly horizontal position. The developed load cell is shown in Figure 5.11.

To verify the adequacy of the new load cell a study was conducted using two plastic specimens which were loaded using repeated loading in circumstances identical to testing conditions. The load was monitored using both load cells, the existing one located externally in relation to the testing sub-frame and the new load cell. The first specimen used was an irregular specimen in which a non-parallelism of the surfaces in contact with the loading strips was artificially created with the objective of simulating an uneven specimen to the extreme. The second was a regular specimen perfectly machined to assure a completely uniform contact with the loading strips. Both specimens had the dimensions of 150 mm diameter by 70 mm thickness generally used throughout this research. The results are shown in Figures 5.12 and 5.13 respectively.

There was an important reduction in the vertical load applied to the irregular specimen and an excellent agreement of the readings from both load cells when a perfectly regular and homogeneous specimen was tested.

Those situations correspond to both extremes in relation to what would happen in practice. In a current test the load transferred to the frame would fall somewhere in the middle depending on the regularity of the specimen and its homogeneity. This is illustrated in Figure 5.14 where the load was monitored during an indirect tensile fatigue test performed on stabilised fly ash.

The results of this study seem to confirm an important qualitative improvement of the testing sub-frame, enabling the load to be more accurately and consistently applied and measured. This will also enable the use of a unique specimen frame for all tests performed in the repeated load indirect tensile apparatus with evident advantages for the performance of these tests on a routine basis.

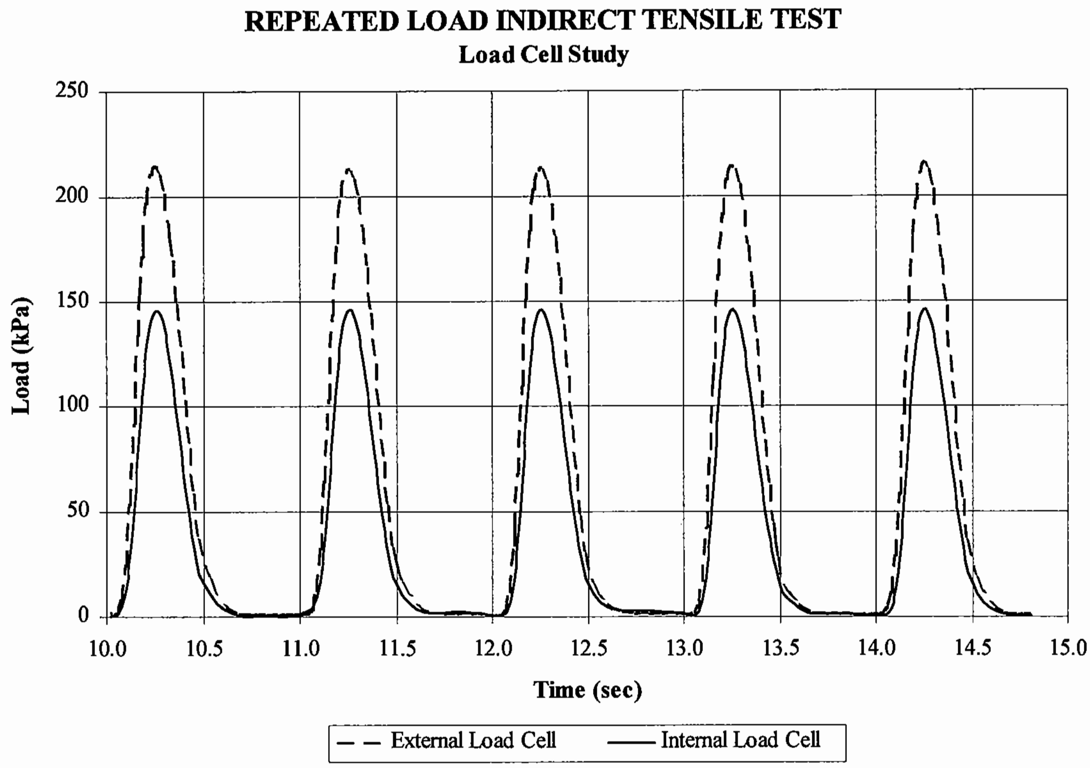

Figure 5.12 Comparison of load cells using irregular plastic specimen.

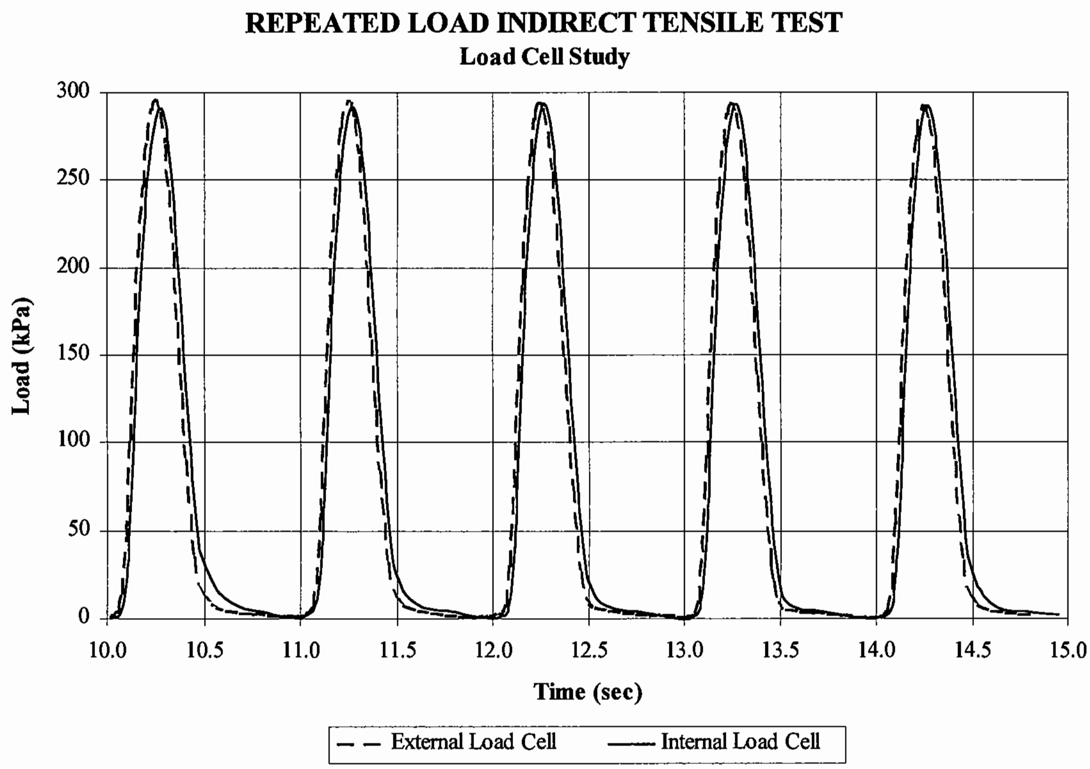

Figure 5.13 Comparison of load cells using regular plastic specimen.

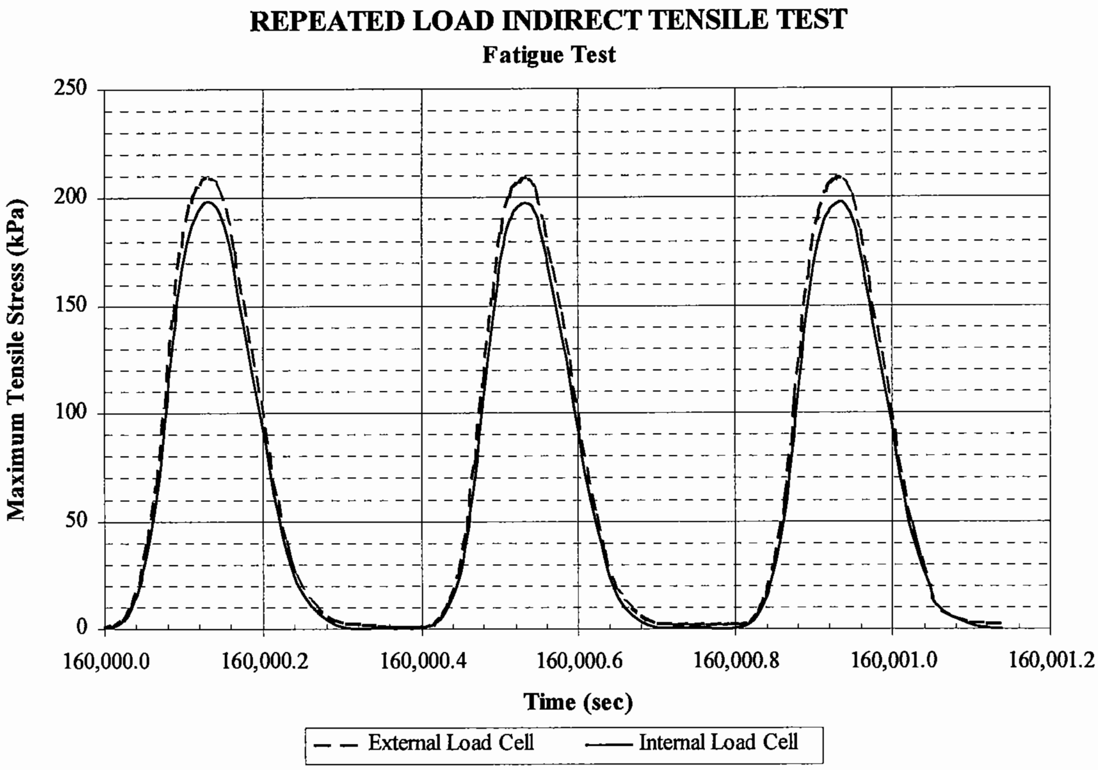

Figure 5.14 Comparison of load cells during a fatigue test (fly ash specimen).

5.4.7. Development of an alternative LVDT frame

The LVDTs frame currently used for measuring the horizontal deformation of the specimens is described in the Draft Standard BS DD 213 [1993] for the determination of the indirect tensile stiffness modulus. This aluminium frame is supported directly on-sample by four points on the flat surfaces along the horizontal diameter which may restrain the free deformation of the specimen and have repercussions in the stress state induced.

Since the indirect tensile specimen deforms outwards along that diameter, the centre being a neutral point in the deformation process, the use of a LVDT frame supported by single 'points' at the centre of each side of the specimen would lead, in principle, to a more accurate measurement of the deformations. This arrangement was made with the areas of contact with the specimen coinciding with its centre and limited to a circle of 17 mm in diameter to reduce the effects on stresses and strains. The alternative frame is shown in Figure 5.15.

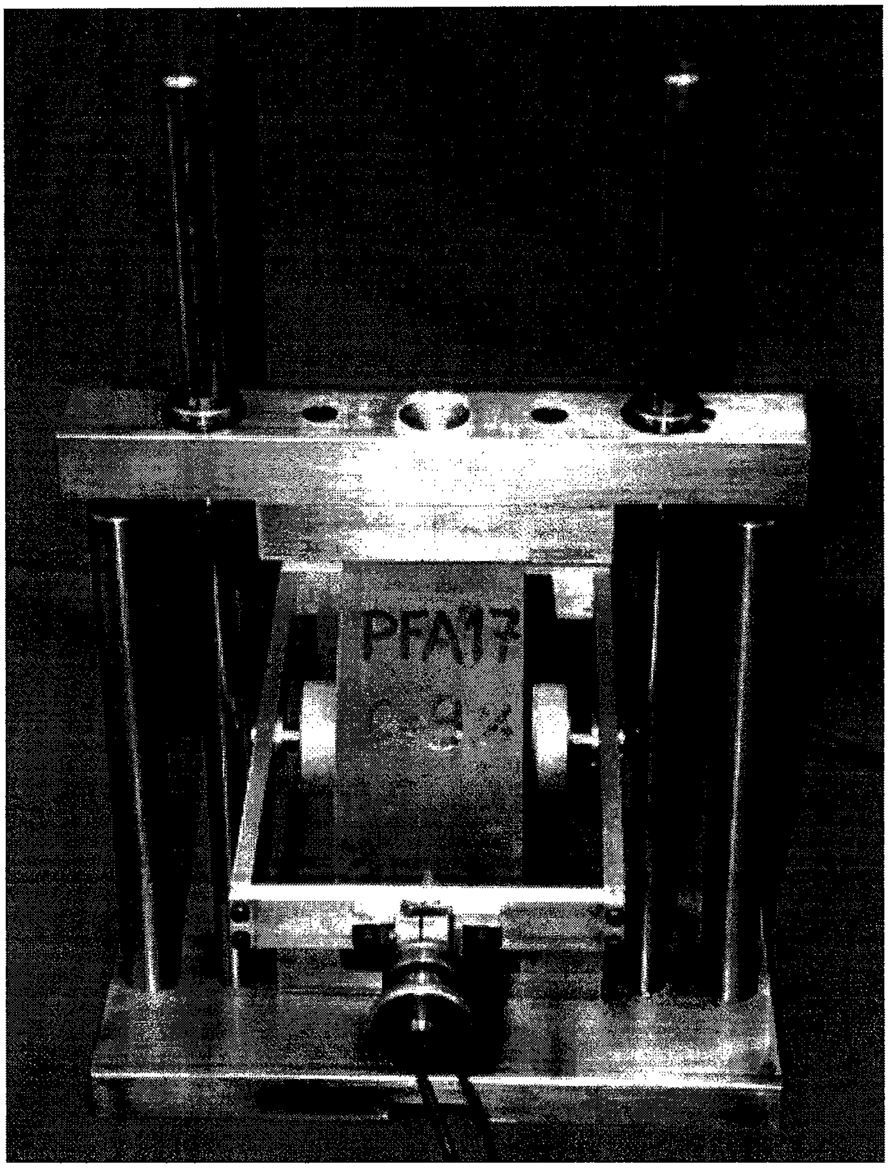

Figure 5.15 New LVDT frame.

Comparative studies were made testing a particular specimen with both LVDT frames and in identical loading conditions. These studies involved the artificial plastic specimen and stabilised fly ash with the two extreme binder contents considered in this research: 2 and 9 % of cement. The results are presented in Figures 5.16 and 5.17 and in Table 5.1. In the case of fly ash stabilised with 9 % of cement, a first test for stiffness modulus determination was carried out using the old LVDT frame. Then the test was repeated using the new frame after which a second test was conducted again with the old frame.

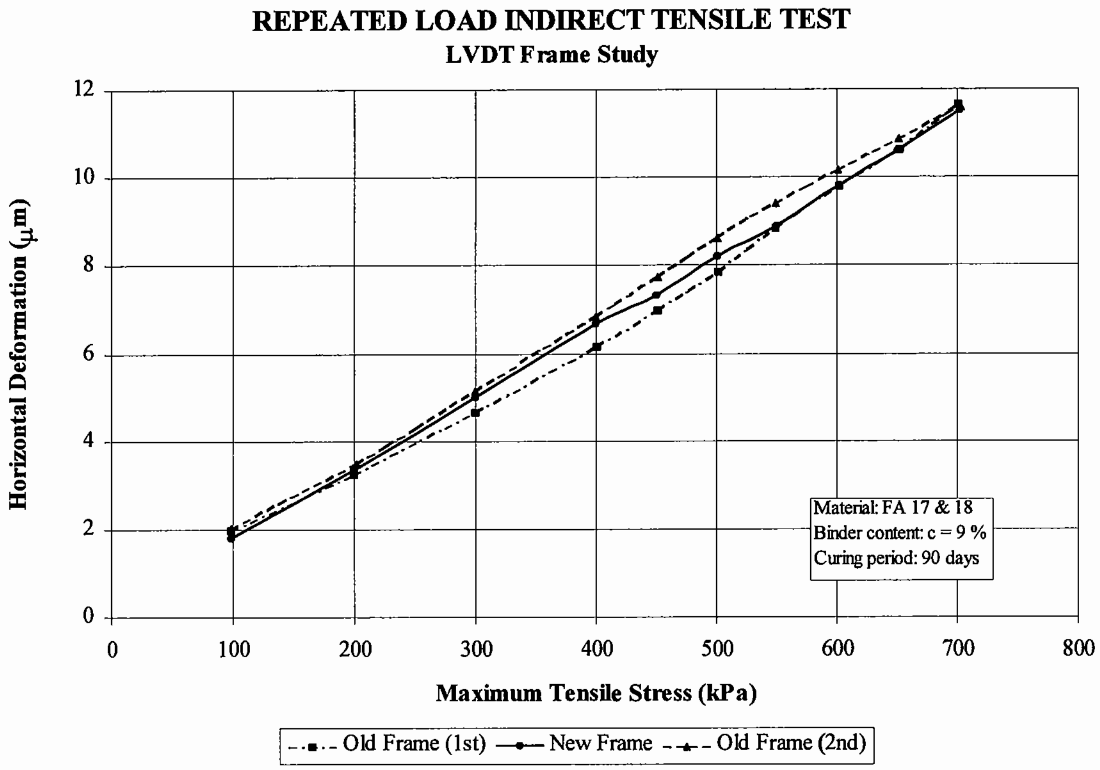

Figure 5.16 Comparison of LVDT frames (horizontal deformation).

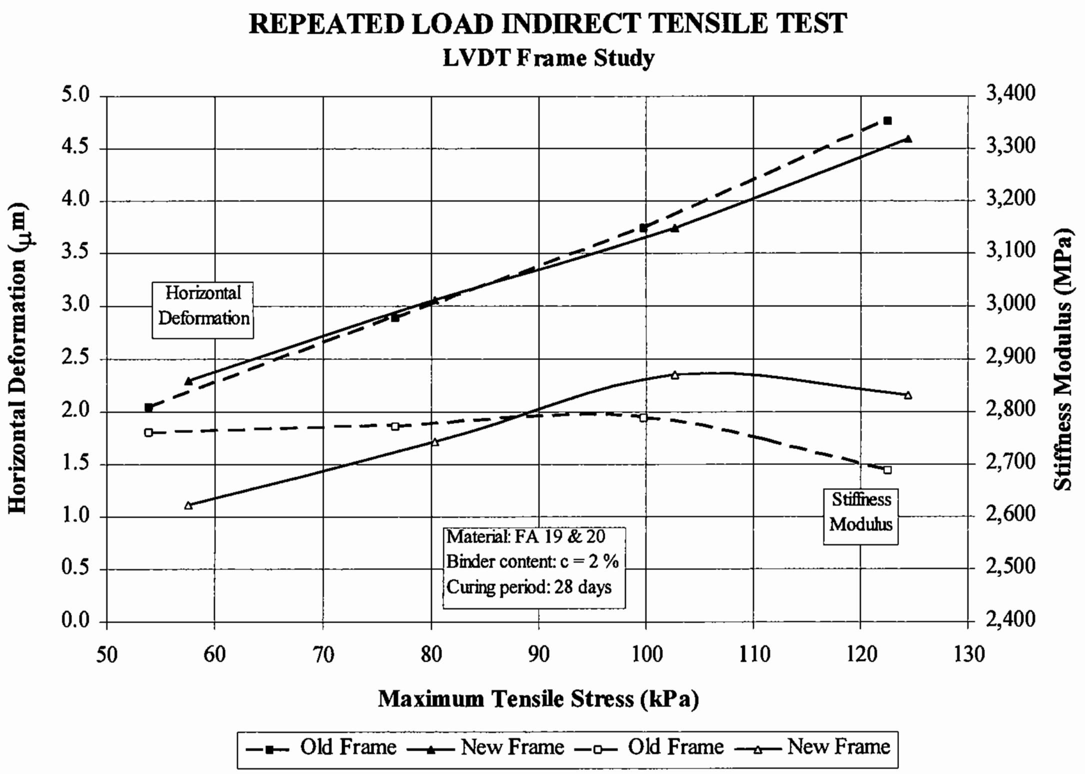

Figure 5.17 Comparison of LVDT frames (horizontal deformation and stiffness modulus).

Table 5.1 Comparison of LVDT frames.

Frame		Current [BS DD 213, 1993]			New		
		Sm (MPa)	SD (MPa)	Cv (%)	Sm (MPa)	SD (MPa)	Cv (%)
Plastic		3519	45	1.3	3461	116	3.3
FA c=2% @ 28 d		2753	38	1.4	2767	95	3.4
FA c=9% @ 90 d	1st	6289	167	2.7	6087	86	1.4
	2nd	5895	80	1.4			

Key: Sm Stiffness modulus — FA Pulverised fuel ash
SD Standard deviation — c cement content
Cv Coefficient of variation — d days (curing period)

When a second test was conducted with the old frame, Figure 5.16 and Table 5.1, the measurement with the new frame was between the other two and although some internal damage may be responsible for the small reduction in stiffness (increase in deformation), overall the results were very similar.

In Figure 5.17 the effect of the differences in horizontal deformations on the stiffness moduli are also plotted. They confirm previous observations and in average terms the results are of 2753 and 2767 MPa respectively for the old and new frames (Table 5.1). In practice, those values can both be rounded to 2.8 GPa. The test performed on the plastic specimen also gave identical results.

The results show that the measurements from both frames are similar, producing sensibly identical stiffness values, the differences observed being in accordance with the variability obtained in the repeatability study presented later in this chapter. This may be explained by the small areas of contact between the LVDT frame and the specimen and its very light weight, meaning in practice that very small pressures are necessary to support the frame and LVDTs.

5.4.8. Replication of NAT load pulse

In the first instance, the load pulse applied by the pneumatic NAT was replicated in the repeated load indirect tensile apparatus to compare the results obtained from both equipments when testing a plastic specimen. A typical load pulse obtained in the NAT is shown in Figure 5.18. The servo-hydraulic equipment is capable of closely replicating this load pulse enabling a superior control of the load wave as shown in Figure 5.19, where a risetime of 125 ms and a rest period of 750 ms were considered. From the comparison of both load pulses it is evident that both loading and unloading times (and consequently the frequency of loading) can be better controlled in the indirect tensile apparatus. Nevertheless, the stiffnesses obtained from both equipments were identical and similar to the ones already presented in Table 5.1.

The response to the load pulse illustrated in Figure 5.19 for a stabilised fly ash specimen is shown in Figures 5.20 and 5.21, for both the measured vertical and horizontal deformations.

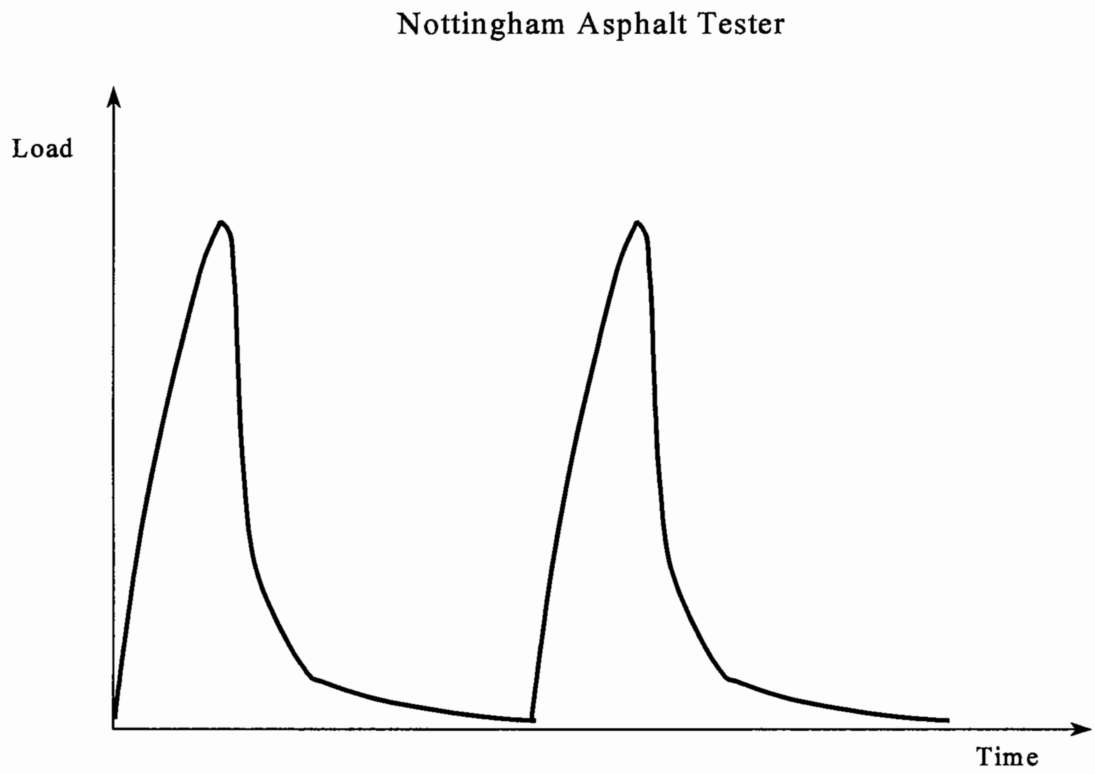

Figure 5.18 Typical load pulse in the Nottingham Asphalt Tester.

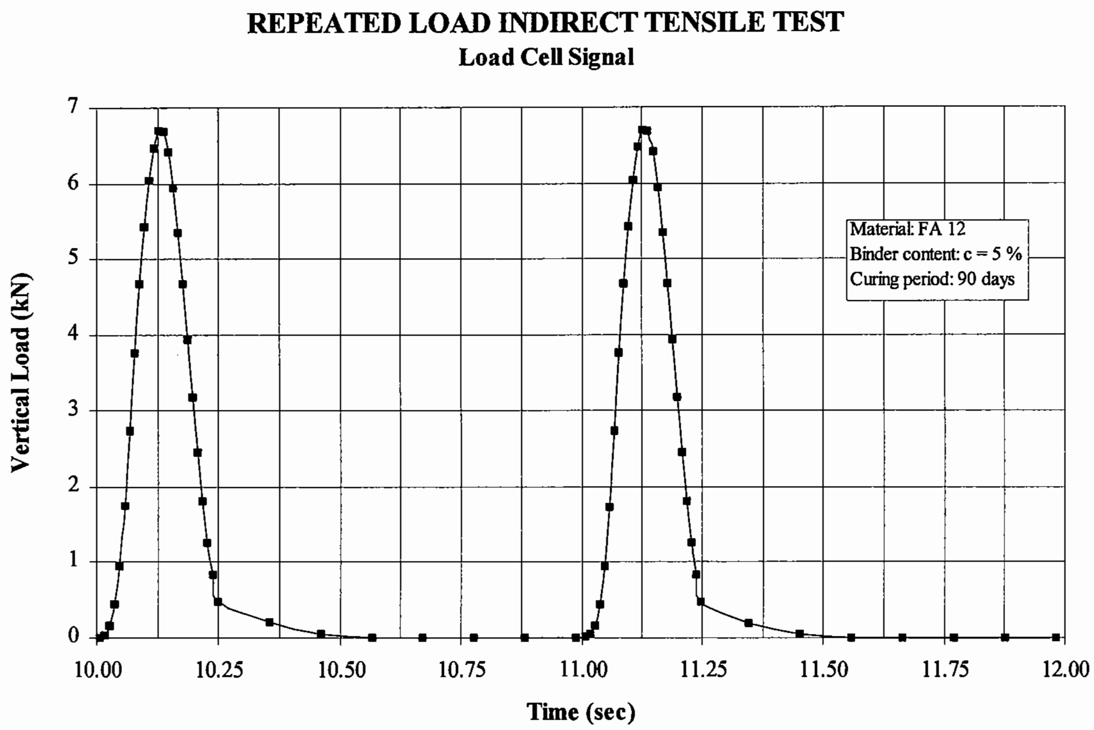

Figure 5.19 Load signal in the indirect tensile apparatus.

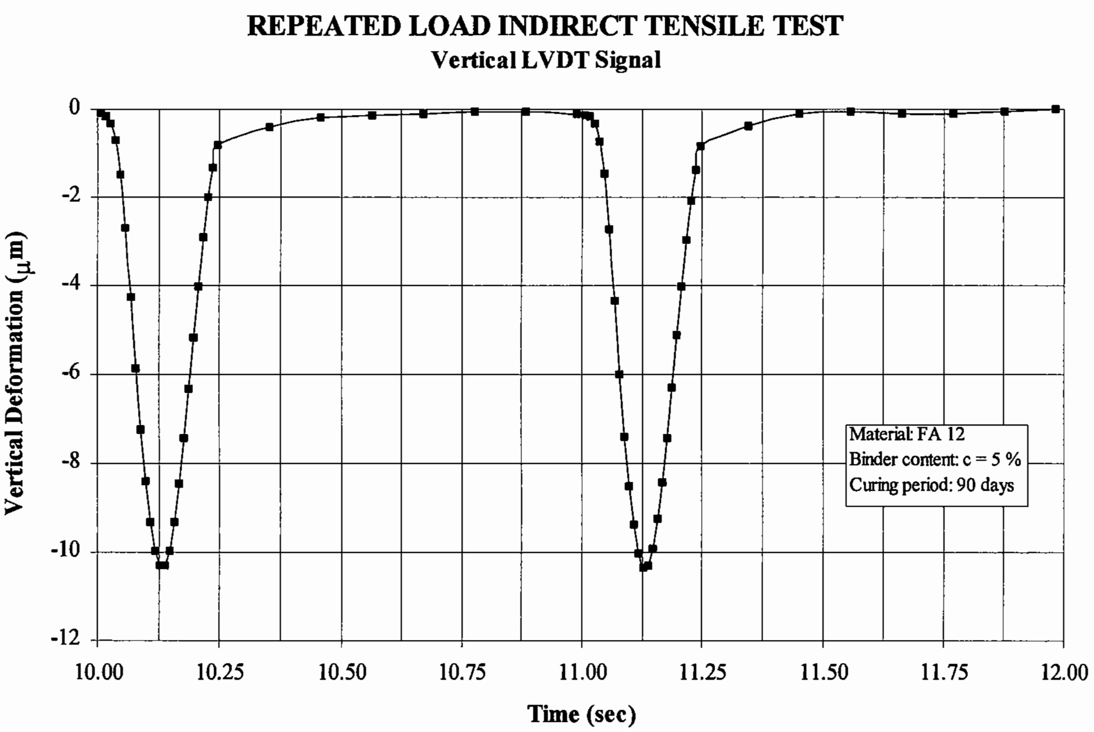

Figure 5.20 Material response - measured vertical deformation.

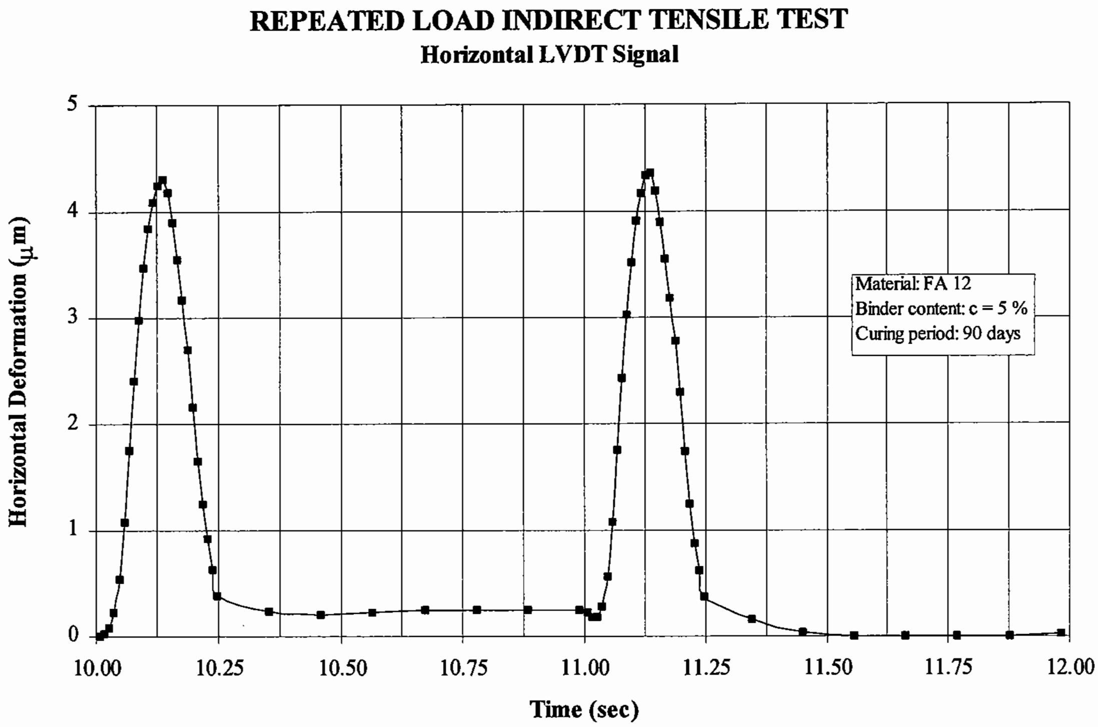

Figure 5.21 Material response - measured horizontal deformation.

5.4.9. Repeatability study

A study was carried out on the repeatability of the equipment / technique for the determination of stiffness modulus. Repeatability is defined as the value below which the absolute difference between two single test results is not statistically significant at the 95 % confidence level. For repeatability studies the tests should be performed in the same conditions of test specimen, operator, method, equipment, laboratory and should be carried out in a short interval of time. For that confidence level the repeatability (r) is given by [Dawson, 1996]:

$$r = 2.8 \times SD \qquad 5.1$$

where SD is the standard deviation.

A total of 25 tests were performed on the same 100 mm specimen of fly ash stabilised with 5 % cement and the results are listed in Table 5.2. The values obtained for the coefficient of variation and repeatability seem to be acceptable and compatible with the ones obtained for other laboratory testing techniques [Sherwood & Pike, 1984; CRR, 1988; Nunes, 1991].

The average obtained in this study (4664 MPa) compares with the stiffness of 4277 MPa obtained on 150 mm specimens (reported in Chapter 7). Although this result seems to indicate an influence of the diameter of the specimen on the stiffness value, it is not possible to draw a definite conclusion due to the limited data available. For bituminous material 100 mm specimens, with thicknesses as low as 40 mm , are currently used for stiffness and fatigue assessment.

Table 5.2 Results of repeatability study on stabilised fly ash (c = 5 % @ 28 days).

Vertical Force kN	**Tensile Stress kPa**	**Horizontal Deformation µm**	**Risetime ms**	**Stiffness Modulus MPa**	**Summary**		
2.3	365.4	5.0	125	4739	Stiffness	mean	4664
2.3	361.8	5.0	125	4639	modulus	min	4379
2.3	362.3	5.1	125	4627	(MPa)	max	4804
2.3	367.4	5.1	125	4681	Standard	(MPa)	107
2.3	363.7	5.0	125	4705	deviation		
2.3	370.4	5.1	125	4718			
2.3	360.5	5.0	125	4654	Coefficient	(%)	2.3
2.3	361.1	5.0	125	4685	of variation		
2.3	358.7	4.9	125	4734			
2.3	365.5	5.0	125	4693	Repeatability	(MPa)	300
2.3	366.6	5.0	125	4777		(%)	6.4
2.3	365.4	5.0	125	4747			
2.3	364.5	5.0	125	4707			
2.3	366.7	5.1	125	4678			
2.3	370.3	5.0	125	4784			
2.3	370.1	5.0	125	4804			
2.4	386.7	5.2	125	4785			
2.2	354.0	4.9	125	4638			
2.4	376.4	5.3	125	4637			
2.2	349.9	5.1	125	4460			
2.2	344.4	5.1	125	4395			
2.1	339.7	5.0	125	4379			
2.3	363.1	5.1	125	4638			
2.3	370.2	5.1	125	4668			
2.3	370.6	5.2	125	4631			

5.5. Summary

- The repeated load triaxial apparatus was briefly described and the improvements made during this research identified. These were, in particular, the modification of the location studs and the development of a new load cell with the capacity to perform small rotations to apply more uniform axial stresses and to reduce the susceptibility to electrical noise.

- The Nottingham Asphalt Tester used for the assessment of bituminous materials was succinctly presented and its limitations for testing hydraulic bound materials explained.

- The necessary modifications of the NAT resulted in the development of a repeated load indirect tensile apparatus based on existing laboratory components of the triaxial rig and the design of a testing sub-frame that may be incorporated in other testing machines. This frame was further refined through the development of an 'internal' load cell positioned below the cross-head accounting for any reduction in the magnitude of the load due to friction. The frame enables a better control and measurement of the load applied and may be used for all tests performed in the indirect tensile apparatus.

- An alternative LVDT frame was designed supported in single 'points' at the centre of the specimen. However, the results obtained were identical to the ones obtained with the current frame not demonstrating superior performance for measuring the horizontal deformations.

- The repeatability of the indirect tensile apparatus and the technique used for the determination of stiffness modulus was assessed. Overall, the coefficients of variation and repeatability obtained seem to be reasonable and comparable to those obtained in other laboratory testing techniques.

CHAPTER 6

REPEATED LOAD TRIAXIAL TESTS

6.1. Introduction

Aggregates for use in pavement foundations must be able to spread the load, due to traffic, which comes down through the pavement structure. By so doing they prevent overstressing of the underlying soil (and thus premature failure of the pavement due to rutting) and prevent fatigue of any bound layers due to excessive resilient movement. The important property of the aggregate which controls load spreading is the resilient modulus (M_r). In certain cases the Poisson's ratio (ν) and angle of shearing resistance (ϕ) can also be useful as they permit aggregate confinement and ultimate load conditions to be more accurately assessed. In the analytical modelling of pavement performance all these are required. Another important characteristic for pavement materials is their resistance to permanent deformation to prevent rutting. Both resilient modulus and permanent deformation properties can be determined by performing repeated load triaxial tests and strength can be determined by monotonic loading in the same apparatus.

In previous chapters the importance of performance tests for the assessment of materials' properties was stressed. The repeated load triaxial test is able to act as a reasonably good performance test as pavement materials can be tested in the laboratory, at the correct stress levels, at full grading and subjected to loading which simulates, in a broadly satisfactory way, the complex loading system due to traffic.

6.2. General aspects of the behaviour of granular materials under repeated loading

The stress state of an element of soil or granular material which is under bound layers of the pavement structure is presented schematically in Figure 6.1, and includes vertical (σ_V), horizontal (σ_H) and shear stresses (τ_{VH}, τ_{HV}). From this rather complex system of loading result some of the limitations of the repeated load triaxial test [Nunes, 1991]:

- impossibility of simulating the rotation of the principal stress directions occurring in the pavement under a moving wheel (Figure 6.2) by applying simultaneously and directly to the specimen, normal and shear stresses. This test reproduces the repeated normal stresses, Figure 6.3, but is unable to apply reversible shear stresses and, therefore, only when the element of material *in situ* is underneath the load, does the stress state correspond to the one of the triaxial test;
- inability to apply tensile stresses which occur in the base of treated or bound layers, simultaneously with the other stresses;
- inadequate evaluation of permanent deformations;
- simplification of the tridimensional stress state existing in the field to axisymetric conditions.

It should be emphasised that shear stresses are more significant in those layers in the vicinity of the road surface, where bound materials are used, and for which the repeated load triaxial test has little or no meaning, since they form 'slabs' which behave very differently from unbound materials.

In addition, the ability to apply tensile stresses is clearly not important when studying unbound materials, since these materials will not develop tensile stresses due to their unbound nature. However, tensile stresses are increasingly important when studying lightly and heavily-treated materials.

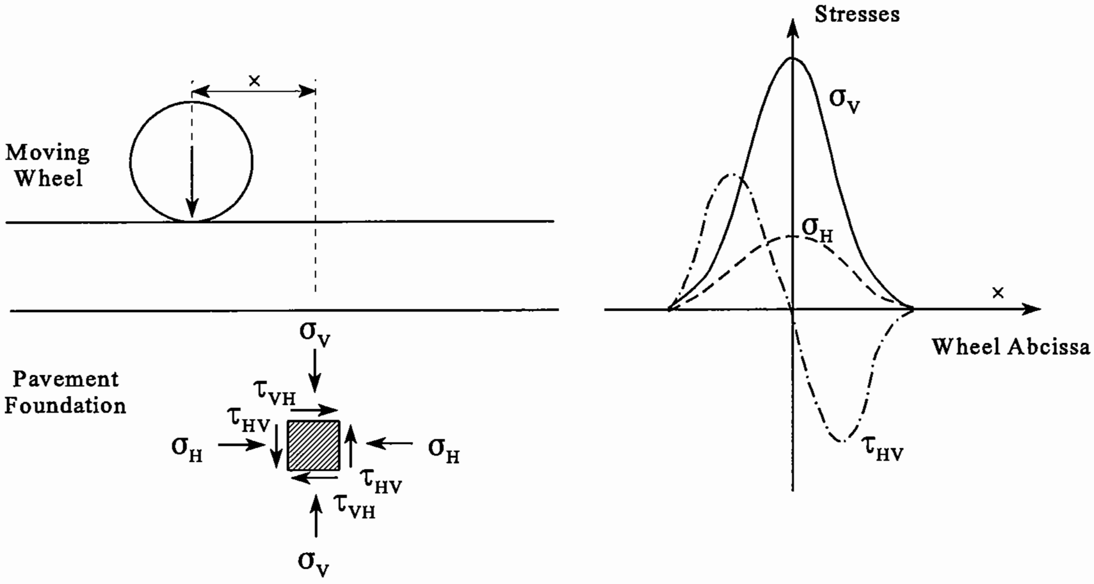

Figure 6.1 Loading system in pavement structures under a moving wheel.

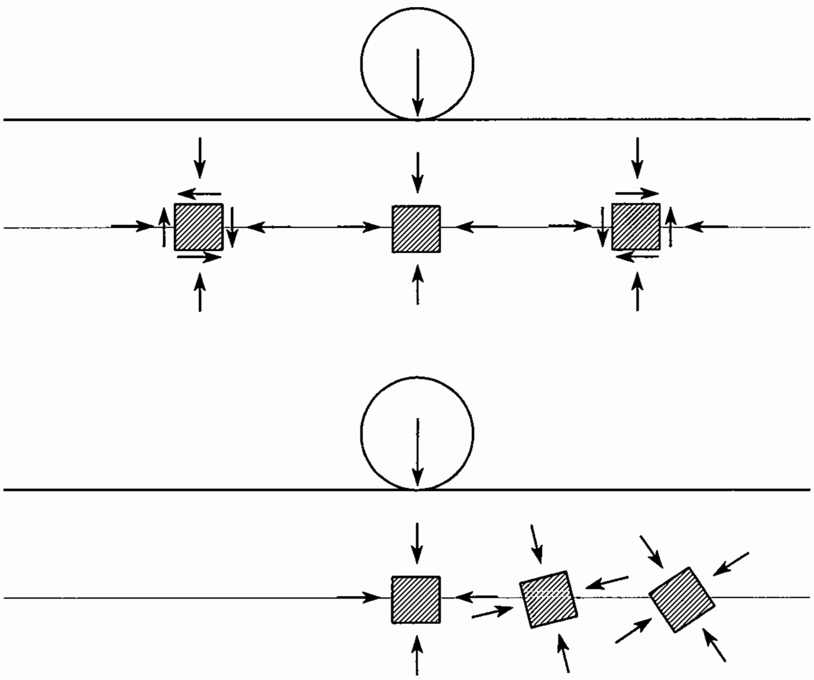

Figure 6.2 Rotation of principal stress directions in pavement structures.

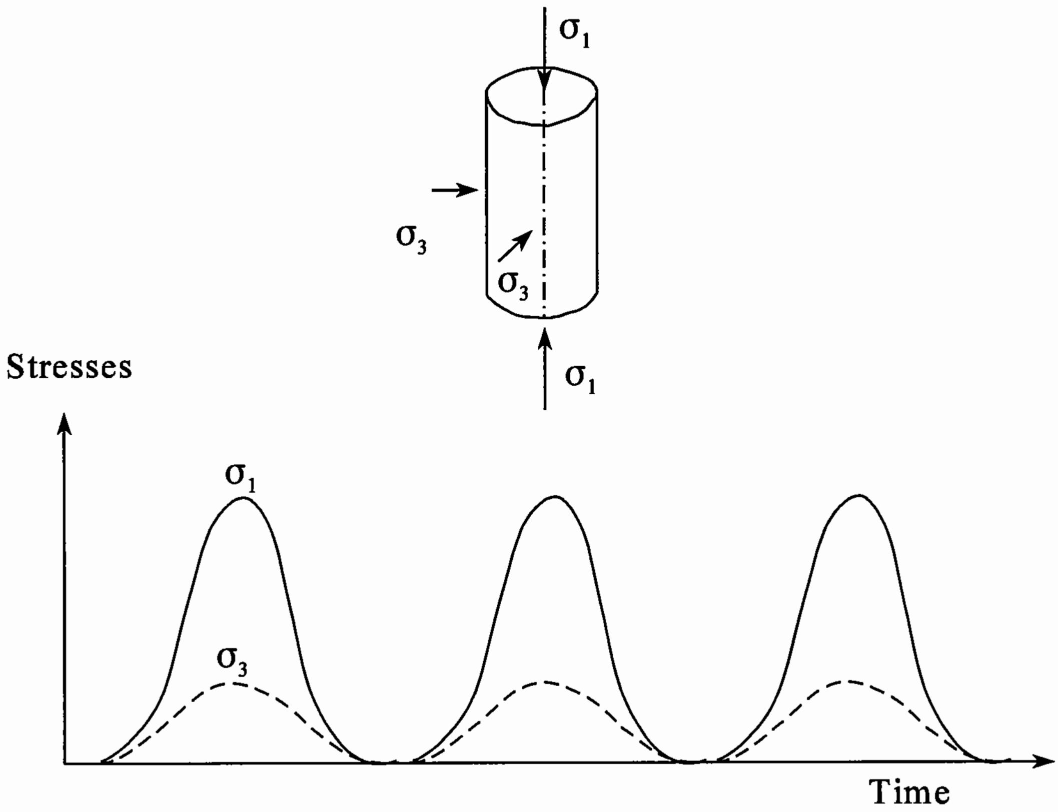

Figure 6.3 Loading system in repeated load triaxial tests.

With regards to permanent deformation, there are several factors responsible for the difficulty in assessing permanent deformation behaviour accurately in the triaxial apparatus, namely:

- measurement of permanent deformations in the same direction of the applied principal stresses throughout the test. Owing to the rotation of the principal stress directions in the field this should contribute to an overestimation of the deformations in the triaxial test;
- lateral expansion of the specimen, with similar implications on the deformations. *In situ* the lateral confinement of adjacent material is likely to reduce this effect;
- effect of repeated shear stresses on the development of permanent deformations. Those stresses can be applied in the hollow cylinder apparatus and are responsible for the overall underestimation of permanent deformations in triaxial testing [Chan, 1990].

By comparing test results carried out in a hollow cylinder apparatus and in the repeated load triaxial rig, it is possible to conclude [Chan, 1990] that the permanent strain behaviour of granular materials is affected by the application of reversed shear stresses. This influence on the rate of development of permanent deformations became more significant as stress conditions approached failure, while, for low stress conditions and when the ratio of the torsional shear to vertical stress is reduced, the triaxial results were found to be sufficiently accurate. As far as resilient behaviour is concerned, the applications of reversed shear stresses (either uni- or bi-directional) did not result in any significant difference between the two equipments.

In summary, it is possible to say that the repeated load triaxial test still gives a good indication of the rutting susceptibility of the material. It is perfectly acceptable for material discrimination as will be demonstrated in this chapter and has a particular advantage over the hollow cylinder apparatus (mentioned above) in the sense that most pavement materials may be tested at their full grading. It has been concluded [Hight, Gens & Symes, 1983] that for a hollow cylinder apparatus with a 25.4 mm wall thickness the maximum particle size should be approximately 1 mm. This is the present practice for testing in the Nottingham hollow cylinder apparatus [Richardson, 1994], and represents a serious limitation of this equipment for testing the type of materials generally used in road construction.

Hence, despite some limitations, the repeated load triaxial test is a reasonable reliable test for determining the mechanical properties used in analytical modelling of pavement performance. It represents a good compromise in complexity, simulation of field performance and reliability of results. It is, for this reason, a good candidate for consideration in new performance-based specifications for:

- the determination of mechanical properties to be incorporated in analytical methods of pavement design;
- the understanding and modelling materials behaviour;
- materials discrimination and ranking.

A typical curve representative of unbound granular materials' behaviour in a conventional triaxial test is presented in Figure 6.4.i and their behaviour under repeated loading is reproduced in Figure 6.4.ii. From these figures, some typical properties can be pointed out which will be discussed below.

Unbound granular materials exhibit **non-linear behaviour** under repeated loading. The deformations of these materials result from different mechanisms due to the interaction of individual particles forming the bulk material:

- elastic deformation of the particles forming the aggregate at their contact points;
- attrition and crushing at the particle contact points;
- particle movements (sliding and rotation) eventually resulting in the reorganisation of the structure of particles.

The last two points are responsible for the permanent deformations and can help to explain the non-linear characteristics.

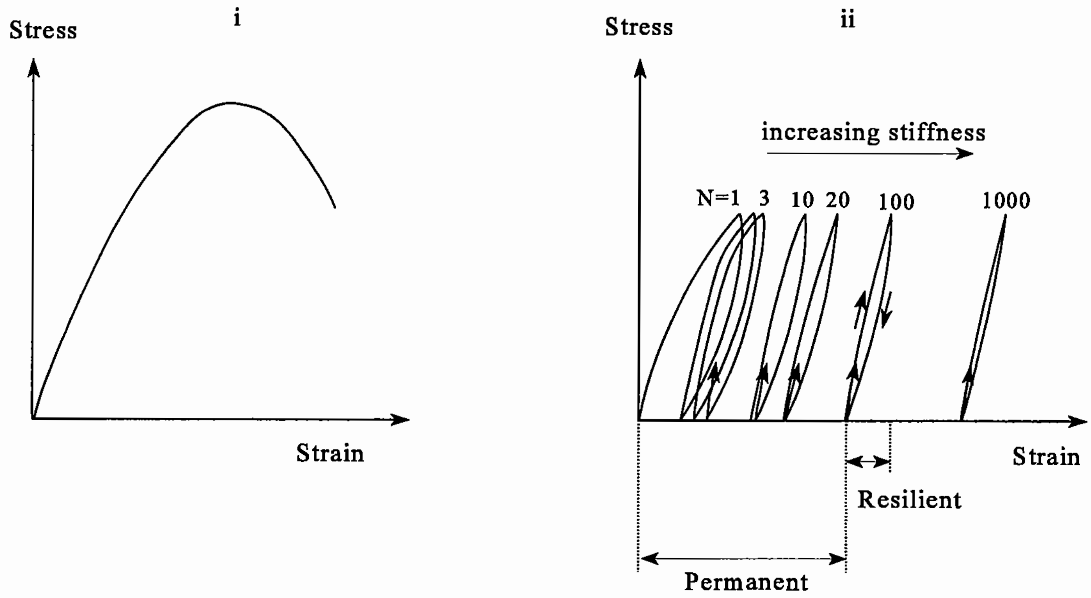

Figure 6.4 Typical stress / strain behaviour of granular materials: i) conventional triaxial test; ii) repeated load triaxial test.

Another typical property is the **inelastic behaviour**, since load-displacement curves do not coincide with the unload-displacement ones (hysteresis of the material). The area between those curves represents the work dissipated during one loading cycle.

The term resilient is used in this circumstance when the stress/strain behaviour of the material is represented by a closed loop in opposition to the term elastic, used when the loading and unloading lines coincide. In this way, the concept of resilient modulus replaces that of elastic or Young's modulus. Owing to the non-linearity of granular materials, a given resilient modulus is only valid for a specific stress condition which emphasises another important property of this type of material, its **stress-dependent behaviour**.

The initial part of the stress/strain curve, Figure 6.4.ii, agrees with the one obtained in conventional triaxial tests (Figure 6.4.i). However, stresses are unloaded at certain limits, distant from failure, and part of the strains which have already developed is recovered. After about 50 to 100 cycles, strains become almost entirely recoverable [Hicks & Monismith, 1971; Sweere, 1990] and are called resilient strains. The non-recovered strains are known as permanent strains (also called plastic or irrecoverable strains).

Resilient axial and radial strains ($\epsilon_{1r\ (N)}$ and $\epsilon_{3r\ (N)}$) are defined by the following expressions:

$$\epsilon_{1r\ (N)} = \frac{\delta_{1r\ (N)}}{H_0 - \delta_{1p\ (N-1)}} \qquad 6.1$$

$$\epsilon_{3r\ (N)} = \frac{\delta_{3r\ (N)}}{R_0 - \delta_{3p\ (N-1)}} \qquad 6.2$$

where: $\delta_{1r\ (N)}$ resilient axial deformation at the Nth cycle;

H_0 vertical distance between deformation measuring points on the triaxial specimen;

$\delta_{1p\ (N-1)}$ permanent axial deformation after cycle (N-1);

$\delta_{3r\ (N)}$ resilient radial deformation at the Nth cycle;

R_0 horizontal distance between deformation measuring points on the triaxial specimen;

$\delta_{3p\,(N-1)}$ permanent radial deformation after cycle (N-1).

Permanent axial and radial strains at cycle N ($\epsilon_{1p\,(N)}$ and $\epsilon_{3p\,(N)}$) are defined as follows:

$$\epsilon_{1p\ (N)} = \frac{\delta_{1p\ (N)}}{H_0} \qquad 6.3$$

$$\epsilon_{3p\ (N)} = \frac{\delta_{3p\ (N)}}{R_0} \qquad 6.4$$

where: $\delta_{1p\,(N)}$ permanent axial deformation at the Nth cycle;

$\delta_{3p\,(N)}$ permanent radial deformation at the Nth cycle.

The **development of permanent deformation** with the number of cycles is evident from Figure 6.4.ii, the rate of accumulation normally decreasing with the number of load applications. Load magnitude also affects permanent deformations in the following way [Thom & Dawson, 1996]:

- for stress paths of equal length, higher maximum deviatoric stresses produce faster straining; and
- longer stress paths result in higher permanent strains.

6.3. Preliminary testing on slate waste

A series of preliminary repeated load triaxial tests were performed on specimens of unbound slate waste 300 mm high by 150 mm diameter. The aim was to gain an introductory insight into the complexity of the equipment and an initial assessment of a secondary material's behaviour under repeated loading. The testing conditions in this preliminary phase were not the same as adopted for materials at a later phase. The aim was to assess the effect of different conditions, in particular the degree of compaction / maximum dry density, on the mechanical properties of the material.

The samples of slate waste were compacted in a mould, using a vibrating table and applying surcharge, at its natural moisture content (3.0 %), under light compaction effort achieving a dry density of 1.88 Mg/m^3. This density is clearly below the optimum values obtained for this material in the compactibility test (Chapter 3) and, therefore, is under the normal values that can be achieved in the field. This result was taken into account for the later testing programme, in which a different method of compaction was developed to control the final densities more accurately and produce higher levels of compaction.

The failure line was determined experimentally by testing four specimens at different cell pressure levels. For each one of those the confining pressure was maintained constant throughout the test and the deviatoric stress was increased monotonically until the specimen failed. The failure line is shown in Figure 6.5.

To study the resilient behaviour, a single specimen was subjected to a large number of stress paths as indicated in Table 6.1 and Figure 6.5.

Table 6.1 Summary of repeated load triaxial test results on slate waste.

σ_3 (kPa)		q (kPa)		q_r / p_r	Resilient Strains ($\mu\epsilon$)	
min	**max**	**min**	**max**		ϵ_{1r}	ϵ_{3r}
50	50	0	42.0	3.0	231.6	-39.8
100	100	0	42.0	3.0	150.0	-28.3
100	100	0	87.7	3.0	368.4	-60.4
100	100	0	134.5	3.0	583.0	-92.8
100	100	0	184.3	3.0	786.8	-130.8
150	150	0	42.5	3.0	131.6	-24.0
150	150	0	90.6	3.0	298.5	-47.2
150	150	0	137.5	3.0	462.5	-74.4
150	150	0	185.0	3.0	638.8	-100.0
200	200	0	46.5	3.0	120.0	-22.5
200	200	0	90.2	3.0	250.2	-37.4
200	200	0	135.8	3.0	379.7	-55.3
200	200	0	186.4	3.0	545.2	-86.2
250	250	0	44.0	3.0	95.0	-19.0
250	250	0	89.9	3.0	217.7	-36.3
250	250	0	138.0	3.0	332.3	-54.8
250	250	0	184.0	3.0	469.6	-70.3

To improve the quality of the readings from the instrumentation which are automatically recorded in a computer file, a modified least squares filter was applied. The smoothed value, u_n, is given as a weighted average of five adjacent values from the original data [Hamming, 1983], successfully decreasing eventual levels of noise and facilitating the data analysis:

$$u_n = \frac{7\,u_{n-2} + 24\,u_{n-1} + 34\,u_n + 24\,u_{n+1} + 7\,u_{n+2}}{96} \qquad 6.5$$

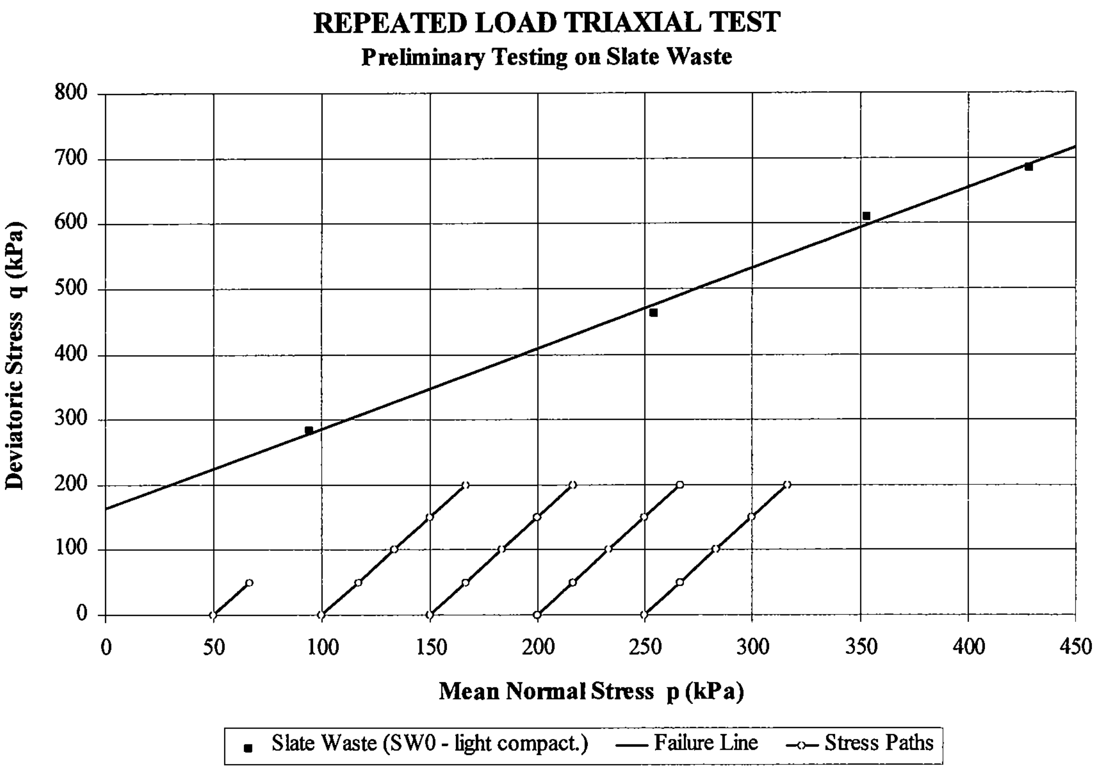

Figure 6.5 Triaxial stress paths and failure line (slate waste - SW0).

A summary of the results is presented in Table 6.1 and representative results obtained for the axial strain, radial strain and Poisson's ratio are shown in Figures 6.6 to 6.8. Some typical characteristics of granular materials behaviour are evident, such as, the non-linear relationship between stresses and strains (evident in the shape and spacing of the lines in Figures 6.6 and 6.7). The increase of resilient modulus with confining stress is particularly noticeable. A more remarkable feature is the very low Poisson's ratio obtained, 0.14-0.15, which remains approximately constant with the deviator stress and cell pressure. At low levels of deviatoric stress, and therefore low levels of strains, the accuracy of the system to measure strains decreases (in particular for radial strains). Thus the reliability of Poisson's ratio values for those levels is questionable. The low Poisson's ratio is likely to be a consequence of the high flakiness of the particles which tend to have a predominantly horizontal orientation easily observed after compaction.

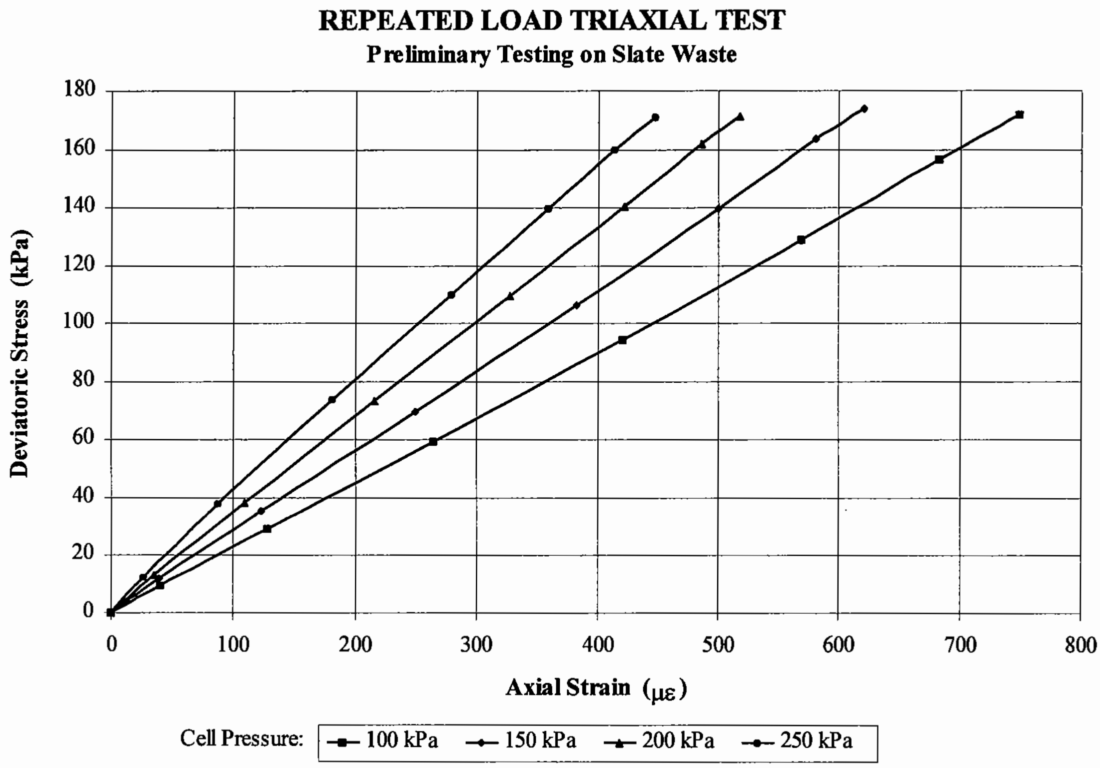

Figure 6.6 Axial strain during one loading cycle (slate waste - SW0).

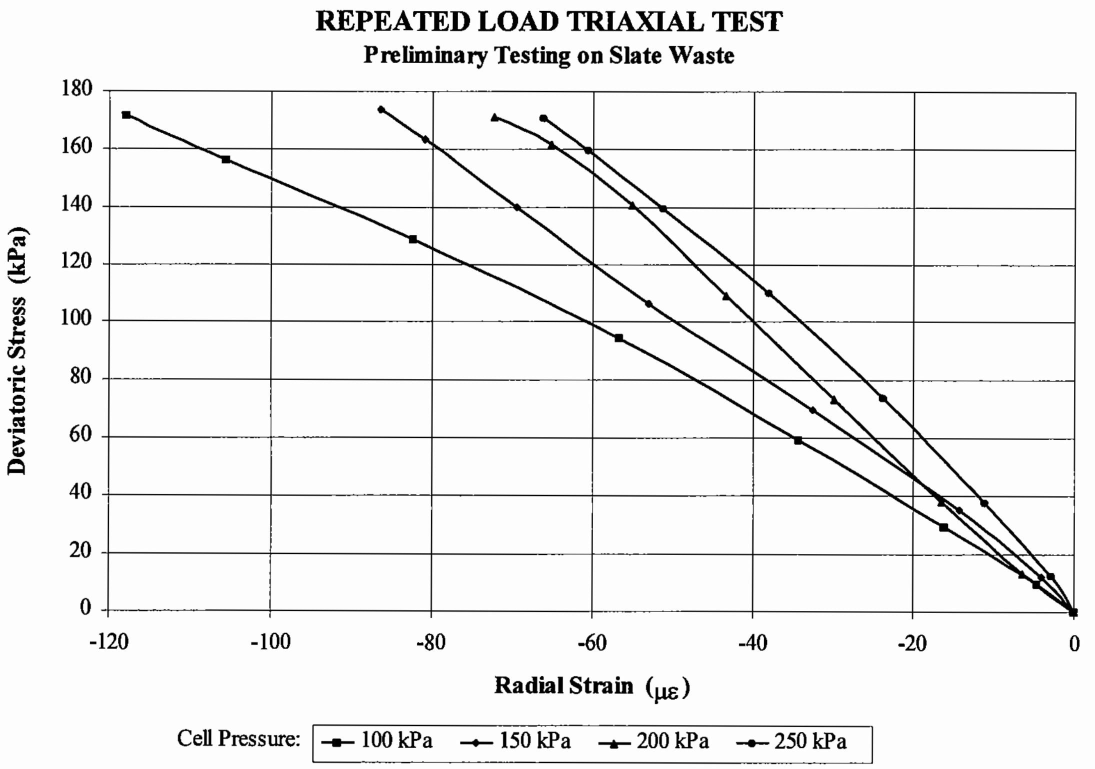

Figure 6.7 Radial strain during one loading cycle (slate waste - SW0).

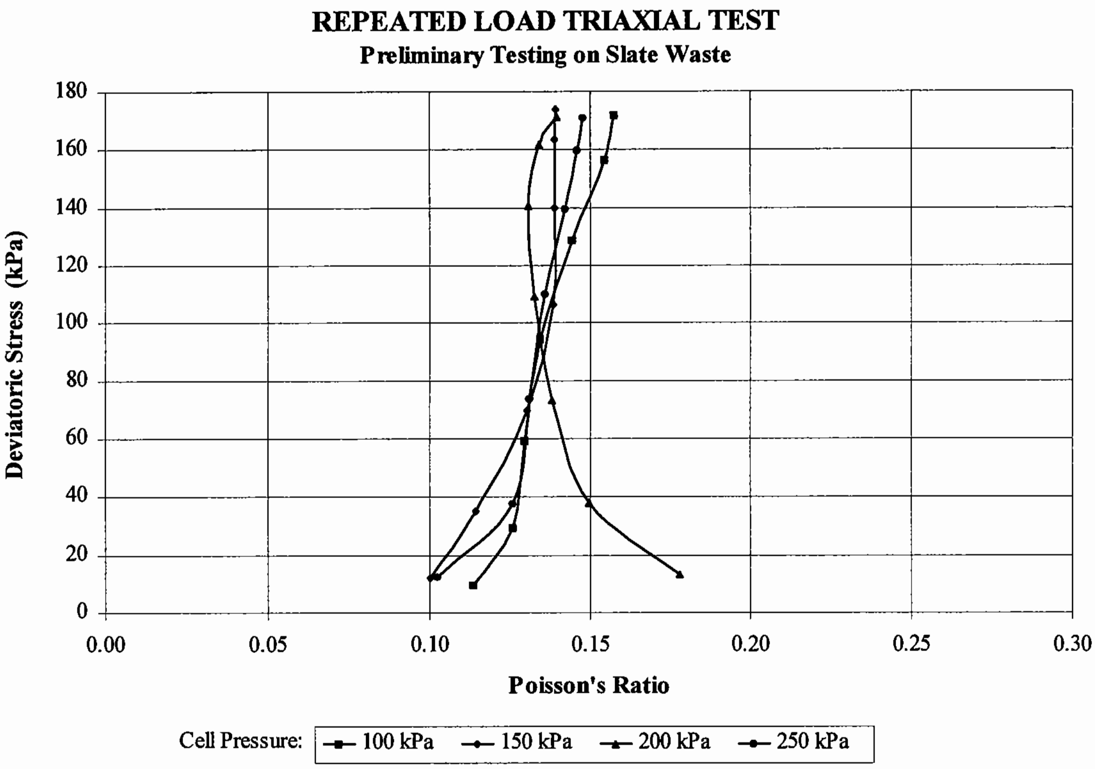

Figure 6.8 Poisson's ratio during one loading cycle (slate waste - SW0).

To model the resilient behaviour of slate waste in this preliminary phase both K-θ and Uzan models were used. These models are presented in Section 6.6.2 and the parameters obtained are listed in Table 6.2. These constants were obtained using a statistical software able to perform multivariate non-linear regression analysis.

Table 6.2 Model constants and correlation coefficient (slate waste).

Model	K_1	K_2	K_3	R^2	ν
K - θ	1136	0.586	-----	0.77	0.15
Uzan	906	0.707	- 0.203	0.98	

It is common practice to determine the constants of non-linear models to transform the model equation to make it linear, in order to calculate the parameters without need to use complex and expensive statistical programs. It should be noted, however, that this does not lead to the same results. One procedure in relation to the K-θ model (Equation *6.19*) is to apply logarithms to both sides of the equation and then use simple linear regression to determine the constants. Following this method the values obtained are $K_1 = 1202$, $K_2 = 0.552$ and $R^2 = 0.79$, which introduces an extra error of up to 3.5 % for the range of stress paths tested and slightly overestimates the correlation coefficient.

Another technique consists of using different sets of data from repeated load triaxial tests to determine different constants of the same non-linear model. For example in the case of isotropic models, it is possible to take advantage of this fact and use experimental results from repeated cell pressure tests to evaluate some constants and then use data from repeated deviatoric stress tests to calculate the other constants of the same model [Karasahin, 1993]. Even though this may simplify the process of calculating the material constants, it is unlikely that the final model will correspond to the best fit for the totality of data available, since the constants were determined with partial data. From experience in this study, it is recommended that constants are determined using multivariate non-linear regression analysis carried out using the totality of adequate experimental results obtained.

In this preliminary phase, only these two models were used due to the simplicity of the stress paths applied for this particular study, in which the initial deviator stress was zero and the confining pressure constant. Another important factor is that the measured Poisson's ratio was found to be approximately constant for the slate as assumed by these models.

The approach using K- θ model is quite poor because this model neglects the influence of the deviatoric stress on the resilient behaviour. This is evident from the variation within the sets of data plotted on Figure 6.9 where each group of data represents results from one cell pressure with deviator stress increasing left to right. The inadequacy of this model has been confirmed by other researchers [Correia, 1985; Karasahin, 1993].

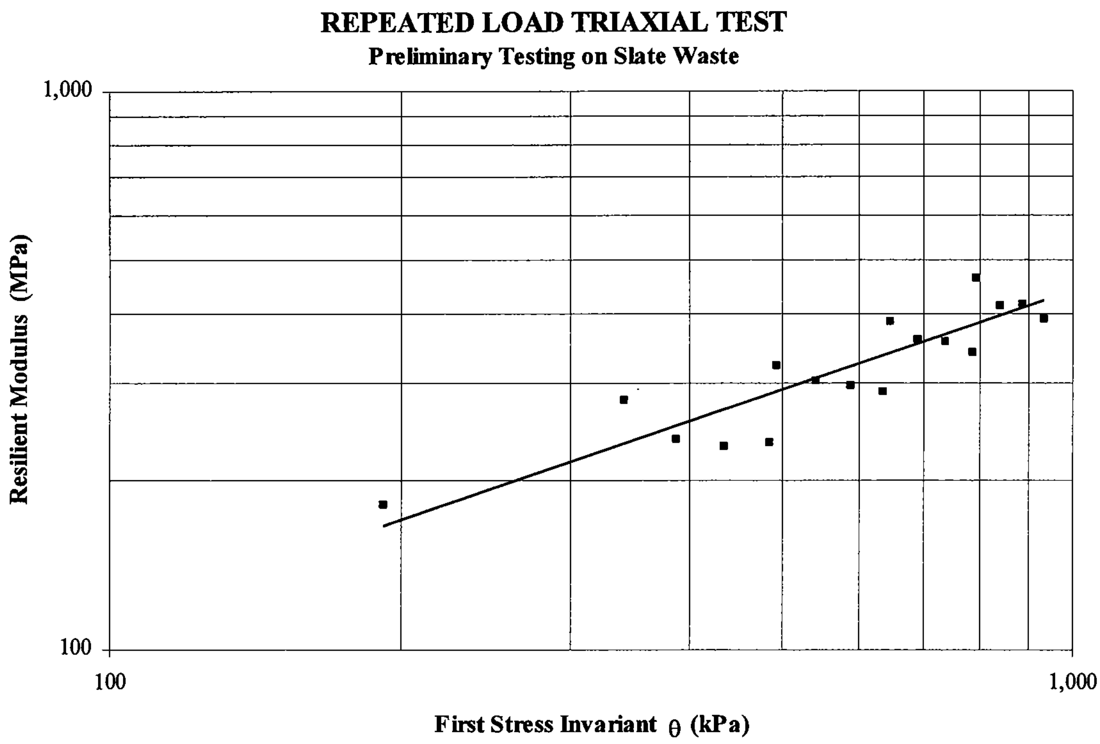

Figure 6.9 Resilient behaviour approach using K - θ model (slate waste - SW0).

For comparative purposes, the resilient modulus value was computed using the parameters of Table 6.2 for a particular stress condition that had been used previously for other materials with similar compaction [Thom, 1988; Thom & Brown, 1989]. This stress path corresponds to the application of a constant confining pressure of 50 kPa and cycling the deviatoric stress between 0 and 100 kPa. Under these conditions the value of resilient modulus was calculated as being 175 MPa. Comparing the slate waste with other materials [Dawson, Brown & Thom, 1989; Dawson & Nunes, 1993], it seems that the slate waste performs slightly better than furnace bottom ash, sand and sand with gravel, as far as resilient properties are concerned. A higher resilient modulus was obtained at higher levels of compaction, more representative of those achieved in the field (this will be reported later in this chapter). The very low Poisson's ratio of the slate waste should be taken into account in pavement analysis and design.

6.4. Test technique

For the later work, it was important to have a detailed test procedure since several materials were to be tested in the repeated load triaxial rig. This is an important aspect since the aim was to make a comparison of the properties of secondary materials and of conventional aggregates.

Some recommendations for the performance of repeated load triaxial tests resulted from a laboratory twinning project that was recently concluded between the Laboratório Nacional de Engenharia Civil (Portugal), the University of Nottingham (United Kingdom), the Laboratoire Central des Ponts et Chaussées (France) and the Delft University of Technology (The Netherlands). The aims of this project were [Galjaard, Paute & Dawson, 1996]: to study the behaviour of soil and granular materials for road pavement construction; to compare the performance of triaxial test equipments; to develop and validate constitutive models; to compare and harmonise equipment, test procedures and design methods. These recommendations seemed to be an excellent starting point for testing secondary aggregates. They have been embraced by the European Committee for Standardisation (CEN) and are now part of an European Standard Draft [CEN, 1995].

The study of granular materials under repeated loading in the triaxial apparatus includes several phases of experimental work: specimen preparation, specimen conditioning, resilient behaviour tests and permanent behaviour tests. Each of these phases are briefly described below, taking into consideration the recommendations of CEN and of the laboratory twinning project referred above [CEN, 1995; Paute, Dawson & Galjaard, 1996; Paute, Hornych & Benaben, 1996]. Some modifications were necessary to test procedures in the light of the specific characteristics of secondary materials which will also be reported.

6.4.1. Development of compaction technique

The compaction technique should be able to reproduce the levels of compaction achieved in road construction in which the target is the maximum dry density obtained in the compactibility test. It should also simulate the compaction mechanism in the field in order to obtain a compacted material whose particles are organised and orientated in a similar manner to that of the materials in the pavement structure. This is especially important for the study of permanent deformations which are particularly sensitive to the method of compaction [Thom, 1988; Brown & Selig, 1991].

From the preliminary tests performed on slate waste it was evident that the use of vibrating table with or without surcharge load was not enough to reach high levels of density and that a different technique would have to be adopted.

As to the simulation of the compaction mechanism, unbound materials are laid in the field and then, after the required amounts of water are added, they are compacted using rollers. This process follows a method that specifies the number of necessary passes which depend on the type of compaction plant, its static mass and the compacted thickness of the material [DoT, 1993]. The most common rollers apply vibration to the material under compaction.

The simulation of this mechanism in the laboratory is difficult. For bituminous materials one of the methods used at the University of Nottingham is the compaction of slabs in the roller compactor rig [Brown et al, 1995]. This equipment simulates the action of a deadweight roller and includes a pivoted steel roller segment that compacts the material while an horizontal ram drives the slab mould backwards and forwards. From those slabs beam specimens can be cut or cylindrical specimens drilled for testing. For unbound materials, however, this method cannot be used.

For the manufacture of specimens with stabilised materials a vibro-compression technique has been used in France [Chi, 1976; NF P 98-230, 1992]. It results from the double action of an axial compression and radial vibration generating specimens with highly homogeneous densities. This method, which could be used for unbound materials since the specimen is compacted in its final form, was not available for this research and could not be developed in time.

It has been demonstrated that moisture content and density affect the properties of pavement materials under repeated loading [Dawson, Thom & Paute, 1996; Thom & Brown, 1987; Thom & Brown, 1988]. In general, an increase in moisture content results in lower resilient moduli, higher Poisson's ratio and larger permanent deformations. Moreover, it is generally accepted that density only slightly affects the resilient properties of granular materials and that its influence on the permanent deformation behaviour is much more significant.

The specimen should be tested at a density and moisture content simulating the equilibrium pavement conditions as closely as possible. According to the Specification for Highway Works [DoT, 1993] these conditions shall be considered as being the optimum moisture content and the density corresponding to a uniform air voids content of 5 %. As an alternative, a reasonable range of water contents and densities may be chosen, for instance, within 1 % above and 2 % below the optimum moisture content and within 95 % and 100 % of the maximum dry density. Paute, Dawson & Galjaard [1996] proposed a similar range of dry densities and three levels of water content: 1, 2 and 4 % below the optimum value. However, for the objectives of this project the analysis of several secondary materials using a range of moisture contents and densities was not practical and, therefore, the study was conducted at the optimum values which correspond to target field requirements and the values obtained should be reasonably on the conservative side. Another reason to support this choice are the results obtained in the conventional testing phase from which it was concluded that the point corresponding to 5 % air voids seems to correspond to an excessive degree of compaction for the materials studied (Chapter 3).

In the light of previous considerations and after several tentative trials the following technique was developed (Figures 6.10 to 6.12 illustrate the method):

- a latex membrane to which four studs have been fixed is placed inside a mould and sealed to both extremities of the mould with two o-rings. To ensure the correct position of the studs, to which the instrumentation will be attached, and hold the membrane in place, vacuum is applied between the internal wall of the mould and the membrane (Figure 6.12);

- unbound samples were compacted in the four-piece aluminium split mould normally used for the preparation of specimens for triaxial testing [Brown, O'Reilly & Pappin, 1989];
- stabilised specimens were compacted in new two-piece cylindrical plastic moulds, reinforced at different levels by "jubilee" clips hindering any lateral deformation of their walls (Figure 6.11). This type of mould is compatible with the need to leave the specimens curing inside the mould during the first few days before the material develops strength since the moulds are easy to manufacture and relatively inexpensive;
- the compaction energy is transmitted to the specimen by both a vibrating table, on top of which the compaction mould is placed and fixed, and a vertically-guided vibrating hammer operating on top of each layer;
- the compaction is done in five layers and controlled layer by layer. The time and characteristics of vibration (frequency and amplitude) may be adjusted according to the type of material, in order to achieve the aimed densities;
- the compaction is performed at the optimum moisture content and maximum dry density obtained in the compactibility test for graded aggregates [BS 5835, 1980];
- triaxial tests must be performed at constant moisture content, and for this reason it is important to guarantee that no water is lost during testing. For this purpose a waterproof geotextile (treated with silicone emulsion) is placed at the interface between specimen and both top and bottom platens.

A more detailed description of sample preparation and setting-up procedure for the repeated load triaxial test may be found in the bibliography [Brown, O'Reilly & Pappin, 1989; Nunes, 1996].

This procedure enables the compaction of materials at high densities, simulates the vibration applied by the rollers during compaction and, by using partial section compaction feet, can in some way simulate their kneading action.

After compaction, unbound specimens are left for about 24 hours to allow the dissipation of pore pressures which may have been generated during the process. For permeable or dry specimens, this may not be necessary and then the triaxial specimen may be tested

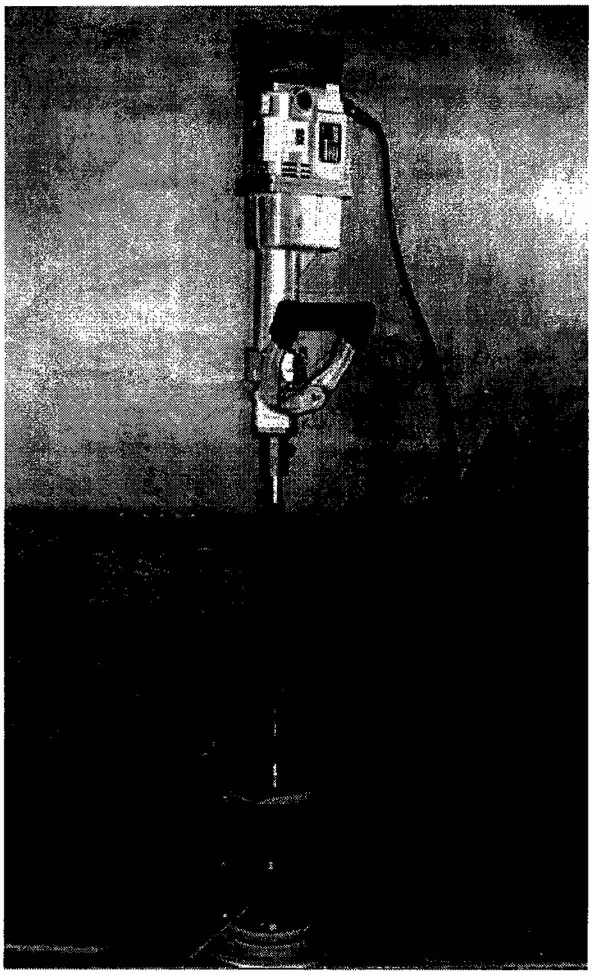

Figure 6.10 Specimen compaction for triaxial testing.

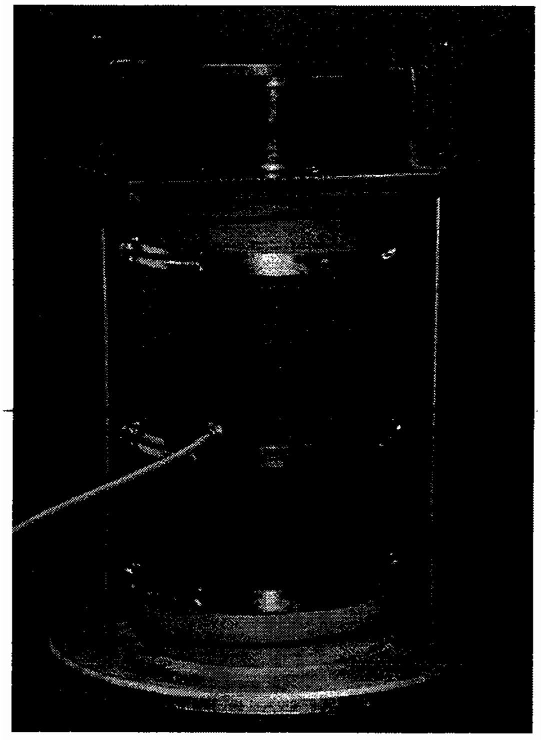

Figure 6.11 Detail of compaction mould.

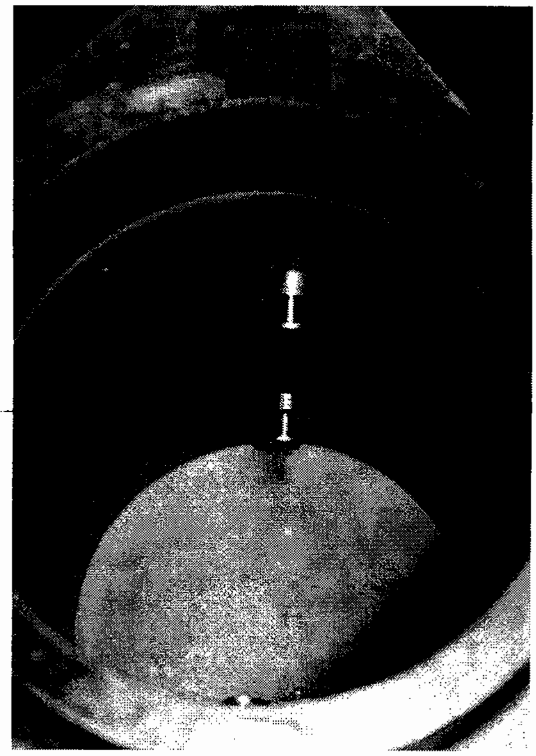

Figure 6.12 Detail of studs' assembly.

immediately after compaction.

For treated materials, however, the specimens have to be left for curing. The following guidelines were followed to assure similar conditions to all materials, including the heavily-stabilised mixtures studied in Chapter 8:

- immediately after compaction, the specimens should be carefully sealed to avoid the loss of moisture. This is essential for strength development and may be achieved by using plastic membranes to wrap the specimens tightly;
- sealed specimens should be stored in a temperature controlled room at constant temperature of 20 °C;
- after a period of 2 to 3 days the specimens should have already developed an initial strength enabling the removal of the moulds and thus releasing them for the compaction of further specimens. The specimen should again be adequately sealed and stored at constant temperature (20 °C) until the end of the curing period;
- especially for highly permeable mixtures, it is important to turn specimens up-side down periodically in order to avoid concentration of moisture in the lower half of the specimens which would result in considerable heterogeneity within the specimen.

6.4.2. Specimen conditioning

The specimen conditioning procedure is aimed at bringing the specimen to a stable state of fully resilient behaviour, in which the deformations are almost completely recovered after each load application. It has been demonstrated [Galjaard, Paute & Dawson, 1996] that resilient strains can decrease significantly from the beginning of the test tending to stabilise after 5000 cycles approximately, demonstrating the importance of the specimen conditioning for the study of resilient properties. In addition, as a large number of cycles is applied to the specimen and both resilient and permanent deformations are recorded throughout the specimen conditioning, it also results in some valuable permanent deformation behaviour data.

In the programme described here, the conditioning of the specimen consists of applying 20000 cycles of loading to the specimen, at the stress levels indicated in Table 6.3. The stresses are applied in the form of half sine waves with a frequency of loading of 1 Hz [Sweere, 1990; Brown & Selig, 1991]. There was a rest period between pulses of 0.5 seconds.

Table 6.3 Stress levels for the conditioning of the specimen [after Paute, Dawson & Galjaard, 1996].

Confining Stress, σ_3 (kPa)		**Deviator Stress, q (kPa)**		
min	**max**	**min**	**max**	q_r/p_r
0	100	0	600	2.0

Stresses and strains (resilient and permanent) are recorded at the following values of the cycle number, N: 20, 50, 100, 200, 400, 1000, 2500, 5000, 7500, 10000, 12500, 15000 and 20000. After finishing the conditioning test, the specimen is left without loading for some time until all deformations have ceased.

The first attempts to condition the specimen at the specified stress path (Table 6.3) resulted in the specimen failure. Two specimens of unbound china clay sand and one of furnace bottom ash treated with 2 % of cement failed during the conditioning phase. These materials are believed to be adequate road construction materials and the fact that they failed under the conditioning stress path seems to point to the inappropriateness of the stress path rather than of the material itself. It is believed that the recommended stress path is too harsh for this phase due to the following factors:

- it subjects the specimen to the highest stress level used for the resilient behaviour study without providing an appropriate confinement to the specimen;
- after conditioning the specimen was already subjected to high stresses and due to the introduction of permanent deformations it is slightly more dense and stronger;
- stress paths applied for all phases (conditioning, resilient behaviour and permanent deformations) should be related to the failure line of the materials tested. They should not surpass approximately 70 % of the values corresponding to failure, except for permanent deformation testing when they may exceed

slightly that value;

- from Figure 6.5 it is evident that p_{max} = 300 kPa and q_{max} = 600 kPa will fall beyond failure. Even taking into consideration that slate waste was under-compacted in the preliminary tests, other secondary materials will probably exhibit lower ultimate strength than conventional aggregates, and for those this conditioning stress path is not acceptable.

As a consequence, the use of a constant confining pressure for conditioning the specimen is proposed to safeguard the integrity of the specimens. Since the main objective is to bring the specimen to a stable state and the mechanical properties will be determined in subsequent phases in which the desired stress paths may be applied, the use of constant cell pressure is not likely to affect the results in a significant manner. The resulting stress path is shown in Table 6.4 and Figure 6.13.

Table 6.4 Proposed conditioning stress path.

Confining Stress, σ_3 (kPa)		Deviator Stress, q (kPa)		
min	**max**	**min**	**max**	q_r/p_r
100	100	0	*Minimum of:* 600 kPa 70 % of failure	3.0

6.4.3. Resilient behaviour tests

According to Paute, Dawson & Galjaard [1996], to study the resilient behaviour a range of stress paths is applied to the specimen corresponding to stress ratios, q_r / p_r, equal to 0.0, 0.5, 1.5, 2.0 and 2.5, and two different minimum confining stresses, $\sigma_{3\ min}$, 0 and 15 kPa. The corresponding stress levels are presented in Table 6.5 and Figure 6.14. The frequency of loading should be of 1 Hz with 0.5 s rest periods, as for the conditioning test.

As in the modified conditioning phase, the stress paths should not be higher than 70 % of the values corresponding to failure. In practice it may be necessary to limit the most demanding stress paths: D3 & E3 of the first series and I3 & J3 of the second series.

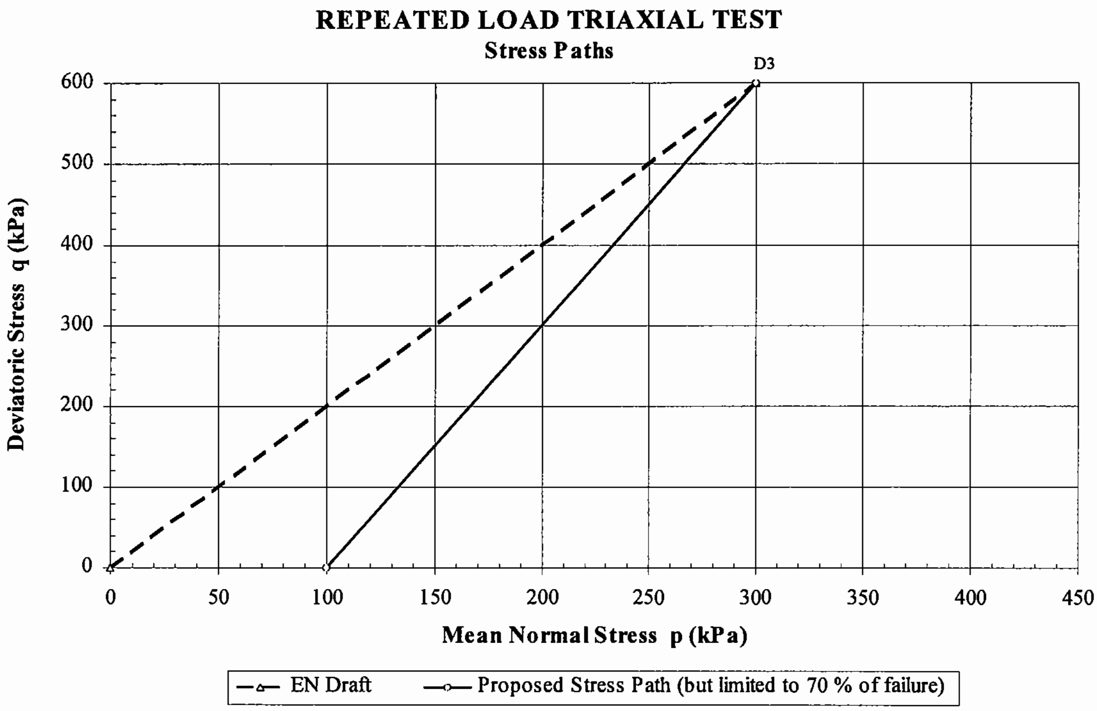

Figure 6.13 Comparison of conditioning stress paths.

Table 6.5 Stress levels for the resilient behaviour study [after Paute, Dawson & Galjaard, 1996].

Path	1				2				3				4				q_r/p_r
Code	σ_3 (kPa)		q (kPa)		σ_3 (kPa)		q (kPa)		σ_3 (kPa)		q (kPa)		σ_3 (kPa)		q (kPa)		
	min	max	min	max	min	max	min	max	min	max	min	max	min	max	min	max	
First Series - $\sigma_{3\,min} = 0$ kPa																	
A	0	250	0	0	10	260	150	150									0.0
B	0	50	0	30	0	100	0	60	0	175	0	105	0	250	0	150	0.5
C	0	50	0	150	0	100	0	300	0	150	0	450	0	200	0	600	1.5
D	0	30	0	180	0	60	0	360	0	100	0	600					2.0
E	0	10	0	150	0	15	0	225	0	20	0	300					2.5
Second Series - $\sigma_{3\,min} = 15$ kPa																	
F	15	265	0	0	25	275	150	150									0.0
G	15	65	0	30	15	115	0	60	15	190	0	105	15	265	0	150	0.5
H	15	65	0	150	15	115	0	300	15	165	0	450	15	215	0	600	1.5
I	15	45	0	180	15	75	0	360	15	115	0	600					2.0
J	15	25	0	150	15	30	0	225	15	35	0	300					2.5

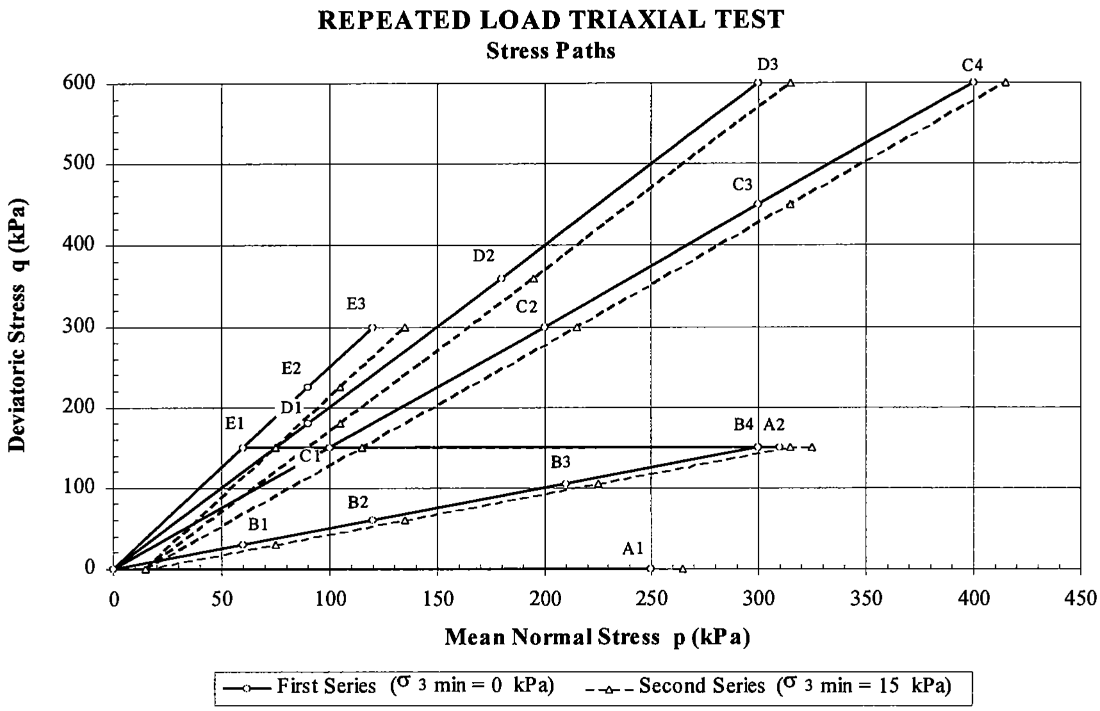

Figure 6.14 Stress paths for the study of resilient behaviour.

Each stress level is applied for 100 cycles in the sequence A1, A2, B1, B2, B3, ..., and the stresses and strains are recorded between cycles 90 and 100 for each path, allowing for strain stabilisation after each. Finally, after applying all stress levels the specimen is removed from the triaxial rig and the moisture content is determined.

The use of stress paths corresponding to confining stresses of up to 265 kPa and deviatoric stresses of up to 600 kPa represents a radical change in the way repeated load triaxial tests are performed. In fact, previous researchers [TRB, 1975; Thom, 1988; Karasahin, 1993; Cheung, 1994] have favoured the use of considerably lower levels of stress that are in accordance with the stresses induced in a finished road pavement. Yet, calculations of the vertical stresses induced in a pavement structure have shown that the stress levels during the construction phase may be ten times higher than the corresponding stresses in the same pavement during service [Cheung, 1994]. This reinforces the need to use a wide range of stress paths for triaxial testing, including the application of high levels of stress in order to simulate the construction phase.

6.4.4. Permanent behaviour tests

For the characterisation of permanent deformation properties several stress paths are used, but unlike for resilient behaviour, each stress path is applied to a new specimen without previous conditioning, since the number of load applications and the levels of stresses will fundamentally alter the specimen conditions. A minimum of four specimens need to be tested. Table 6.6 and Figure 6.15 present the stress levels used in accordance with the CEN draft [CEN, 1995] and the frequency of loading was 1 Hz.

Table 6.6 Stress levels for the permanent behaviour study [after Paute, Dawson & Galjaard, 1996].

Confining Stress, σ_3 (kPa)		**Deviator Stress, q (kPa)**		q_r/p_r
min	**max**	**min**	**max**	
0	20	0	300	2.5
0	100	0	600	2.0
0	200	0	600	1.5
0	75	0	300	1.7

Each stress level is repeatedly applied, 80000 times, and the stresses and strains recorded for the following numbers of load applications, N: 20, 50, 100, 200, 400, 1000, 2500, 5000, 7500, 10000, 12500, 15000, 20000, 40000, 60000, 80000.

The application of extra stress paths may be necessary in some cases. Ideally, the final permanent strains should be within the range 2000 and 20000 $\mu\epsilon$ in order to permit the modelling of results [Paute, Dawson & Galjaard, 1996].

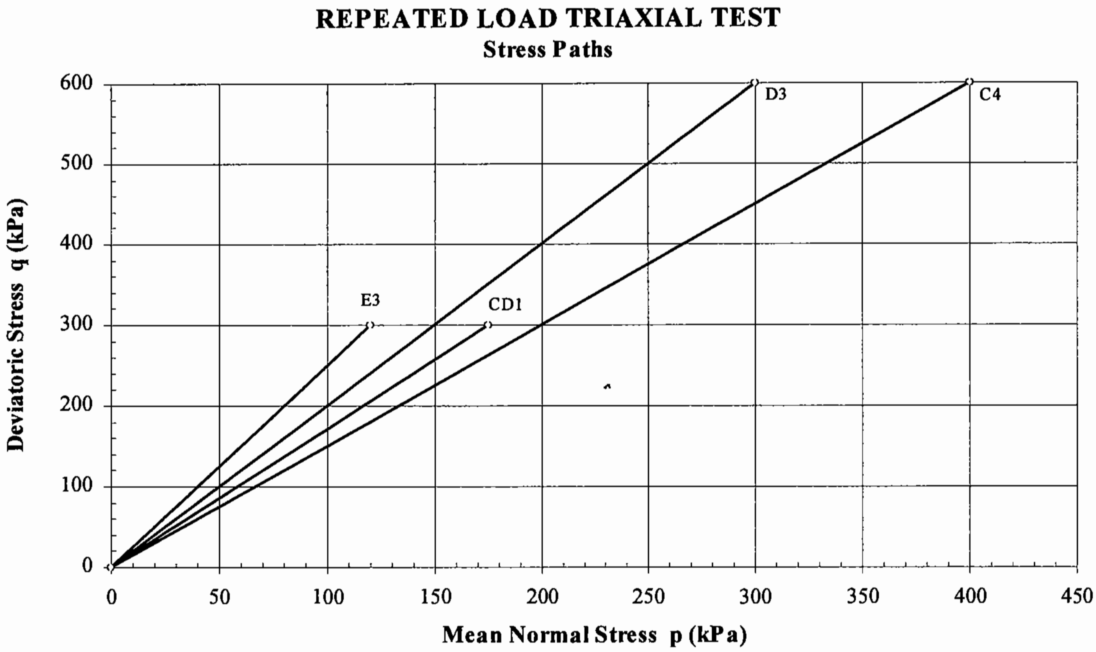

Figure 6.15 Stress paths for the study of permanent deformation behaviour.

6.5. Typical stress / strain results

Stress-strain results are presented in Figures 6.16 to 6.18 illustrating the typical properties of granular materials described in the previous section with experimental data obtained from monotonic and repeated load triaxial tests performed on unbound slate waste.

An interesting aspect that results from these figures is the curvature of the loading part of the stress / strain hysteresis loop corresponding to an increase of the resilient modulus with the stress level. This is contrary to what is widely presented in the bibliography and reproduced in Figure 6.4, which corresponds to a decrease of the resilient modulus as the peak deviator stress increases. Allaart [1992] clearly distinguishes the behaviour of granular materials under repeated loading from that of steel, linear elastic behaviour, and concrete characterised by decreasing resilient modulus with the stress level as a result of internal damaging of the material (Figure 6.19). He explains granular materials' stress-strain relationships by considering two spheres acting against each other. For small pressures the area of contact is small and an increase in overall pressure on the material corresponds to a large displacement of the centres of the spheres. On the other hand, for higher overall pressures the area of sphere contact will be larger and the same increase in overall pressure will result in a smaller increase in the contact stresses. This results in smaller displacements and a higher resilient modulus. In addition, resilient dilation is also responsible for the form of the stress / strain curves.

The same type of results is presented in Figures 6.20 to 6.22 for treatment level I, in this case for furnace bottom ash treated with 2 % of cement (see Chapter 4, Table 4.6, for material codes). For the stronger materials of treatment level II, it was not possible to fail the specimens up to the maximum working load of the triaxial equipment (20 kN) corresponding to a maximum deviatoric stress of approximately 1150 kPa. The rutting susceptibility of these materials is also significantly reduced and as a result the rate of accumulation is very small and more affected by electrical noise from the instrumentation. For this reason, only Figure 6.23 is presented illustrating the resilient behaviour of secondary materials treated with higher levels of binder content.

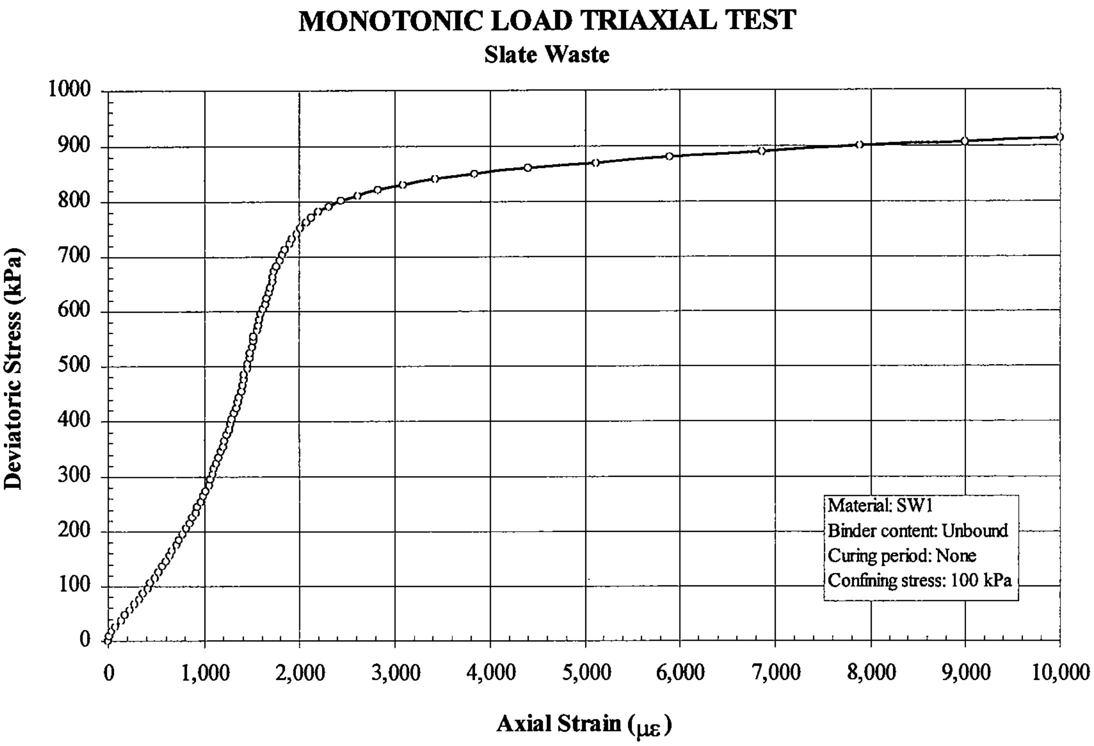

Figure 6.16 Experimental result from triaxial test using monotonic loading (unbound material).

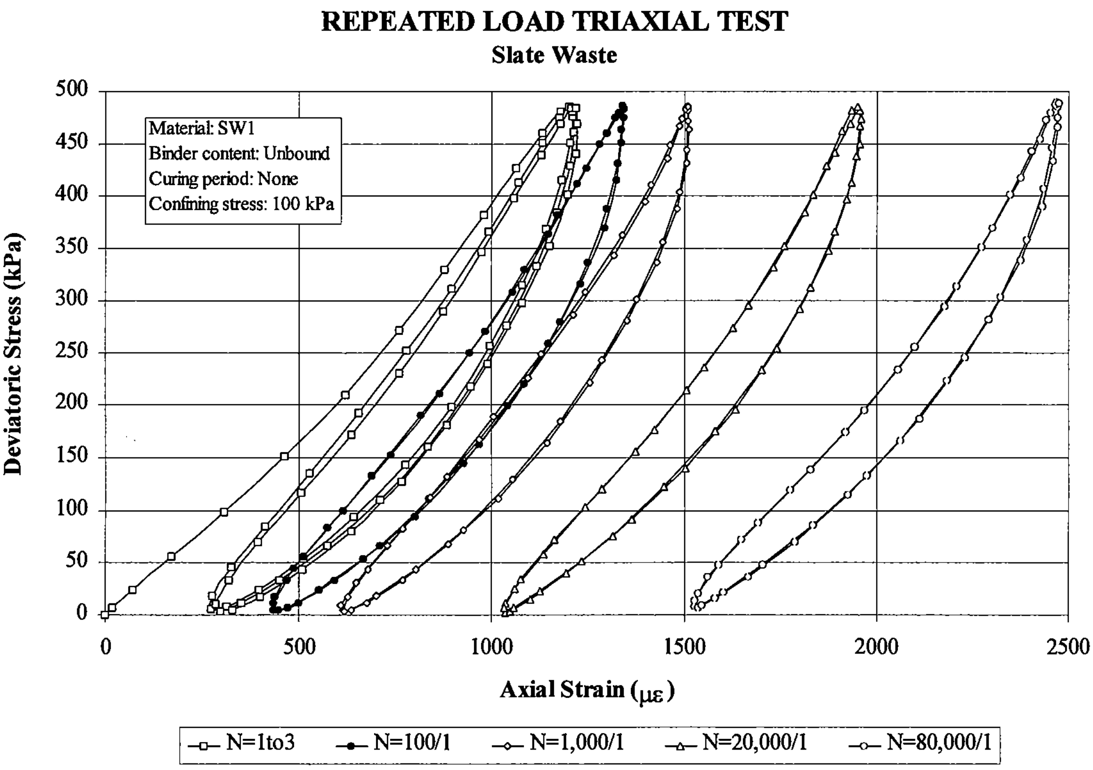

Figure 6.17 Development of permanent deformations in repeated load triaxial tests (unbound material).

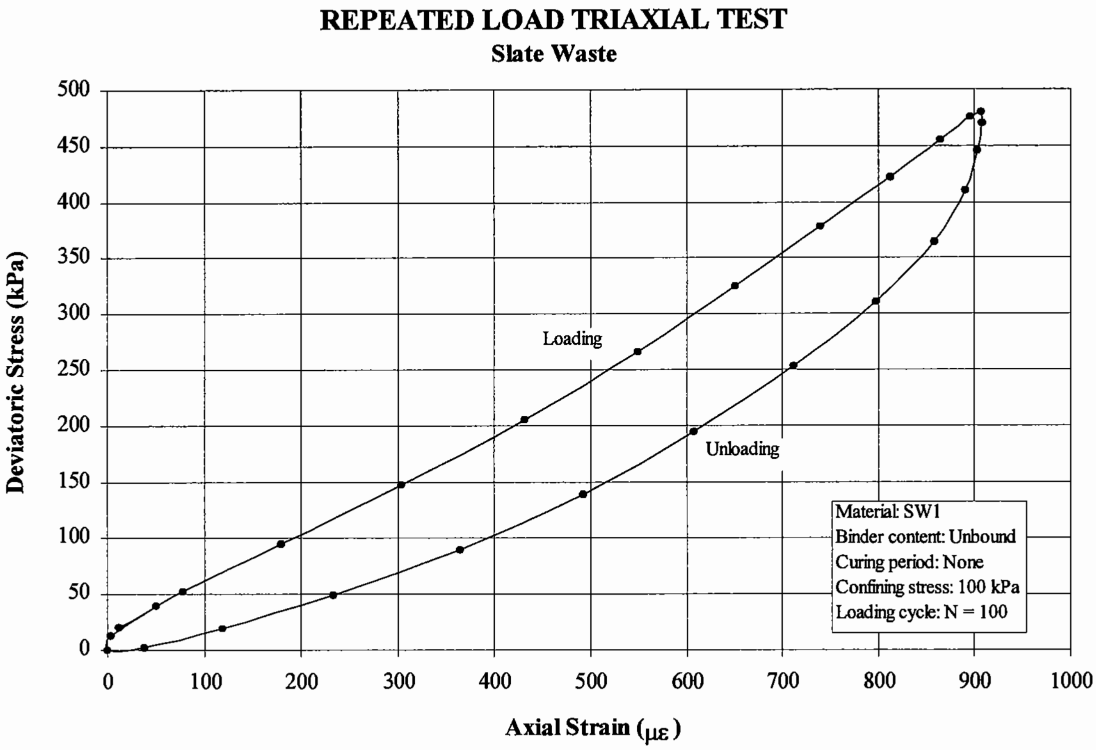

Figure 6.18 Typical resilient behaviour of granular materials under repeated loading (unbound material).

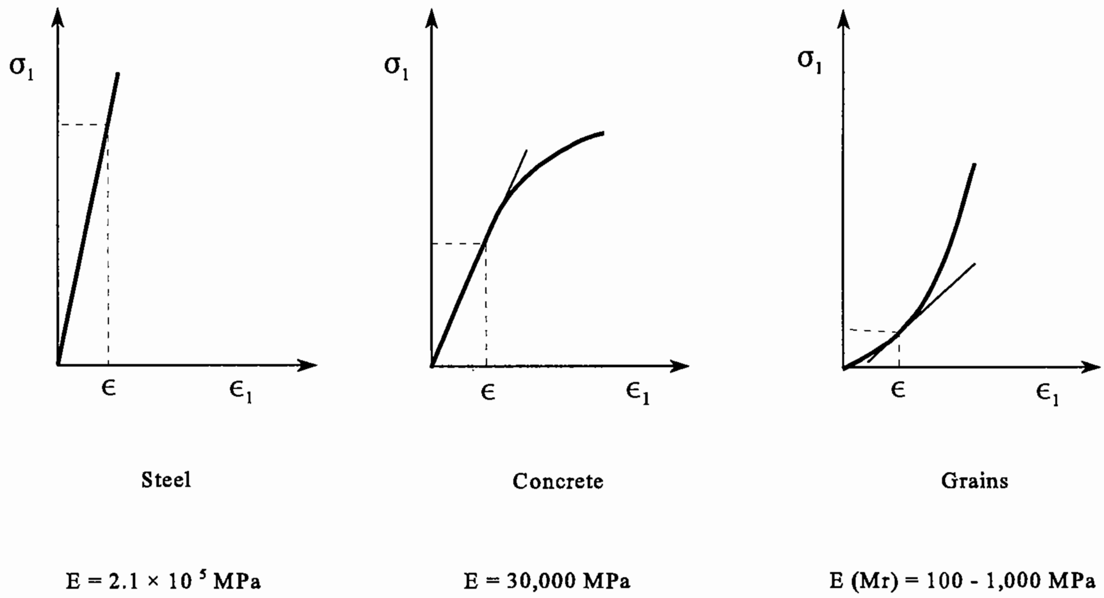

Figure 6.19 Comparison of stress-strain behaviour of steel, concrete and granular materials [Allaart, 1992]

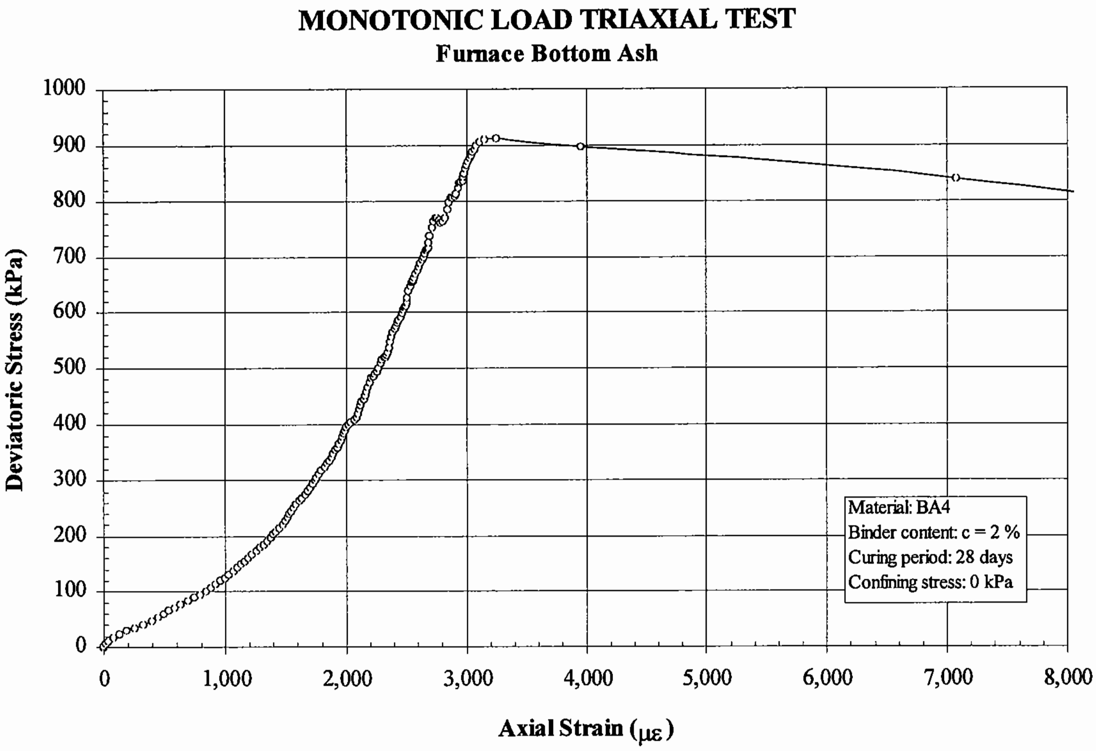

Figure 6.20 Experimental results from triaxial test using monotonic loading (treatment level I).

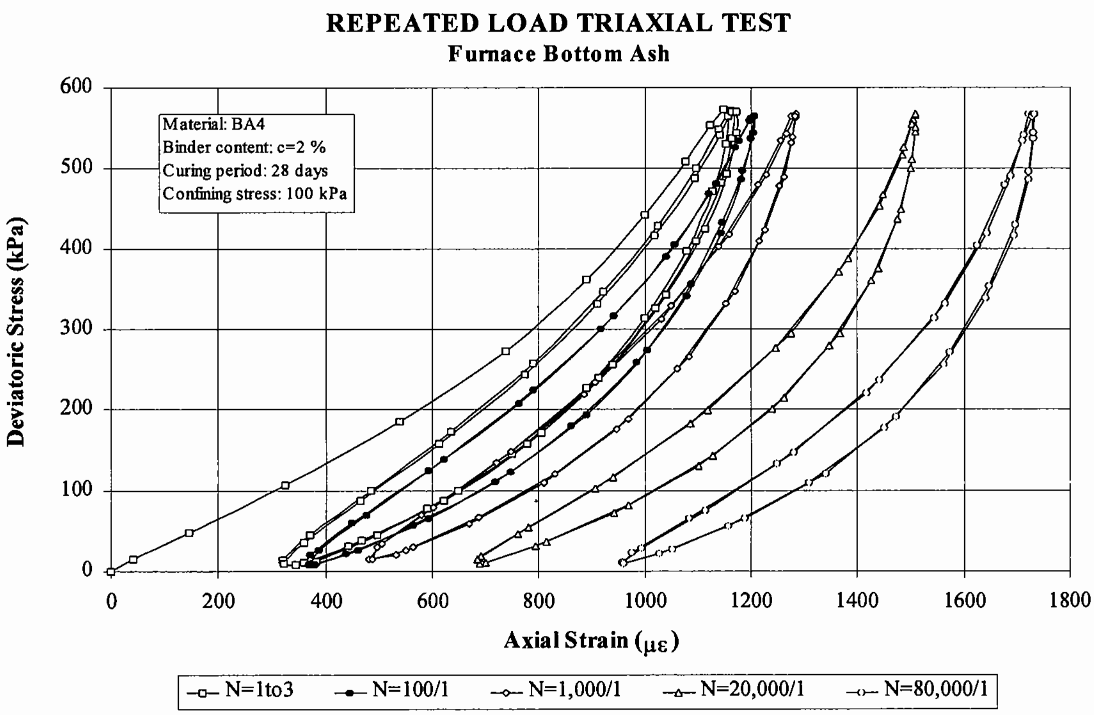

Figure 6.21 Development of permanent deformation in repeated load triaxial tests (treatment level I).

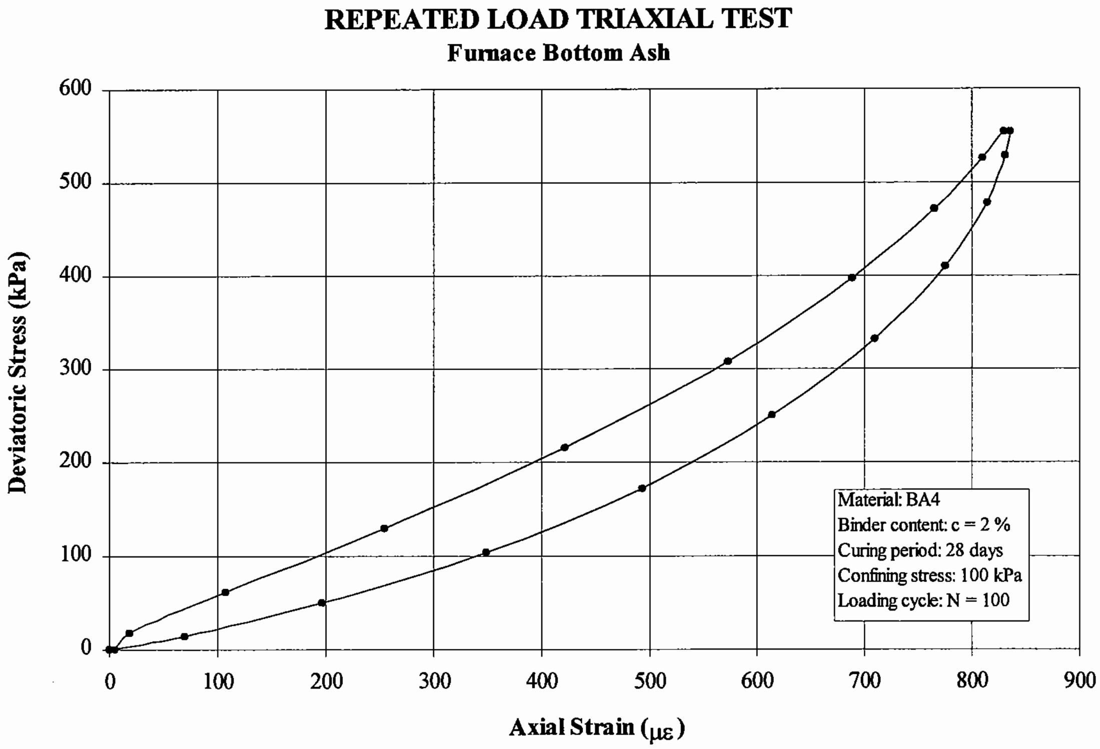

Figure 6.22 Typical resilient behaviour of treated materials under repeated loading (treatment level I).

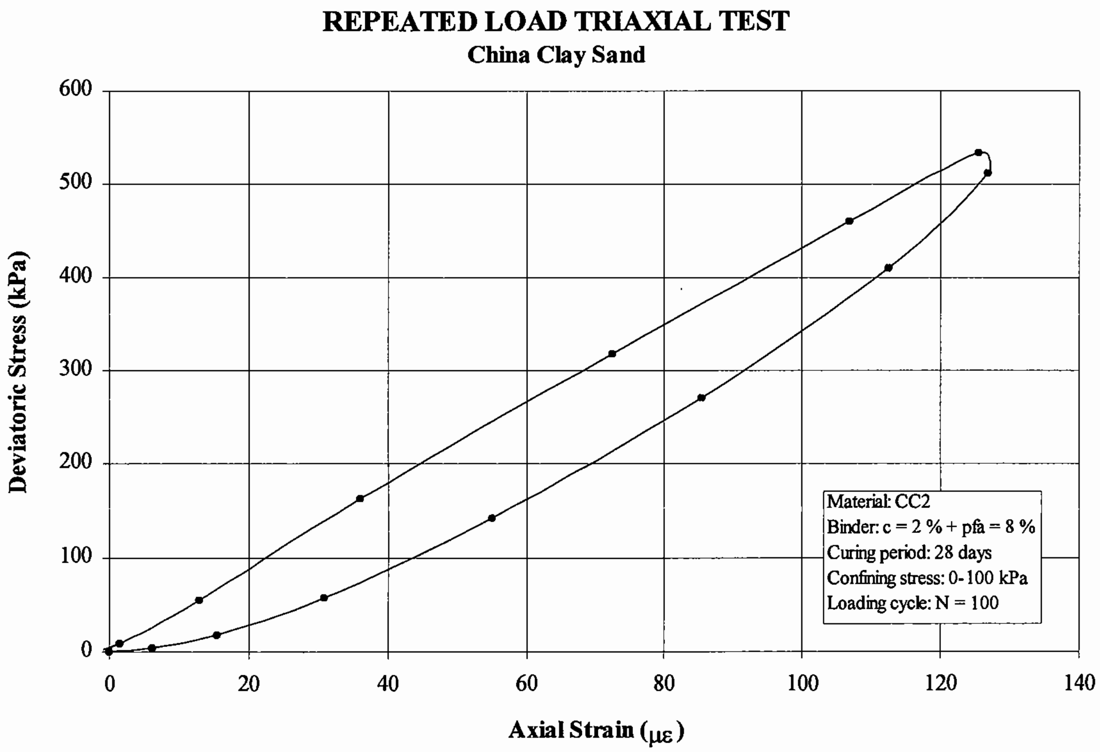

Figure 6.23 Typical resilient behaviour of treated materials under repeated loading (treatment level II).

Comparing treated with unbound materials, it seems that very light treatment does not give rise to a totally different kind of behaviour under repeated loading. This seems to be the case, as long as the final strength / resilient modulus of the stabilised material is not excessively high. This aspect will be discussed in subsequent sections. The main perceptible difference is the amount of energy dissipated during one loading cycle which seems to be reduced for the treated material, reflected by a much narrower hysteresis loop. This tendency is also observed for the stronger china clay sand (treatment level II). In this case, however, the non-linearity of the material and stress dependency is considerably reduced and the stiffness markedly increased.

Finally, taking into account the limitations of the equipment to a maximum load of 20 kN, a summary of the strength results that were possible to obtain is presented in Figures 6.24 and 6.25. In the latter, lime-stabilised minestone (MI1) was only tested with a confining pressure of 50 kPa. An important increase in the strength of slate waste (SW1) was observed in relation to the preliminary test (SW0) when the material was clearly under-compacted at a dry density of 1.88 Mg/m^3 (approximately 84 % of the maximum dry density obtained in the compactibility test). As expected, it is clear from these results that triaxial strength is significantly affected by the density of the material.

The position of the failure lines in Figure 6.24 seems to confirm the idea already expressed that the stress paths applied in repeated load triaxial tests should be related to the failure line of the materials and some of the stress paths proposed in the European Standard Draft [CEN, 1995] may not be possible in practice.

In Figure 6.25, the materials are ranked based on their triaxial strength when tested using monotonic loading without confining pressure ($\sigma_3 = 0$ kPa). The results obtained with a confining pressure of 50 kPa are also given for the weaker materials. The strength improves significantly with treatment of the materials with the exception of minestone stabilised with 5 % of lime (MI1), which remains at the top end of the unbound specimens. Although the mixture is successful in the sense that it brings some cohesion to the material giving a strength slightly lower than for granite Type 1, from the results obtained it would seem that either the inter-particle bonds created are weak, or this is a result indicating a very low particle strength which is not much improved through treatment.

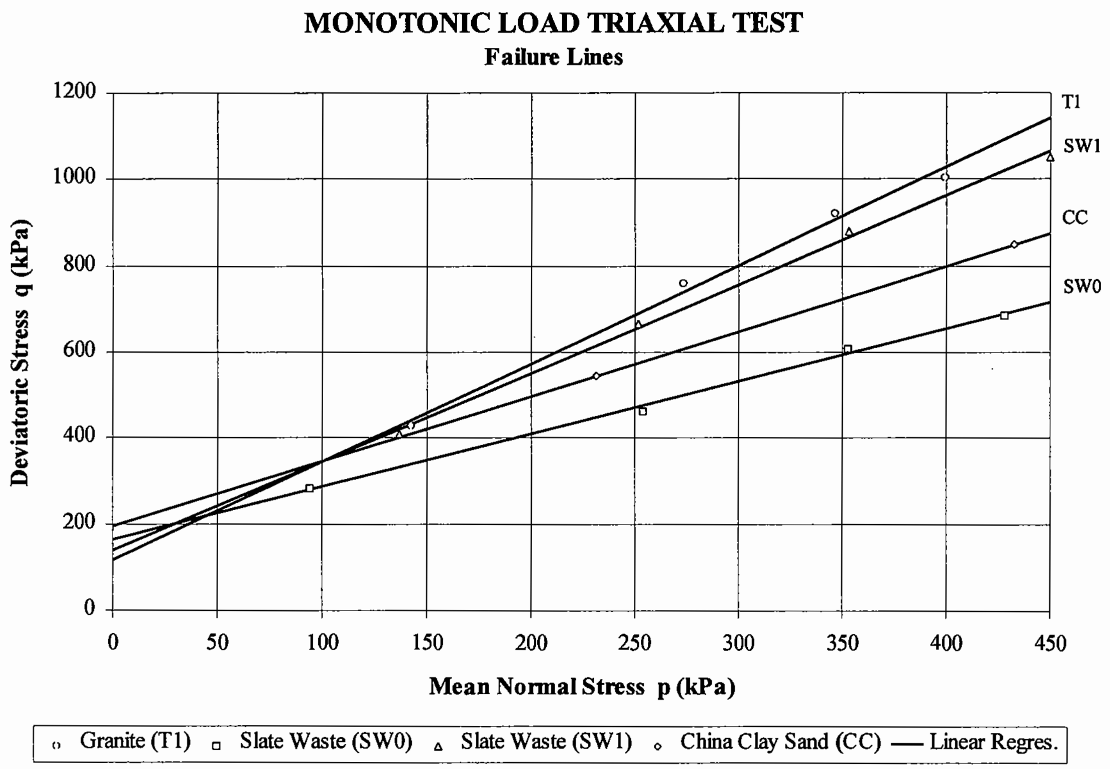

Figure 6.24 Failure lines of the unbound materials.

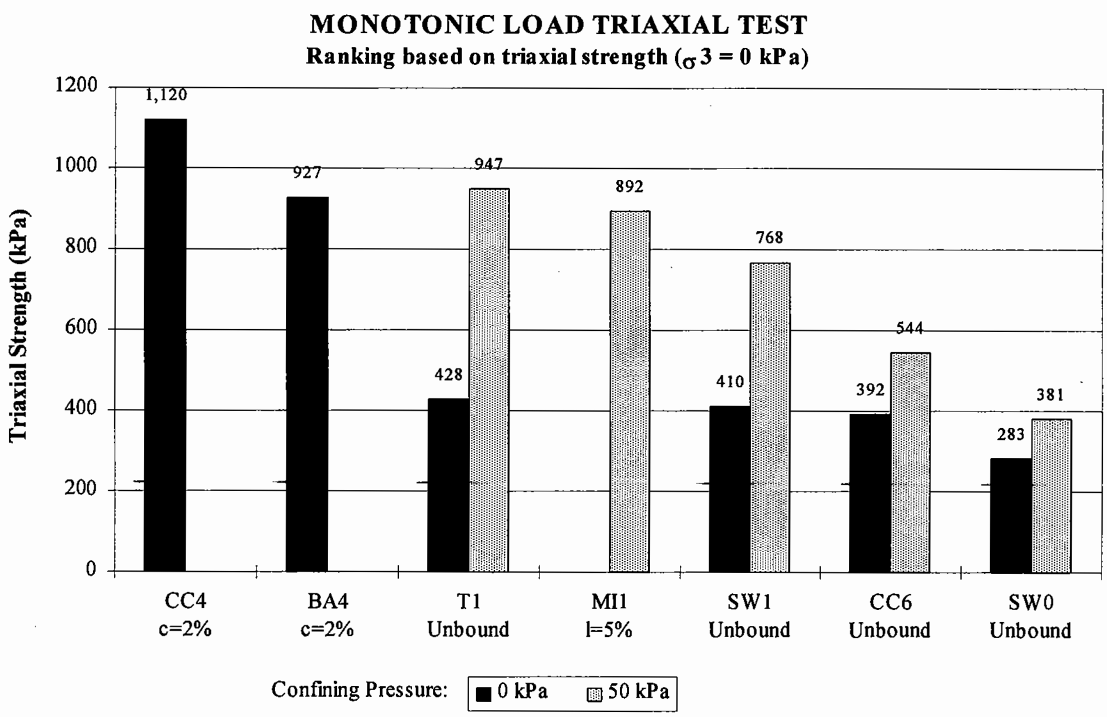

Figure 6.25 Ranking of materials based on triaxial strength.

6.6. Resilient behaviour of secondary materials

Secondary materials are essentially granular materials that may or may not have some specific properties such as: special particle shape (e.g. slate waste), low particle densities (e.g. bottom ash), high particle densities (e.g. steel slag), pozzolanity (e.g. fly ash) etc. In principle, for modelling the behaviour of unbound secondary materials under repeated loading, current models for granular materials should be able to predict adequately the experimental results obtained from repeated load triaxial tests. The situation may change when those materials are treated. The objectives of this section are:

- to review and critically discuss existing models for granular materials in order to identify the ones most appropriate for use with secondary materials;
- to examine whether or not those models can be used with both unbound and bound secondary materials;
- to develop an acceptable model when existing ones do not apply;
- to identify eventual limits of applicability.

6.6.1. General stress-strain relationships

The stress-strain relationships generally used to characterise granular materials make use of Hooke's law governing the behaviour of elastic and isotropic materials:

$$\begin{Bmatrix} \epsilon_1 \\ \epsilon_2 \\ \epsilon_3 \end{Bmatrix} = \frac{1}{E} \begin{bmatrix} 1 & -\nu & -\nu \\ -\nu & 1 & -\nu \\ -\nu & -\nu & 1 \end{bmatrix} \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_3 \end{Bmatrix} \qquad 6.6$$

where: ϵ_1, ϵ_2, ϵ_3 major, intermediate and minor principal strains;

σ_1, σ_2, σ_3 major, intermediate and minor principal stresses;

E Young's modulus;

ν Poisson's ratio.

As mentioned previously, in pavement engineering the term Young's modulus is replaced by resilient modulus due to the non-linear stress-dependent behaviour of granular materials. With this replacement and for triaxial test axisymetric conditions, the following equations are obtained:

$$M_r = \frac{\sigma_{1r}^2 + \sigma_{1r}\,\sigma_{3r} - 2\,\sigma_{3r}^2}{\sigma_{1r}\,\epsilon_{1r} + \sigma_{3r}\,(\epsilon_{1r} - 2\,\epsilon_{3r})} \tag{6.7}$$

$$\nu = \frac{\sigma_{1r}\,\epsilon_{3r} - \sigma_{3r}\,\epsilon_{1r}}{2\,\sigma_{3r}\,\epsilon_{3r} - \epsilon_{1r}\,(\sigma_{1r} + \sigma_{3r})} \tag{6.8}$$

where: M_r resilient modulus;

ν Poisson's ratio;

σ_{1r}, σ_{3r} major and minor repeated principal stresses;

ϵ_{1r}, ϵ_{3r} major and minor resilient principal strains.

In the case of constant confining pressure (CCP) tests (σ_{3r} = 0 kPa) Equations *6.7* and *6.8* reduce to:

$$M_r = \frac{\sigma_{1r}}{\epsilon_{1r}} \tag{6.9}$$

$$\nu_r = -\frac{\epsilon_{3r}}{\epsilon_{1r}} \tag{6.10}$$

As far as modelling is concerned, the formulation in terms of bulk modulus and shear modulus has been favoured in preference to the Young's modulus and Poisson's ratio type of stress-strain relationship [Boyce, 1976; Pappin, 1979; Allaart, 1992]. In this case they are expressed in the following form:

$$\epsilon_{vr} = \Delta \left\{ \frac{p}{K\ (p,\ q)} \right\} \tag{6.11}$$

$$\epsilon_{sr} = \Delta \left\{ \frac{q}{3\ G\ (p,\ q)} \right\} \tag{6.12}$$

where: ϵ_{vr} resilient volumetric strain;

ϵ_{sr} resilient shear strain;

K bulk modulus;

G shear modulus;

p mean normal stress;

q deviatoric stress;

Δ increment.

The following parameters volumetric and shear resilient strains, respectively, have to be defined since they are used in most of the models described below. The last term of the following equations is valid for the axisymetric conditions of the triaxial test:

$$\epsilon_{vr} = \epsilon_{1r} + \epsilon_{2r} + \epsilon_{3r} = \epsilon_{1r} + 2\ \epsilon_{3r} \tag{6.13}$$

$$\epsilon_{sr} = \frac{\sqrt{2}}{3} \sqrt{(\epsilon_{1r}-\epsilon_{2r})^2+(\epsilon_{2r}-\epsilon_{3r})^2+(\epsilon_{3r}-\epsilon_{1r})^2} = \frac{2}{3}\ (\epsilon_{1r}-\epsilon_{3r}) \tag{6.14}$$

where ϵ_{1r}, ϵ_{2r}, ϵ_{3r} are the major, intermediate and minor resilient principal strains respectively;

and the stress invariants usually considered for triaxial test conditions are

$$p_r = \frac{1}{3}(\sigma_{1r} + \sigma_{2r} + \sigma_{3r}) = \frac{1}{3}(\sigma_{1r} + 2\,\sigma_{3r}) \qquad 6.15$$

$$q_r = \frac{1}{\sqrt{2}}\sqrt{(\sigma_{1r}-\sigma_{2r})^2+(\sigma_{2r}-\sigma_{3r})^2+(\sigma_{3r}-\sigma_{1r})^2} = \sigma_{1r}-\sigma_{3r} \qquad 6.16$$

where: p_r repeated mean normal stress;

q_r repeated deviatoric stress;

σ_{1r}, σ_{2r}, σ_{3r} major, intermediate and minor repeated principal stresses.

After modelling the material behaviour, and for materials' comparison purposes, it is possible to determine the mechanical properties, resilient modulus and Poisson's ratio, for a standard stress path using the Equations *6.7* and *6.8*. These can be expressed in terms of the stress invariants p and q as follows:

$$M_r = \frac{9\,p_r\,q_r}{9\,p_r\,\epsilon_{sr} + q_r\,\epsilon_{vr}} \qquad 6.17$$

$$\nu = \frac{4.5\,p_r\,\epsilon_{sr} - q_r\,\epsilon_{vr}}{9\,p_r\,\epsilon_{sr} + q_r\,\epsilon_{vr}} \qquad 6.18$$

This method is adopted by other researchers and it is the base of a classification system for granular materials proposed in France [Paute, Hornych & Benaben, 1996]. The classification is based on two parameters obtained from the results of repeated load triaxial tests: characteristic resilient modulus (M_{rc}) and characteristic permanent strain (A_{1c}). Both are determined for a standard stress path represented by p_r = 300 kPa and q_r = 600 kPa, from zero initial stress conditions (Table 6.3).

In Section 6.4.2 it was noticed that the standard stress path was too severe for some of the unbound secondary materials studied and it was not possible to apply it without experiencing high levels of permanent deformations or even failure in the conditioning phase. Conversely, for some treated mixtures the same stress path was very far from failure levels. Hence it was decided to compute the same characteristic values so as to allow a fair comparison of materials and the use of the French classification system, but to derive these from modelled, rather than measured, response on the standard stress path. This may result in a false extrapolation of resilient data beyond the range of actual resilient behaviour. Thus, it is necessary to emphasise that the characteristic values should be used for comparison and classification purposes and not for pavement design.

6.6.2. Review of existing models for granular materials

The principal models used for analysing granular materials behaviour are described below. Whenever possible, the equations are presented in a non-dimensional form and with uniform symbology between models in order to simplify their comparison. This may cause the model to appear in a slightly different form in relation to its original form proposed by their authors.

K - θ model [Brown & Pell, 1967; Hicks & Monismith, 1971]

$$M_r = p_a\, K_1 \left(\frac{3\, p_{\max}}{p_a} \right)^{K_2} \qquad 6.19$$

where: M_r resilient modulus;

p_a atmospheric pressure, $p_a = 100$ kPa;

p_{max} maximum mean normal stress experienced during a stress excursion;

K_1, K_2 material constants.

The value 3 in the equation above can be removed by considering the following change:

$$K_{1\,(new)} = K_{1\,(old)} \times 3^{K_2}$$

However, in the original model $\theta = 3\ p_{max}$ was considered and this was also adopted in this study.

Uzan model [Uzan, 1985]

$$M_r = p_a \ K_1 \left(\frac{3\ p_{\max}}{p_a} \right)^{K_2} \left(\frac{q_{\max}}{p_a} \right)^{K_3} \qquad 6.21$$

where: q_{max} deviatoric stress; and

K_1, K_2, K_3 material constant.

When compared to K-θ model, Uzan model has the advantage of taking into consideration the influence of the deviatoric stress on the resilient properties. However, these models can only be used to analyse results of constant confining stress tests in which the minimum deviatoric stress is zero. They also assume constant Poisson's ratio which has been experimentally demonstrated not to be correct. As a result these models are used for the prediction of axial strains but not for radial strains.

Boyce model [Boyce, 1980]

This is a non-linear elastic model with three material constants. The volumetric and shear strains are expressed as follows:

$$\epsilon_{vr} = \Delta \left\{ p_a^{\,1-m} \;\; p^{\,m} \;\; \frac{1}{K_a} \left[1 - \beta \left(\frac{q}{p} \right)^2 \right] \right\} \qquad 6.22$$

$$\epsilon_{sr} = \Delta \left\{ p_a^{\,1-m} \; p^{\,m} \; \frac{1}{3\; G_a} \left(\frac{q}{p} \right) \right\} \qquad 6.23$$

where: K_a, G_a, m — independent material constants (K_a, G_a in stress dimensions, kPa);
β — material constant dependent on K_a, G_a and m.

This model was developed using the theory of reciprocity (or principle of the conservation of the energy)

$$\frac{\partial \epsilon_v}{\partial q} = \frac{\partial \epsilon_s}{\partial p} \qquad 6.24$$

As has been demonstrated (Figure 6.18 and Sections 6.2 and 6.5) energy is lost, due to hysteresis, so this theory is actually invalid. It can be applied to Equations *6.22* and *6.23* giving the following relationship as one of several solutions:

$$\beta = \frac{K_a\;(1-m)}{6\;G_a} \qquad 6.25$$

The principal advantage of Boyce's model is the use of just three constants. It is included in the European Standard Draft [CEN, 1995] for modelling triaxial results from variable confining pressure tests.

Pappin model [Pappin, 1979; Pappin & Brown, 1980; Brown & Pappin, 1981; Brown & Pappin, 1985]

The full version of this model, also known as 'contour model' and Pappin & Brown model, incorporates stress path length dependency for the shear strain and no longer assumes that reciprocity holds true. It consists of a six-parameter model expressed by the following equations:

$$\epsilon_{vr} = \Delta \left\{ p_a^{1-m} \; p^{m} \; \frac{1}{K_a} \left[1 - \beta \left(\frac{q}{p} \right)^2 \right] \right\} \qquad 6.26$$

$$\epsilon_{sr} = \Delta \left\{ p_a \; \frac{1}{3 \; G_a} \left(\frac{q}{p + p^*} \right) \right\} \left(\frac{\sqrt{p_r^2 + q_r^2}}{(p_1 + p_2) \; / \; 2} \right)^n \qquad 6.27$$

where: p_r repeated mean normal stress;

q_r repeated deviatoric stress;

p_1 initial mean normal stress;

p_2 final mean normal stress;

K_a, G_a, m, n, β, p^* independent material constants.

Note, in particular, the different stress-dependency in the two relationships, with $m \neq n$, and the introduction of an independent β which means that reciprocity is not valid. The constant p^* resulted from re-writing the original equation. It is here viewed with the same meaning as in the model presented later in this chapter used for permanent deformation behaviour. Its significance is shown schematically in Figure 6.26 where it can be seen to correspond to a translation of the deviatoric stress axis and, thus, may be considered a measure of suction. Assuming zero apparent cohesion (in effective stress terms) for a granular material $p + p^*$ equals the effective stress p'. This line of argument leads to the requirement that $p + p^*$ should replace p in both relationships and not only in the shear strain equation.

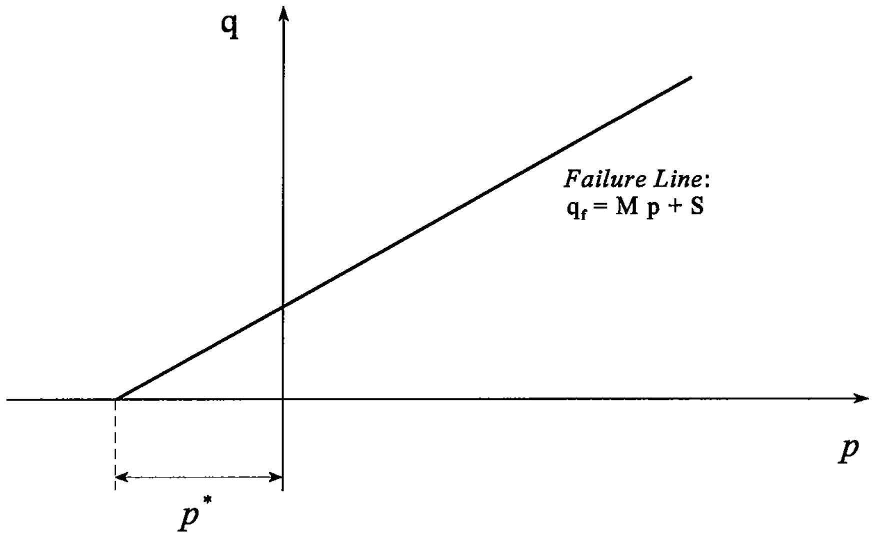

Figure 6.26 Significance of parameter p^*.

Mayhew model [Mayhew, 1983]

Mayhew found that considering stress path length dependency, the shear strains could be predicted more accurately for stress paths far from zero stress but there was no improvement for stress paths starting from near zero stress [Mayhew, 1983]. As this was believed to be the case in granular pavement layers, he proposed the following non-reciprocal five-parameter model:

$$\epsilon_{vr} = \Delta \left\{ p_a^{1-m}\ p^m\ \frac{1}{K_a} \left[1 - \beta \left(\frac{q}{p} \right)^2 \right] \right\} \qquad 6.28$$

$$\epsilon_{sr} = \Delta \left\{ p_a^{1-n}\ p^n\ \frac{1}{3\ G_a} \left(\frac{q}{p} \right) \right\} \qquad 6.29$$

where K_a, G_a, m, n, β are independent material constants.

Jouve & Elhannani model [Jouve & Elhannani, 1994]

The model first proposed by Elhannani is based on Boyce's model, but it entails other possibilities for granular materials behaviour, namely:

- the theory of reciprocity may not always be valid and, thus, the existence of an elastic potential may not be verified;
- the assumption of isotropy of granular materials implicit in Boyce's model may not be valid and, thus, the possibility of isotropic or of cross-anisotropic materials is included.

As a result of these hypotheses there are four versions for this model presented below, according to whether the material is isotropic ($\xi = 0$) or orthotropic ($\xi \neq 0$), and whether it satisfies the theory of reciprocity or not.

I) Isotropic with potential (reciprocity valid)
As for the Boyce model - Equations *6.22*, *6.23* and *6.25*.

II) Isotropic without potential (reciprocity invalid): Equations *6.22* and *6.23* hold but *6.25* is replaced with β now being an independent material constant, increasing the number of constants to four: K_a, G_a, m and β.

III) Orthotropic with potential (reciprocity valid)

$$\epsilon_{vr} = \Delta \left\{ p_a^{\;1-m} \; p^{\,m} \left[\frac{1}{K_a} - \frac{\beta}{K_a} \left(\frac{q}{p} \right)^2 - m \, \xi \left(\frac{q}{p} \right) \right] \right\} \qquad 6.30$$

$$\epsilon_{sr} = \Delta \left\{ p_a^{\;1-m} \; p^{\,m} \left[\frac{1}{3 \; G_a} \left(\frac{q}{p} \right) - \xi \right] \right\} \qquad 6.31$$

where K_a, G_a, m, ξ are independent material constant and β is defined by Equation *6.25*.

IV) Orthotropic without potential (reciprocity invalid): same equations as case III) with β being an independent material constant. Thus the total number of constants is five: K_a, G_a, m, β and ξ.

This model has the same advantages and disadvantages as the Boyce model, with the difference that it introduces anisotropy, but is more difficult to use in practice.

Karasahin model [Karasahin, 1993]

This model has 2 versions: the first is an isotropic model for the prediction of results from constant confining pressure tests (CCP) and in this case the model includes 7 material constants; the second version incorporates anisotropy and it is used in particular for variable confining pressure tests (VCP), cycling both axial and radial stresses. This version has 13 constants in total. The equations of this model may be found in the bibliography [Karasahin, 1993].

The main advantage of Karasahin's model is the incorporation of materials' anisotropy. However, the high number of constants necessary for its characterisation tend to restrict its application in practice. Some problems in modelling radial strains still remain.

6.6.3. Brief comparison of models

From the re-written equations it is evident that Boyce, Pappin, Mayhew and Jouve & Elhannani models are of the same form. Table 6.7 summarises their capabilities and limitations.

Table 6.7 Comparison of models for resilient behaviour of granular materials.

		Model Type		**Reciprocity Invalid (1)**	**Non-linked non-linearity (2)**	**Orthotropic**	**Stress path length dependency**	**Other (p^*)**
		Curve Fitting	Boyce					
K - θ		✓		✓				
Uzan		✓		✓				
Pappin			✓	✓	✓		✓	✓
Mayhew			✓	✓	✓			
Jouve & Elhannani	I (Boyce)		✓					
	II		✓	✓				
	III		✓			✓		
	IV		✓	✓		✓		

Notes: (1) β independent constant
(2) magnitude of power in ϵ_v and ϵ_s equations is different ($m \neq n$)

Jouve & Elhannani versions III and IV, are considered to be of less interest than the other models because of the difficulty in explaining the meaning of the parameter ξ in the case of non-elastic behaviour [Paute, Dawson & Galjaard, 1996]. Additionally, for the application to treated secondary materials the consideration of anisotropy is less important. This is demonstrated by the results presented in Chapter 7 where specimens of stabilised fly ash were compacted following the same procedures as for triaxial specimens, dry-cut into diametral specimens and tested in the perpendicular directions in relation to the triaxial tests. The resilient moduli obtained were similar.

It has been found in previous studies that Boyce and Elhannani models are able to predict shear strains, but poor predictions were observed for the volumetric strains [Karasahin, 1993]. This may suggest better performances of those models which do not respect the theory of reciprocity. Hence, with β as an independent variable the models are more general and encompass the possibility of complying with that theory.

Furthermore, the consideration of the exponents $m = n$ as proposed by Boyce and Jouve & Elhannani does not seem to be realistic. In this research, extensive testing was carried out and the results obtained for those exponents were invariably different for the volumetric and shear strains. To contemplate the possibility of m $\neq$ n seems for this reason not only advisable but logical since this will also cover the possibility of m being equal to n.

6.6.4. Models and modified models considered

As a result of the considerations exposed in the last section, the hypotheses considered for evaluation to model the behaviour of secondary materials under repeated loading were:

- Pappin volumetric model, Equation *6.26*;
- Pappin shear model, Equation *6.27*;
- Mayhew shear model, Equation *6.29* (Mayhew's volumetric model is equal to Pappin's);
- Modified Mayhew volumetric model, incorporating the parameter p^* ($p+p^*$ replaces p)

$$\epsilon_{vr} = \Delta \left\{ p_a^{1-m} \, (p+p^*)^m \, \frac{1}{K_a} \left[1 - \beta \left(\frac{q}{p+p^*} \right)^2 \right] \right\} \qquad 6.32$$

- Modified Mayhew shear model, incorporating the parameter p^*

$$\epsilon_{sr} = \Delta \left\{ p_a^{1-n} \, (p+p^*)^n \, \frac{1}{3\,G_a} \left(\frac{q}{p+p^*} \right) \right\} \qquad 6.33$$

- Modified Mayhew shear model, incorporating the parameter p^* and stress path length dependency

$$\epsilon_{sr} = \Delta \left\{ p_a^{\,1-n} \, (p+p^{*})^{n} \, \frac{1}{3\,G_a} \left(\frac{q}{p+p^{*}} \right) \right\} \left(\frac{\sqrt{p_r^{\,2} + q_r^{\,2}}}{(p_1+p_2)/2 \, + \, p^{*}} \right)^{n} \qquad 6.34$$

In the last equation it would be possible to separate the degree of effect of stress path length dependency (indicated by the last n) from that of non-linearity (represented by the first two 'n's). Yet, that would imply the introduction of another material constant and the results show that this equation is sufficiently accurate for unbound materials.

6.6.5. Modelling results

The results of this evaluation are presented in Tables 6.8 to 6.10 and some examples representative of the performance of G-K models are shown in Figures 6.27 to 6.32, for each level of treatment considered in this research.

For the **unbound materials** the Pappin and Mayhew models give very good results (notice that Mayhew and Pappin volumetric models are expressed by the same equation). Slightly better performance is obtained in the case of unbound granite with the Pappin model.

The two modified Mayhew models incorporating the parameter p^*, and p^* with stress path length dependency, in general perform slightly better than the original model but this improvement is very small to justify the introduction of another material constant (p^*).

With respect to the results obtained with **treated materials** the situation changes. The performance of Pappin and Mayhew models clearly declines in particular for the higher level of treatment (e.g. CC2, MI4 and FA3). In this case we can notice an increased importance of the parameters p^* (in particular for the treated furnace bottom ash, BA3 and BA4) and stress path length dependency (e.g. CC4, BA3, BA4, MI1, MI4 and FA3).

The modified Mayhew model incorporating the parameter p^* and stress path length dependency performs consistently better than the others (with the exception of BA4).

Table 6.8 Constants and correlation coefficients for the hypotheses considered to model the resilient behaviour (unbound materials).

UNBOUND MATERIALS										
Material	Model	Volumetric Model					Shear Model			
		R^2	K_a	m	β	p^*	R^2	G_a	n	p^*
CC6	Pappin	0.99	69575	0.602	1.802E-1	-----	0.91	68670	0.166	9
	Mayhew						0.91	70064	0.335	-----
	Mayh+p^*	0.99	62399	0.542	1.690E-1	2	0.92	65684	0.225	4
	Mayh ++						0.93	85132	0.241	2
T1	Pappin	0.85	120909	0.642	2.995E-1	-----	0.83	551997	1.840	228
	Mayhew						0.77	110284	0.344	-----
	Mayh+p^*	0.86	105287	0.544	2.776E-1	1	0.77	105314	0.291	1
	Mayh ++						0.82	177192	0.357	0
SW1	Pappin	0.93	77096	0.395	4.928E-2	-----	0.96	76644	0.349	91
	Mayhew						0.95	91449	0.413	-----
	Mayh+p^*	0.94	13	8.1E-05	1.337E-5	21	0.95	89083	0.352	4
	Mayh ++						0.98	141756	0.382	1

KEY: Mayh + p^* Modified Mayhew model, incorporating the parameter p^*
Mayh ++ Modified Mayhew shear model, incorporating the parameter p^* and stress path length dependency
R Correlation coefficient
K_a, G_a, m, n, β, p^* Independent material constants
Unbound Specimens: CC6 China Clay Sand
T1 Granite Type 1 Sub-base material
SW1 Slate Waste

Table 6.9 Constants and correlation coefficients for the hypotheses considered to model the resilient behaviour (treatment level I).

		TREATMENT LEVEL I								
Material	Model	Volumetric Model					Shear Model			
		R^2	K_a	m	β	p^*	R^2	G_a	n	p^*
CC4	Pappin	0.83	424943	0.619	0	-----	0.88	35926	0.997	4407
c = 2 %	Mayhew						0.73	452112	0.886	-----
	Mayh+p^*	0.84	224533	0.395	0	28	0.73	437077	0.858	18
	Mayh ++						0.88	1468630	0.938	0
BA4	Pappin	0.92	173232	0.626	8.297E-2	-----	0.87	88727	0.786	189
c = 2 %	Mayhew						0.69	103912	0.640	-----
	Mayh+p^*	0.97	34	1.7E-04	3.866E-5	35	0.75	60006	0.224	74
	Mayh ++						0.84	193960	0.576	3
MI1	Pappin	0.92	205800	0.787	1.195E-1	-----	0.96	27543	0.820	1051
l = 5 %	Mayhew						0.85	113463	0.707	-----
	Mayh+p^*	0.94	42094	0.358	1.163E-1	205	0.86	93612	0.562	47
	Mayh ++						0.97	301347	0.826	3
FA2	Pappin	0.82	1270490	0.641	0	-----	0.88	0.261	0.573	1.3E+09
c = 2 %	Mayhew						0.94	2820460	1.564	-----
	Mayh+p^*	0.85	241677	0.201	0	72	0.94	4506120	1.801	89
	Mayh ++						0.95	5924800	1.183	64

KEY: Mayh + p^* Modified Mayhew model, incorporating the parameter p^*

Mayh ++ Modified Mayhew shear model, incorporating the parameter p^* and stress path length dependency

R Correlation coefficient

K_a, G_a, m, n, β, p^* Independent material constants

Treatment Level I: CC4 China Clay Sand + 2 % cement

BA4 Furnace Bottom Ash + 2 % cement

MI1 Minestone + 5 % lime

FA2 Pulverised Fuel Ash + 2 % cement

Table 6.10 Constants and correlation coefficients for the hypotheses considered to model the resilient behaviour (treatment level II).

TREATMENT LEVEL II											
Material	Model	Volumetric Model					Shear Model				
		R^2	K_a	m	β	p^*	R^2	G_a	n	p^*
CC2	Pappin	0.63	560683	0.601	0	-----	0.80	20704	0.000	5059
c = 2 %	Mayhew						0.80	1121020	1.017	-----
pfa=8%	Mayh+p^*	0.66	244849	0.337	0	35	0.80	94454	0.820	8.7E+07
	Mayh ++						0.80	0.422	0.003	2.6E+08
BA3	Pappin	0.82	269647	1.333	1.896E-1	-----	0.88	83304	0.669	246
c = 2 %	Mayhew						0.77	148131	0.764	-----
pfa=8%	Mayh+p^*	0.97	91387	0.464	1.163E-1	2	0.82	103480	0.387	20
	Mayh ++						0.89	237602	0.565	2
MI4	Pappin	0.79	2136950	0.620	3.168E-2	-----	0.81	0.374	0.607	1.2E+09
l = 3 %	Mayhew						0.89	4787740	1.835	-----
pfa=12%	Mayh+p^*	0.81	674267	0.261	2.301E-2	34	0.89	2.3E+07	2.553	201
	Mayh ++						0.96	1.2E+07	1.481	99
FA3	Pappin	0.94	1924180	0.756	5.409E-3	-----	0.73	0.645	0.353	7.1E+08
c = 5 %	Mayhew						0.81	6790680	1.864	-----
	Mayh+p^*	0.94	1924940	0.756	5.390E-3	0	0.81	7011160	1.880	5
	Mayh ++						0.90	11	1.244	1.9E+08

KEY: Mayh + p^* Modified Mayhew model, incorporating the parameter p^*
Mayh ++ Modified Mayhew shear model, incorporating the parameter p^* and stress path length dependency
R Correlation coefficient
K_a, G_a, m, n, β, p^* Independent material constants
Treatment Level II: CC2 China Clay Sand + 2 % cement + 8 % pulverised fuel ash
BA3 Furnace Bottom Ash + 2 % cement + 8 % pulverised fuel ash
MI4 Minestone + 3 % lime + 12 % pulverised fuel ash
FA3 Pulverised Fuel Ash + 5 % cement

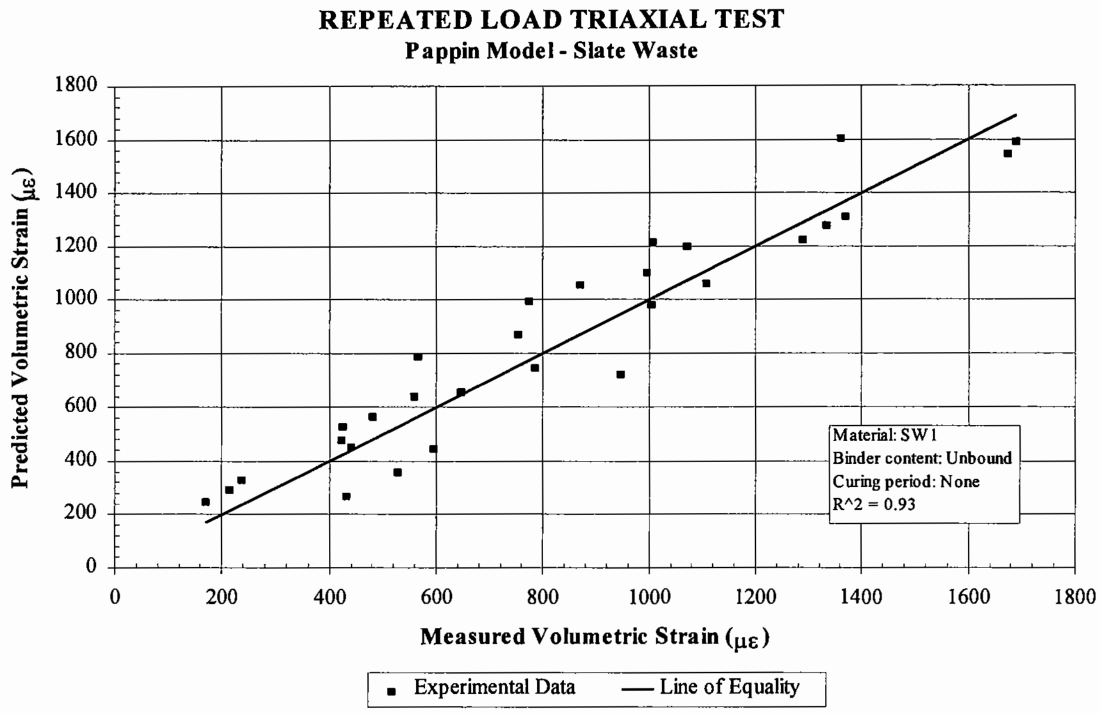

Figure 6.27 Performance of Pappin volumetric model (unbound material).

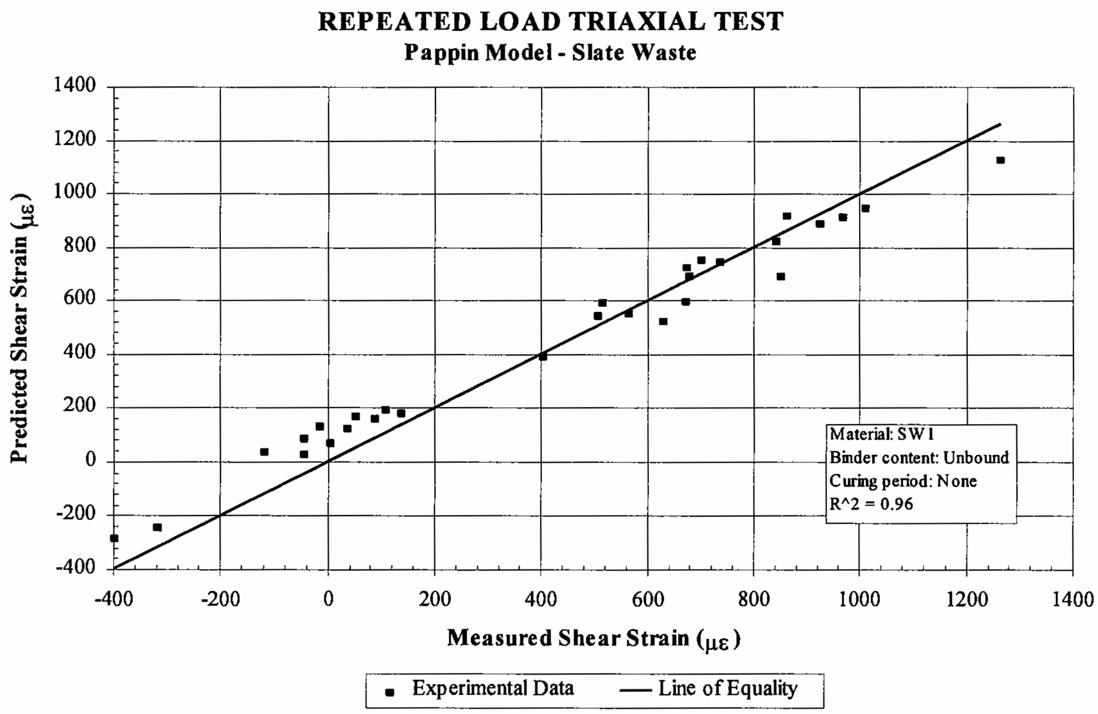

Figure 6.28 Performance of Pappin shear model (unbound material).

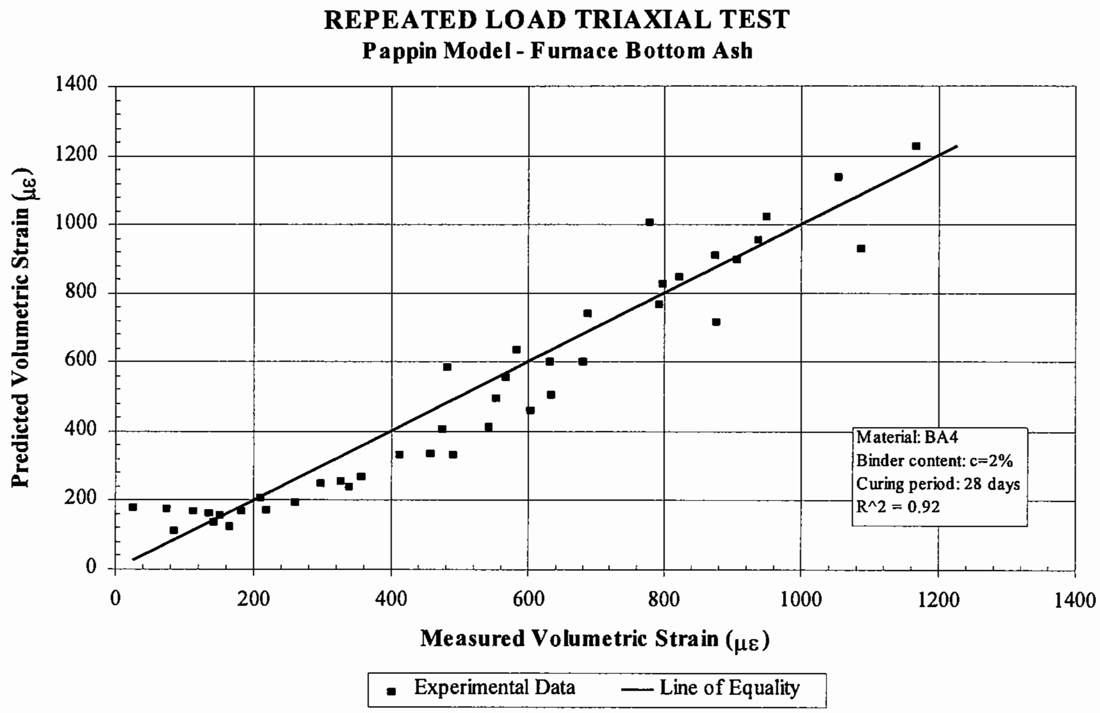

Figure 6.29 Performance of Pappin volumetric model (treatment level I).

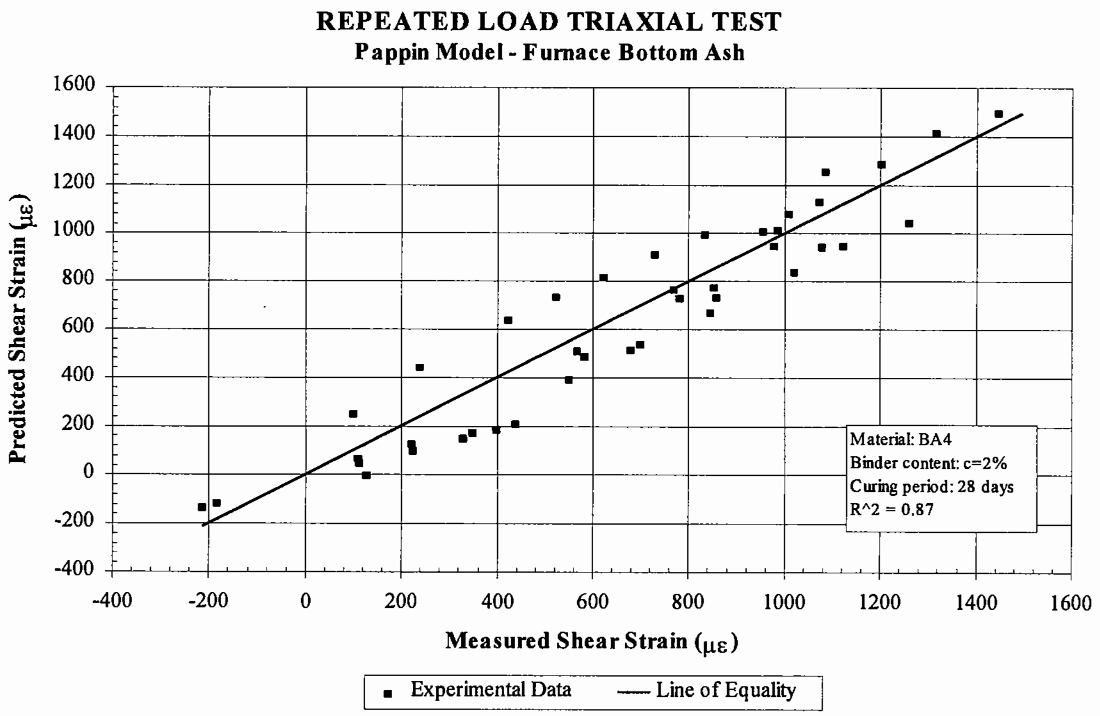

Figure 6.30 Performance of Pappin shear model (treatment level I).

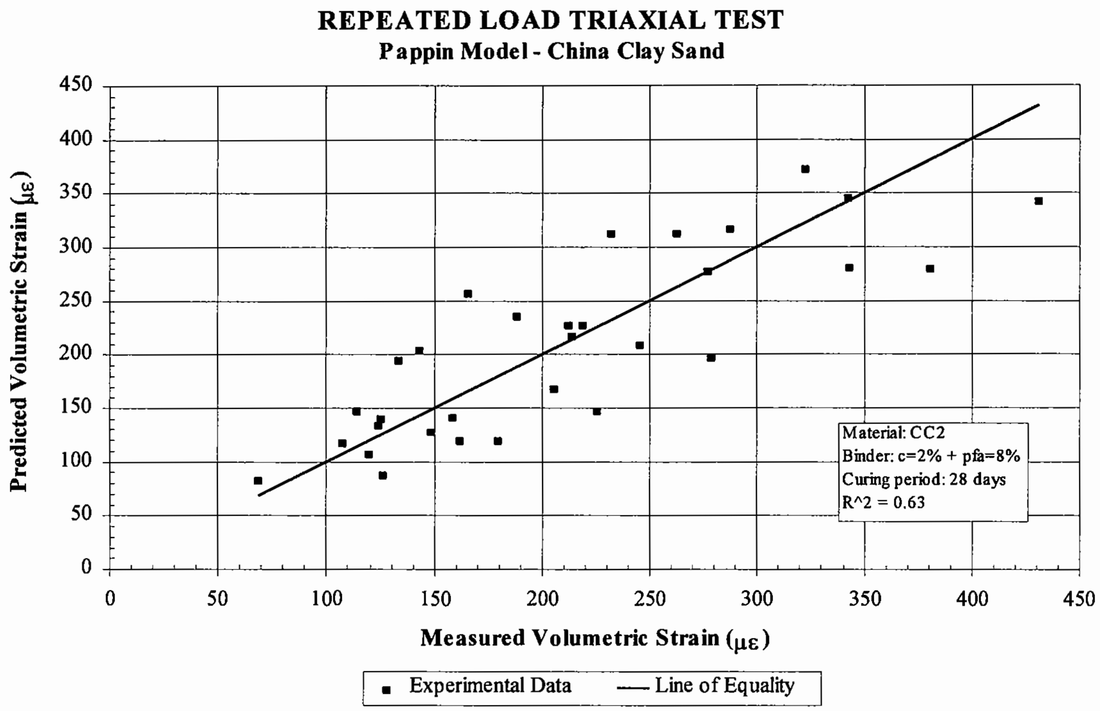

Figure 6.31 Performance of Pappin volumetric model (treatment level II).

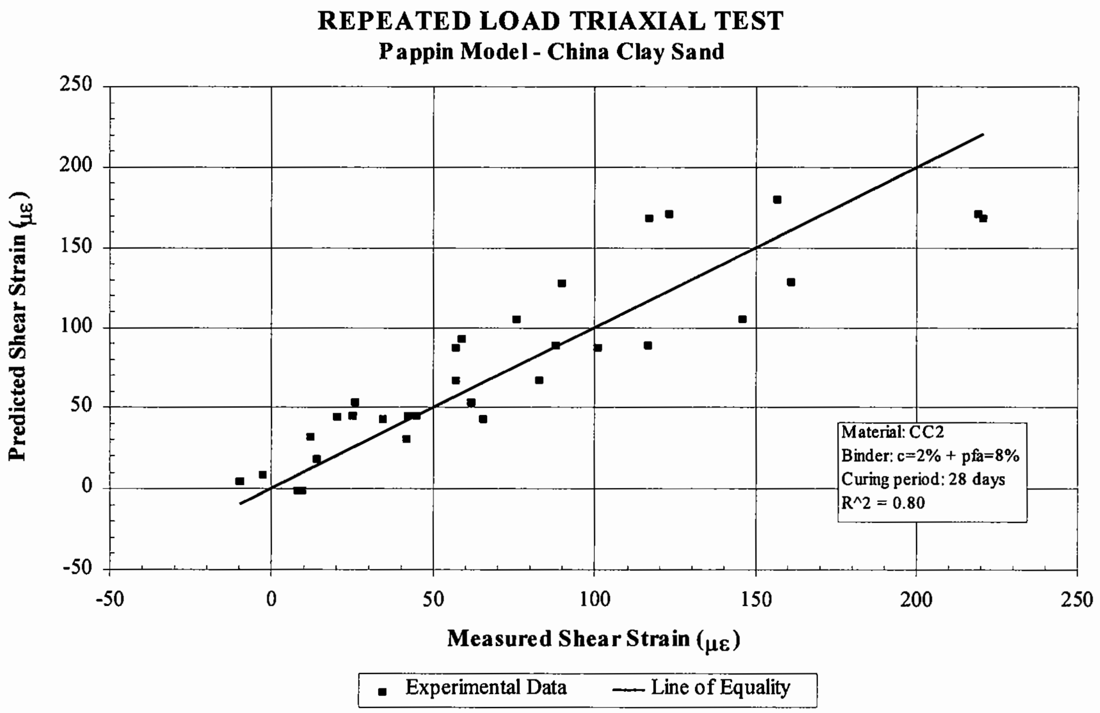

Figure 6.32 Performance of Pappin shear model (treatment level II).

However, in some cases this performance is still poor and, what is more, the constants K_a, G_a and p^* start having random values, with excessively low / high values in particular cases, not consistent with those usually associated with these variables and in sharp contrast to the significance of p^* as given in Figure 6.26.

The aspects discussed above indicate the need to find another model capable of predicting adequately the resilient behaviour of treated secondary materials. Another attempt was made considering a different family of models that predicts the results in terms of resilient modulus and Poisson's ratio (M_r-ν models). In contrast with these, the G-K models presented and analysed above model shear and volumetric strains or through Equations *6.11* and *6.12*, the bulk and shear moduli. Intuitively, it is anticipated that dilative or compressive behaviour should reduce with the treatment level, moving from a granular-type to a continuum-type behaviour. The following M_r-ν models were considered:

- Wellner model [Gleitz, 1995];
- Pezo model [Pezo, 1995];
- Uzan type of model
  Resilient modulus defined by Equation *6.21* but including Poisson's ratio evaluation using similar type of equation

$$\nu = K_4 \left(\frac{p}{p_a}\right)^{K_5} \left(\frac{q}{p_a}\right)^{K_6} \qquad 6.35$$

where K_4, K_5 and K_6 are material constants.

These models were not evaluated for unbound materials (for which they were originally developed). The objective was to find / develop a suitable model for treated materials for which the G-K models evaluated in previous sections were not satisfactory. Neither of those models was found to be appropriate for the objective in mind, not fitting the experimental results adequately. This fact was inferred from the very low correlation coefficients obtained.

6.6.6. Development of a new model for treated secondary materials

From previous analysis with G-K models the importance of considering the stress path length dependency was identified. Indeed as stated earlier, the modified Mayhew model which incorporated stress path length dependency performed consistently better than the other hypothesis (except for BA4). This aspect is traditionally considered in Pappin shear model. Nevertheless, from the analysis of Figure 6.33 it appears that the consideration of stress path length may also improve the modelling of volumetric strains.

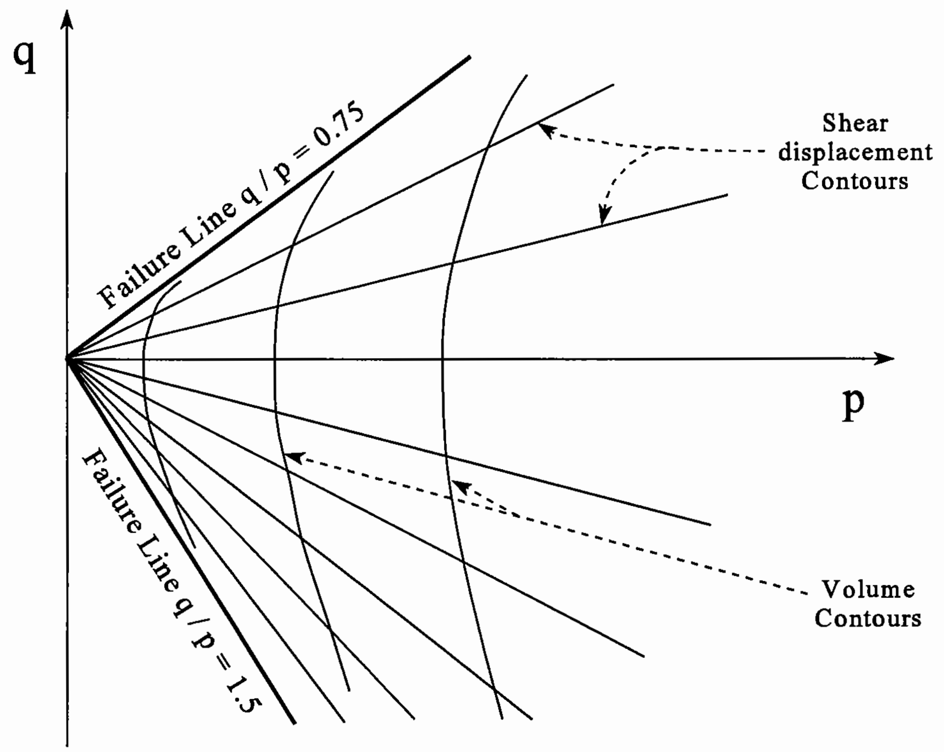

Figure 6.33 Volume and shear displacement contours [Pappin, 1979].

It is possible to contemplate the length of the stress path in a different manner by entering the repeated values of the mean normal and deviatoric stresses (p_r and q_r) into an equation. In this case, there is no need to consider the increment Δ between initial and final conditions. However, it will be necessary to include other parameters for a complete characterisation of a stress path, namely to define its position in relation to the origin of the p-q space. This may be achieved by considering the average values of p and q as shown in Figure 6.34.

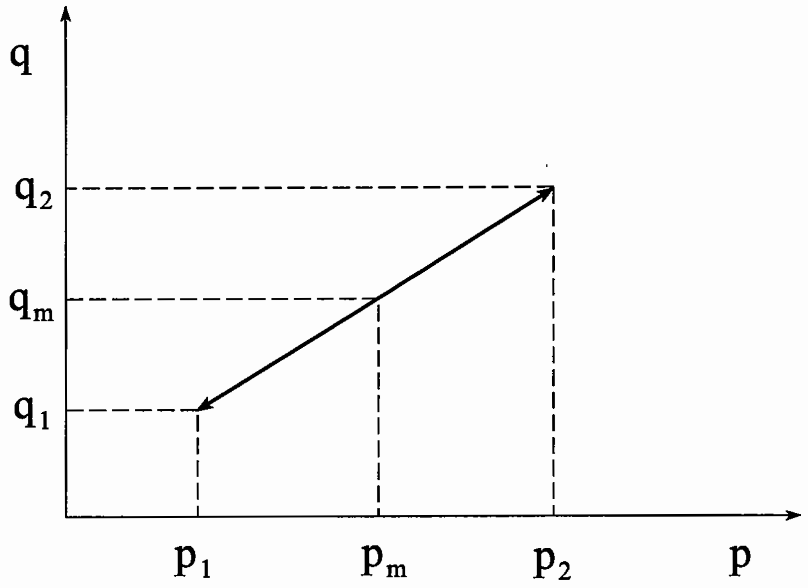

Figure 6.34 Complete definition of a stress path in a p-q space.

Based on these considerations, the following curve-fitting model is then proposed for modelling the resilient behaviour of treated granular materials under repeated load triaxial conditions:

$$\epsilon_{vr} = K_1 \left(\frac{p_r}{p_a}\right)^{K_2} \left(\frac{q_r}{p_a}\right)^{K_3} \left(\frac{q_m}{p_m}\right)^{K_4} \qquad 6.36$$

$$\epsilon_{sr} = K_5 \left(\frac{p_r}{p_a}\right)^{K_6} \left(\frac{q_r}{p_a}\right)^{K_7} \left(\frac{q_m}{p_m}\right)^{K_8} \qquad 6.37$$

where: p_m average 'mean normal stress' during one stress path application;

q_m average deviatoric stress during one stress path application;

$K_1, ..., K_8$ material constants.

Multivariate non-linear regression analysis was again used to determine the constants for this model and the results are listed in Table 6.11. Some results exemplifying the performance of the new model are presented in Figures 6.35 to 6.40, for each of the treatment levels considered.

The new model performs consistently well in terms of shear strains and, for the treated specimens, also with respect to volumetric strains, with the exception of furnace bottom ash (BA3 and BA4). When compared with the G-K models analysed previously, i.e. the Pappin, Mayhew and modified Mayhew models, the new model performs better for treated materials (except for fba). Further discussion of the model will be carried out after presentation of the experimental results in order to understand the particular features of the furnace bottom ash.

6.6.7. Stiffness properties

The mechanical properties of the mixtures are ranked in Figures 6.41 and 6.42. It is worth noting that unbound materials are tested approximately 24 hours after compaction while the results for treated materials refer to tests performed after a curing period of 28 days. The values plotted were calculated for the standard stress path (Table 6.3) using the parameters given in Table 6.11.

With respect to the resilient modulus, the results may be divided in two groups. The first including the unbound materials (namely china clay sand, granite Type 1 and slate waste) and those that despite being lightly treated still have resilient moduli of the same magnitude as for unbound materials (this is the case of furnace bottom ash, BA3 and BA4, and minestone, MI1). The second group exhibits considerably higher values which are characteristic of treated materials.

For the furnace bottom ash, stabilisation goes some way to improve the resilient modulus of approximately 230 MPa that may be expected when unbound [Dawson & Nunes, 1993], bringing the material with just 2 % of cement to a level equivalent to other unbound materials such as china clay sand and slate waste. The heavier treatment with 2 % of cement and 8 % of pulverised fuel ash continues improving the performance of the material. Yet that enhancement is relatively small and it is questionable, in economic terms, whether or not it is worth including an extra 8 % of fly ash taking into consideration the benefits (for comparative purposes see the difference in results obtained for china clay sand with 2 % cement, CC4, and with an extra 8 % of fly ash, CC2).

Table 6.11 Constants and correlation coefficients for the new model for treated secondary materials.

Material	Volumetric Model					Shear Model				
	R^2	K_1	K_2	K_3	K_4	R^2	K_5	K_6	K_7	K_8
	UNBOUND									
CC6	0.70	3.527E-4	1.626	-0.232	-0.000	0.85	1.983E-4	-0.277	1.118	0.970
T1	0.70	8.599E-5	8.181	-7.162	4.771	0.87	6.334E-5	-2.132	3.126	-0.416
SW1	0.72	6.037E-4	0.569	-0.004	-0.292	0.92	2.190E-4	-0.003	0.750	0.940
	TREATMENT LEVEL I									
CC4	0.98	2.056E-4	0.543	0.125	0.031	0.93	8.765E-5	0.576	0.278	0.833
BA4	0.61	3.477E-4	0.842	-0.068	-0.032	0.91	3.578E-4	0.253	0.268	0.922
MI1	0.97	3.415E-4	0.992	-0.030	-0.297	0.96	2.663E-4	0.298	0.583	0.775
FA2	0.89	6.717E-5	0.624	0.053	0.038	0.97	1.343E-5	0.655	0.790	0.388
	TREATMENT LEVEL II									
CC2	0.88	1.489E-4	0.504	0.144	0.031	0.92	1.679E-5	-2.850	4.271	-3.298
BA3	0.73	3.242E-4	1.223	-0.094	-0.615	0.84	1.866E-4	0.009	0.660	0.964
MI4	0.85	3.724E-5	0.703	0.036	-0.091	0.97	4.422E-6	0.637	1.321	0.555
FA3	0.91	3.755E-5	0.996	0.018	0.051	0.94	1.340E-6	-0.221	2.559	-0.023

KEY: R Correlation coefficient
$K_1, ..., K_8$ Independent material constants
Unbound Specimens: CC6 China Clay Sand
T1 Granite Type 1 Sub-base material
SW1 Slate Waste
Treatment Level I: CC4 China Clay Sand + 2 % cement
BA4 Furnace Bottom Ash + 2 % cement
MI1 Minestone + 5 % lime
FA2 Pulverised Fuel Ash + 2 % cement
Treatment Level II: CC2 China Clay Sand + 2 % cement + 8 % pulverised fuel ash
BA3 Furnace Bottom Ash + 2 % cement + 8 % pulverised fuel ash
MI4 Minestone + 3 % lime + 12 % pulverised fuel ash
FA3 Pulverised Fuel Ash + 5 % cement

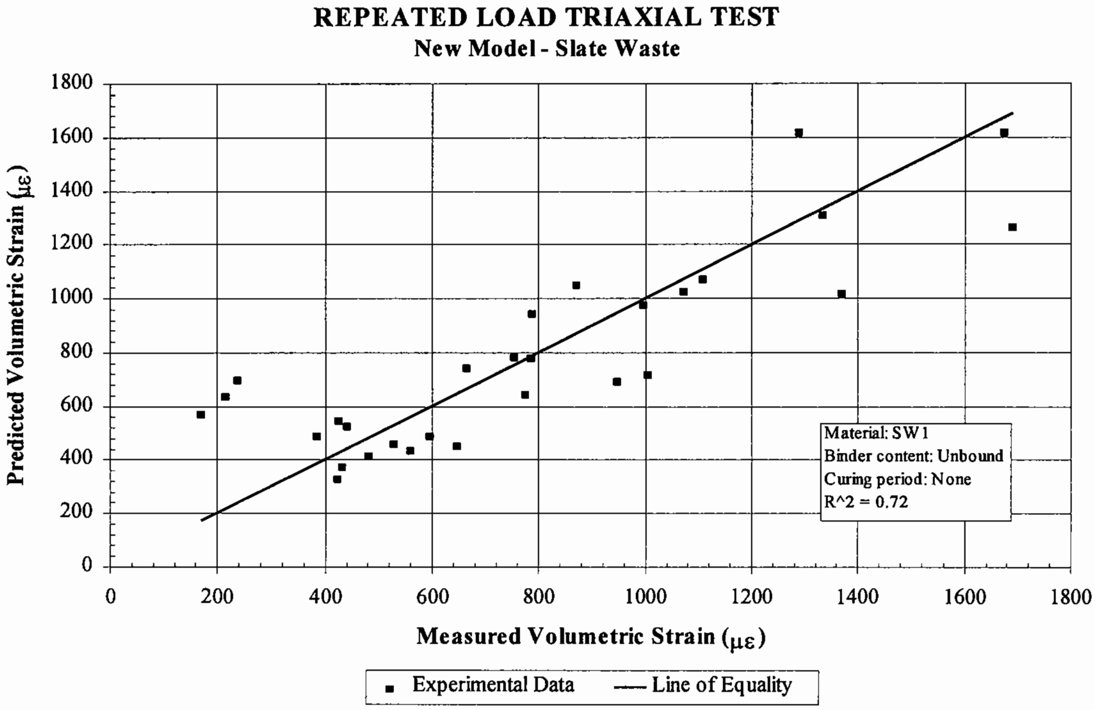

Figure 6.35 Performance of the new volumetric model (unbound material).

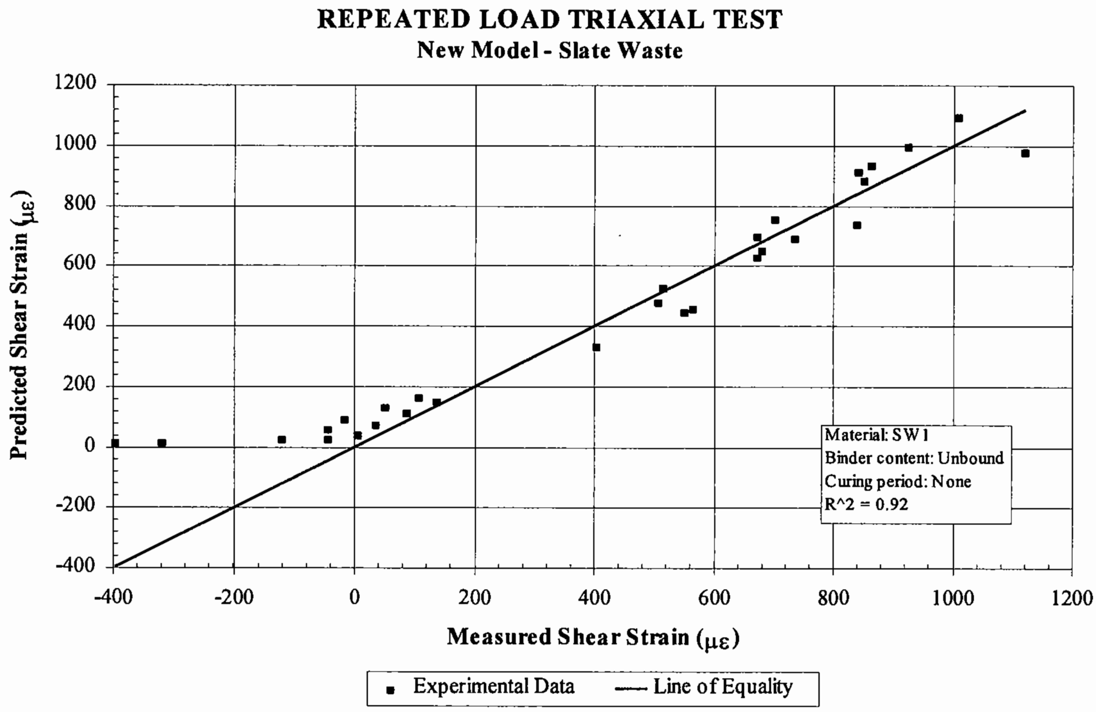

Figure 6.36 Performance of the new shear model (unbound material).

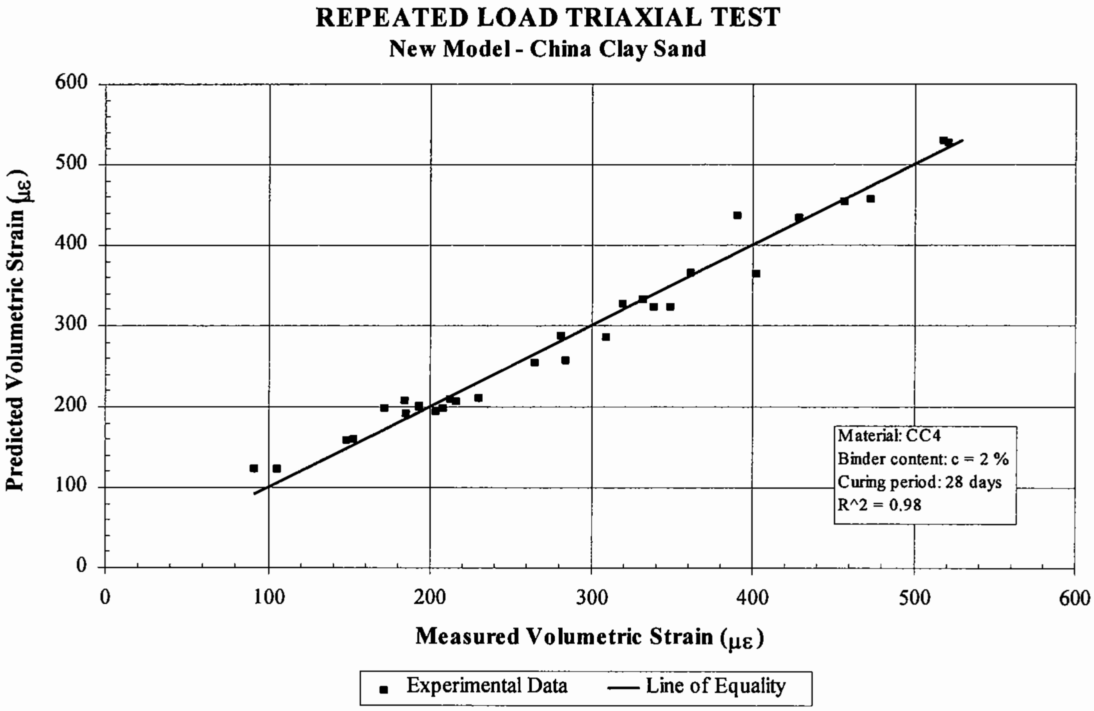

Figure 6.37 Performance of the new volumetric model (treatment level I).

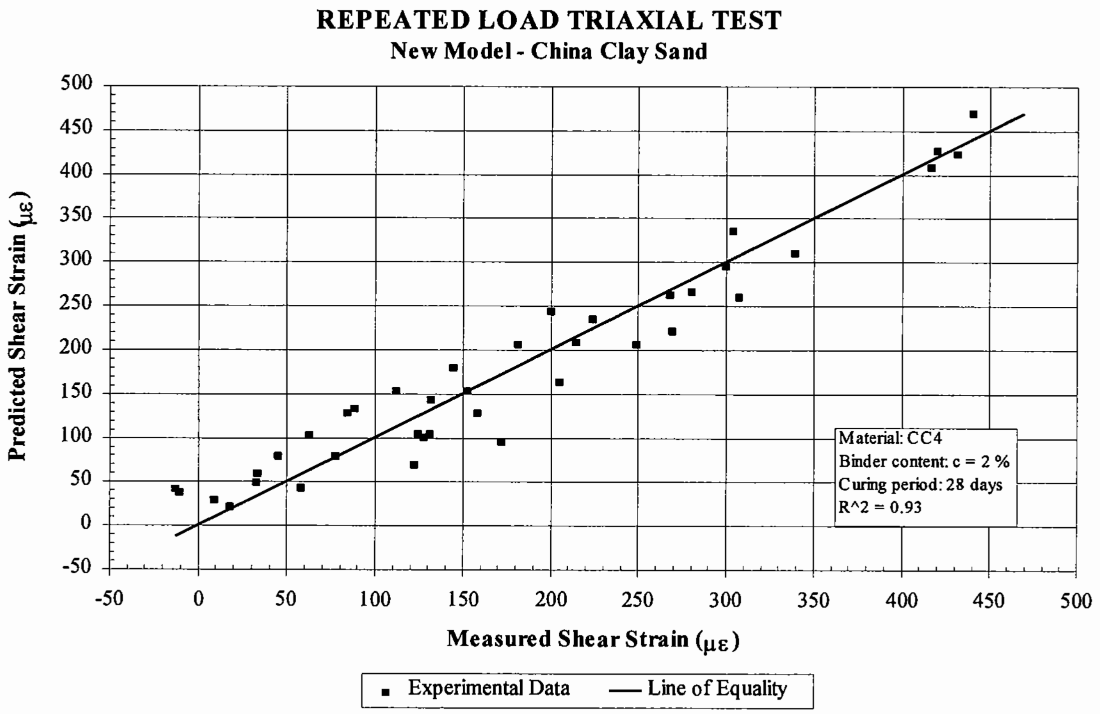

Figure 6.38 Performance of the new shear model (treatment level I).

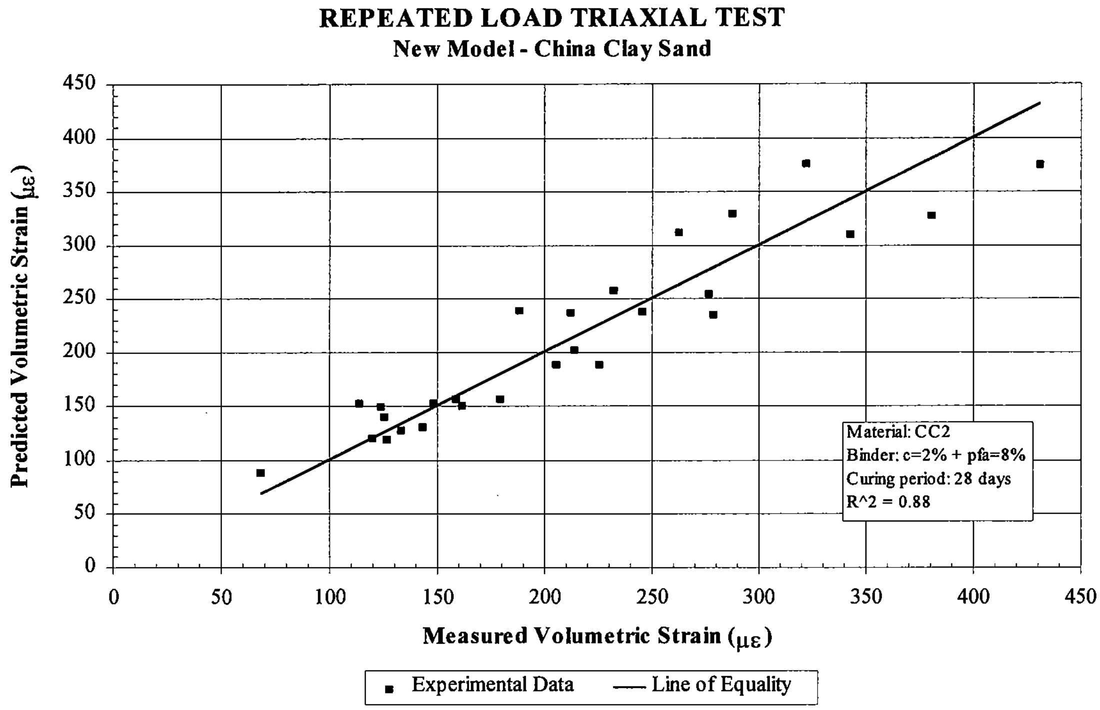

Figure 6.39 Performance of the new volumetric model (treatment level II).

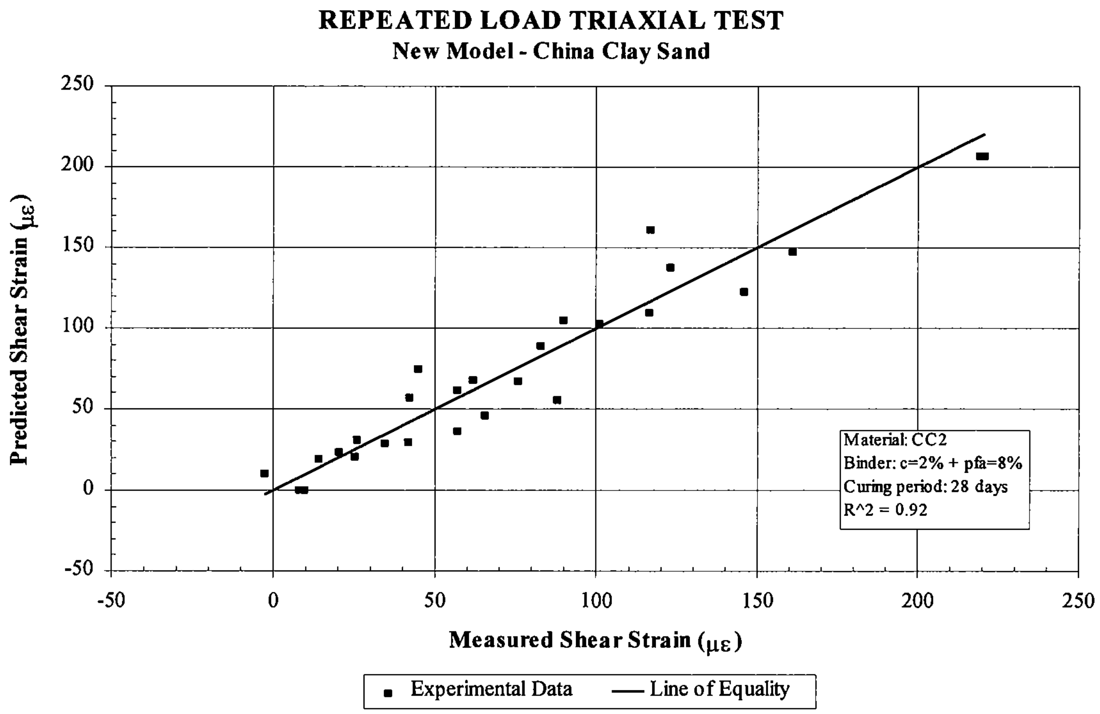

Figure 6.40 Performance of the new shear model (treatment level II).

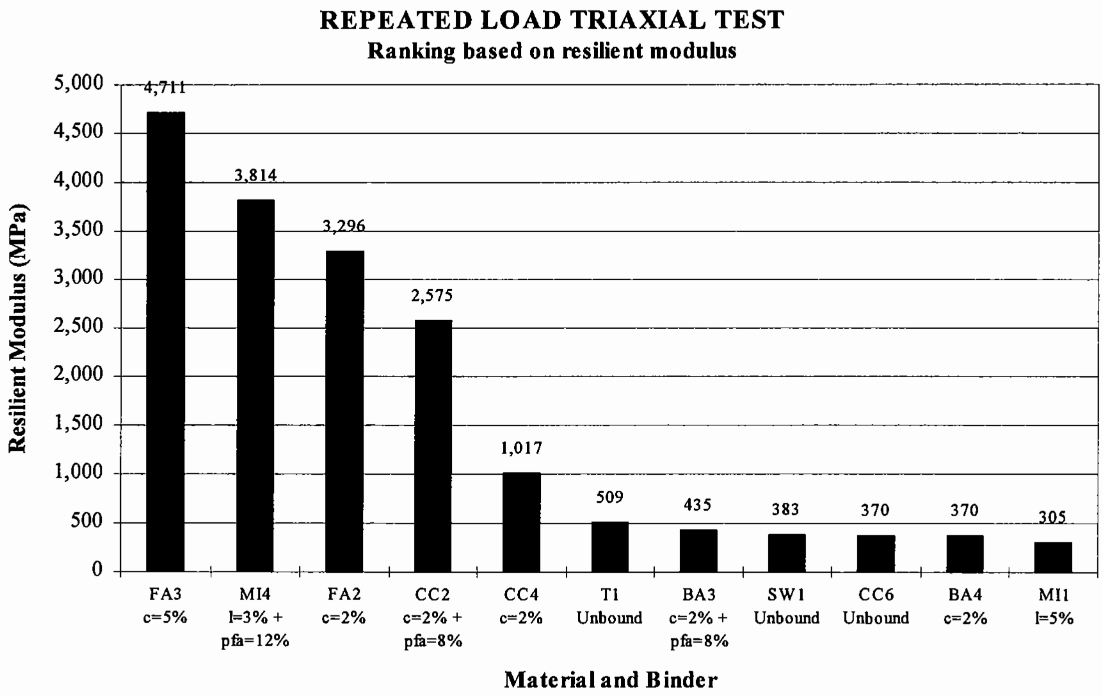

Figure 6.41 Resilient modulus results.

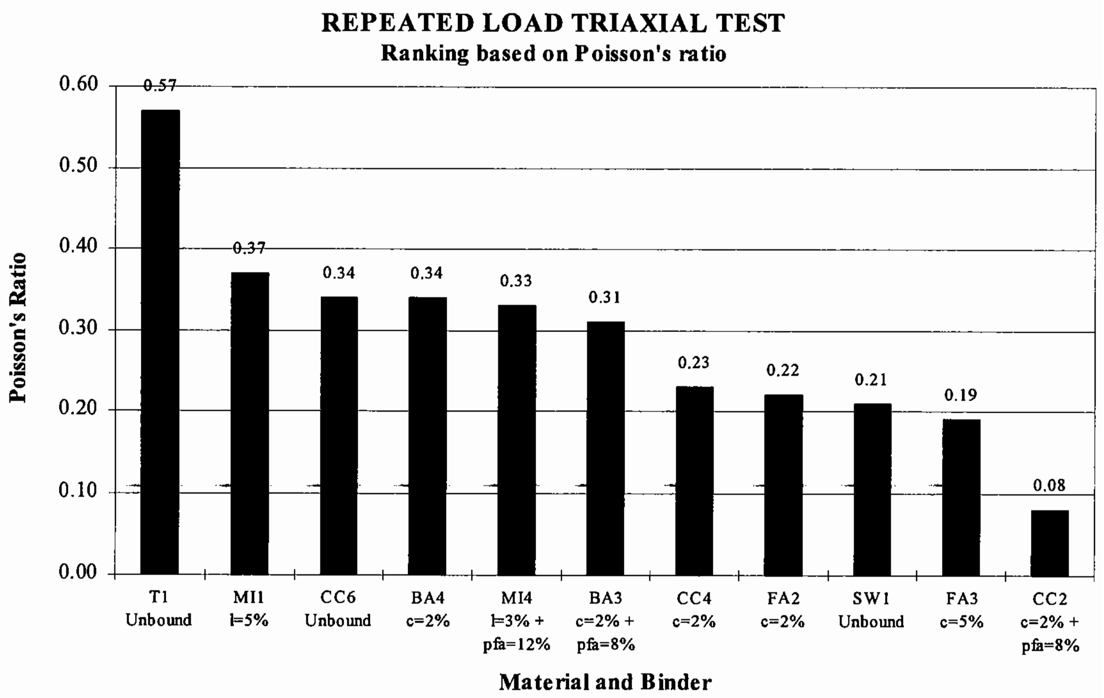

Figure 6.42 Poisson's ratio results.

The stabilisation of minestone with 5 % of lime still gives a performance inferior to the other unbound materials which seems to indicate that this particular minestone studied does not possess pozzolanic properties nor is its mineralogy beneficially altered by the lime. This aspect will be further investigated in Chapter 7 when different curing periods of up to 360 days are considered. The lime certainly improves the handling of the material before and during compaction but this result could easily be achieved with lower percentage of lime [Correia, 1990]. In summary, it may be said that this mixture is not very successful when compared with the performance of conventional unbound materials but it brings the material to acceptable levels, a fact that is in sharp contrast with its totally unacceptable performance when unbound. A heavier stabilisation of 3 % of lime and 12 % of fly ash, in MI4, produces an incomparably better result taking much greater advantage of the binder used.

Regarding Poisson's ratio, the same material groups as for resilient modulus can be broadly identified but in this case several materials fall outside the range of what might be expected. Let us consider first the unbound materials and those with a resilient modulus of a similar magnitude (BA3, BA4 and MI1). The typical Poisson's ratio for this group ranges from 0.30 to 0.40, with the exceptions of granite ($\nu = 0.57$) and slate waste ($\nu = 0.21$). The result obtained for granite seems high but it was confirmed by the value obtained during the permanent deformation test and so no reason to exclude it was found. It is also in line with previous researchers' data [Sweere, 1990]. Both results may be due to the form of the particles. On the one hand, the granite is crushed in plants to bring the rock to the gradings normally used in road construction. The particles are blocky and sub-angular and, under loading, the rotation of a particle will easily disturb others in the vicinity giving rise to resilient dilation. On the other hand, due to their high flakiness the particles of slate waste will be predominantly in the horizontal position within the compacted material, thus restraining lateral deformations.

For the group of treated materials, the Poisson's ratio ranges from approximately 0.10 to 0.25. In this group the exception is minestone (MI4) with a higher Poisson's ratio ($\nu = 0.33$). This wide range of values obtained experimentally emphasises the need to characterise pavement materials both in terms of resilient modulus and Poisson's ratio, in opposition to what is suggested in some resilient behaviour models, such as K-θ and

Uzan, in which Poisson's ratio is estimated and assumed to be constant.

By comparing the results of the preliminary test on slate waste (SW0) and those in the main testing programme (SW1), it is possible to draw some conclusions on the influence of the material density on the resilient properties of granular materials. Thom & Brown [1988] showed that the degree of compaction does not affect resilient properties significantly. Yet, a tendency for the increase of the stiffness as the level of density increases was obtained experimentally. Boyce [1976] performed an extensive bibliographic review on factors affecting the resilient behaviour of granular materials and concluded that the density can affect resilient properties up to 50 % and that similar behaviour is expected from different materials when compacted at the same relative density.

The resilient modulus obtained in the preliminary test (84 % of maximum dry density approximately) was of 298 MPa for the standard stress path. This compares with 383 MPa when the material was compacted at optimum values which corresponds to an increase of about 30 %. It appears from these limited results that density has a significant influence on the resilient properties.

6.6.8. Implication of results on resilient behaviour modelling

On the basis of the results presented in the last section it was possible to identify two groups: one in which the materials are characterised by having an 'unbound type of behaviour' and the second in which the materials exhibit a 'treated type of behaviour'. Treated furnace bottom ash was identified as part of the first group a fact that explains the better performance of the G-K models in comparison to the new model. This aspect is corroborated by Figure 6.22 which shows an hysteresis loop for this material, not significantly different to that which may be expected for an unbound material (Figure 6.18).

Accordingly, it is possible to attempt the definition of boundaries of validity for the models studied:

unbound type of behaviour

mechanical properties (indicative values)

resilient modulus < 700 - 800 MPa

Poisson's ratio > 0.25 (with some exceptions for specific flaky materials)

triaxial strength (σ_3 = 0 kPa) < 1000 kPa

model

Pappin model (Equations *6.26* and *6.27*)

or modified Mayhew (Equations *6.32* and *6.34*)

treated type of behaviour

mechanical properties (indicative values)

resilient modulus > 700 - 800 MPa

Poisson's ratio < 0.25 (with some exceptions for specific materials)

triaxial strength (σ_3 = 0 kPa) > 1000 kPa

model

new model for treated granular materials (Equations *6.36* and *6.37*)

6.7. Permanent deformation behaviour of secondary materials

6.7.1. Introduction

The study of the permanent deformation behaviour under repeated loading is another important part of the characterisation of unbound materials for road construction purposes. The accumulation of permanent deformations within one layer of unbound material may result in failure of the pavement due to rutting, an aspect that is generally taken into consideration during design by limiting the maximum vertical stress on the top of unbound layers.

The number of studies for modelling the behaviour of granular materials in terms of permanent deformations is reduced when compared with those for resilient behaviour. This is reflected in the present state-of-the-art in this domain which remains essentially at an empirical level. Among the factors contributing to this situation, the following are noted:

- for the study of permanent deformation it is necessary to test a new specimen for each stress path applied, while for resilient behaviour all stress paths may be applied to the same specimen. Yet, only four stress paths are usually considered for permanent deformations whilst that number for resilient behaviour is above thirty;
- the number of load applications necessary to study permanent deformation is higher and so the tests are longer. However, the analysis of resilient behaviour results is substantially more complex and time consuming which may limit the time advantage;
- last, but not least, most current design methods only consider the resilient properties of the materials.

The study of permanent deformations is, however, decreasingly important when the material is treated. In fact, it is very unlikely that permanent deformations may be of concern in bound layers as it will be demonstrated later in this Chapter. Nevertheless, in some of the mixtures considered the binder content is very small and there is no previous work involving the study of permanent deformations on lightly-treated secondary materials. Because of this, their investigation was considered worthwhile.

6.7.2. Model for permanent deformations

For the reasons described, no attempt was made to develop a specific model for treated materials. Instead, after a comprehensive review of the existing models it was decided to use the French approach which is believed to be the best available method at the present [Paute, Hornych & Benaben, 1996]. This method is also being considered in the European Committee for Standardisation [CEN, 1995].

The model was developed in the *Laboratoires des Ponts et Chaussées* (LPCs) from results of variable confining pressure tests with the application of 80000 cycles of loading. It considers the permanent deformation with respect to the number of cycles and to the applied stresses. The two equations presented below are valid for stress states well below failure.

Influence of the number of cycles

$$\epsilon^{*}_{1p\ (N)} = \epsilon_{1p\ (N)} - \epsilon_{1p\ (100)} = A_1 \left[1 - \left(\frac{N}{100} \right)^{-B_1} \right] \qquad 6.38$$

where: $\epsilon_{1p}{}^{*}$ corrected permanent axial strain after N cycles;

$\epsilon_{1p\ (N)}$ permanent axial strain after N cycles;

$\epsilon_{1p\ (100)}$ permanent axial strain after 100 cycles;

A_1, B_1 material constants;

N number of cycles.

An important practical significance is attributed to the parameter A_1, representing the limit value of the permanent axial strain for a given stress path. For practical purposes A_1 is considered the lower of the following two values:

- the calculated value from Equation *6.38* using regression analysis;
- twice the corrected permanent axial strain after 20000 cycles.

Influence of the stress level

$$A_1 = \frac{\left(\dfrac{q_{max}}{p_{max} + p^*}\right)}{a - b\left(\dfrac{q_{max}}{p_{max} + p^*}\right)} \qquad 6.39$$

where: p_{max} maximum mean normal stress experienced during a stress excursion;

q_{max} maximum deviatoric stress experienced during a stress excursion;

a, b positive material constants.

6.7.3. Permanent deformation results and discussion

Some changes were introduced during the execution of the programme in relation to the technique previously described. First of all, it was not possible to test four specimens for all the materials considered in this research due to the heavy testing programme. On the one hand, for the treated materials this did not represent any problem since, after an initial test, permanent deformations were found not to be of concern and one test would be enough to have a quantitative measurement of the materials' behaviour. On the other hand, for the unbound material it is possible to apply the French model to the results of only one test in order to have a complete characterisation of the material in terms of permanent deformations both in relation to the influence of the number of cycles and of the stress level. This is possible provided that the failure line of the material is known, as in this case (Figure 6.24). For these reasons, only one permanent deformation test was performed in each case. The results are presented in Tables 6.12 and 6.13 and in Figure 6.43.

The failure of the specimen CC5 after 5000 loading cycles and of another specimen of treated furnace bottom ash after 1000 cycles gave rise to further changes in the way the specimens were tested. Hence, in order to avoid further failures during testing, the remaining materials were tested maintaining the confining pressure constant throughout the tests. This is likely to cause some underestimation of the permanent deformations. Nevertheless, as the stress paths applied during the resilient study are well below failure it is still possible to quantify the rate of accumulation of permanent strains and compare results so as to allow material discrimination and classification.

Table 6.12 Summary of permanent deformation results.

Material	**Binder**	**Stress Path**		$\epsilon_{1p}^*{}_{(80000)}$
		σ_3 **kPa**	**q kPa**	**$\mu\epsilon$**
	UNBOUND			
CC5	Unbound	0 - 33	0 - 200	-----
T1	Unbound	0 - 100	0 - 600	814
SW1	Unbound	100	0 - 500	1099
	TREATMENT LEVEL I			
CC4	c = 2 %	100	0 - 600	201
BA4	c = 2 %	100	0 - 600	589
MI1	l = 5 %	100	0 - 600	404
FA2	c = 2 %	100	0 - 600	24
	TREATMENT LEVEL II			
CC2	c=2% + pfa=8%	0 - 100	0 - 600	15
BA3	c=2% + pfa=8%	100	0 - 600	433
MI4	l=3% + pfa=12%	100	0 - 600	49
FA3	c = 5 %	100	0 - 600	6

Notes: ----- specimen failed after N = 5000.

Table 6.13 Results of modelling permanent deformation.

Material	Binder	Influence of the no. of cycles			Influence of the stress level			A_{1c} ($\mu\epsilon$)
		A_1 (%)	B_1	R^2	p^* (kPa)	a	b	
	UNBOUND							
CC5	Unbound	131.16	0.00306	0.94	128	46.4	30.7	408430
T1	Unbound	0.18	0.09230	0.99	51	5003.6	2199.3	1368
SW1	Unbound	1232.63	0.000010	0.86	68	4062.3	1978.3	1950
	TREATMENT LEVEL I							
CC4	c = 2 %	227.96	0.000012	0.80				
BA4	c = 2 %	469.50	0.000013	0.82				
MI1	l = 5 %	0.026	0.59603	0.90				
FA2	c = 2 %	11.80	0.000024	0.74				
	TREATMENT LEVEL II							
CC2	c = 2 % + pfa = 8 %	-11.20	0.000022	0.19				
BA3	c = 2 % + pfa = 8 %	336.58	0.000014	0.65				
MI4	l = 3 % + pfa = 12%	0.019	0.04475	0.99				
FA3	c = 5 %	-11.53	-0.000023	0.54				

Influence of the number of cycles

The results show that the equation reflecting the influence of the number of load applications, Equation *6.38*, gives good results for unbound materials, but the same does not happen for treated materials, being totally inadequate for some. It is not possible to identify in this case rational groups for which a common model performance is observed.

Another important aspect is that the significance attributed to the parameter A_1 is not always realised in practice. Indeed, that meaning would correspond to a limit value of the permanent axial strain of 1200 % for slate waste (SW1) and a negative value for CC2 and FA3. Of course, when this happens the suggested practical value needs to be adopted, which is equal to twice the corrected permanent deformation after 20000 cycles (2 x 604 $\mu\epsilon$ for SW1).

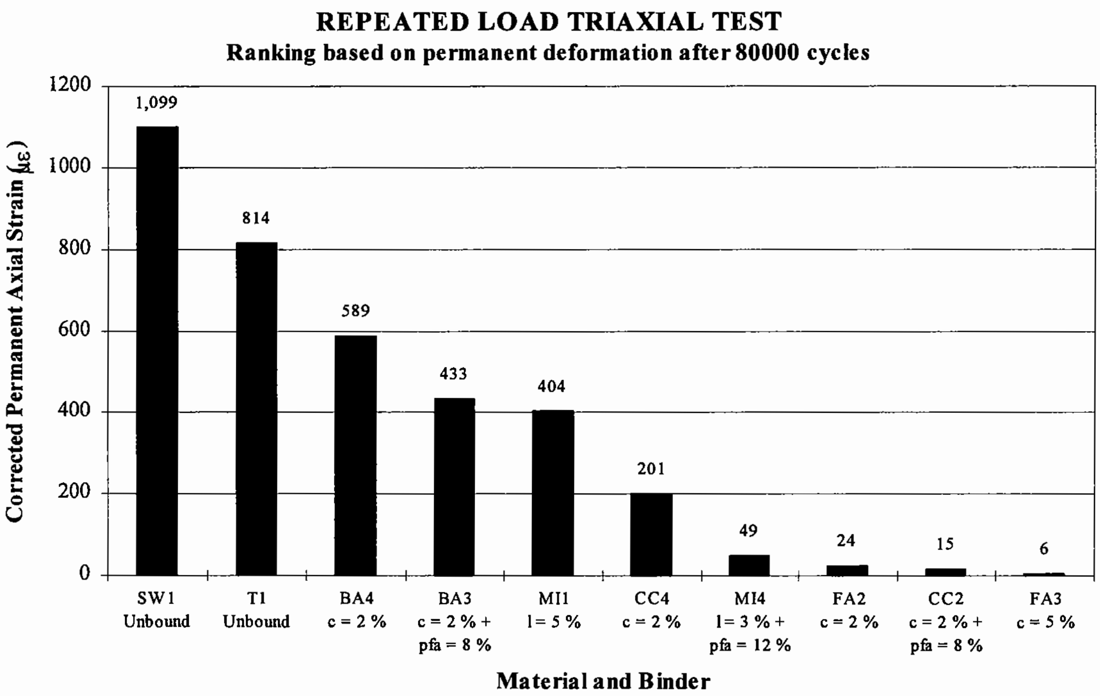

Figure 6.43 Permanent deformation results.

This model predicts well the deformations tending asymptotically to a limit value. However, it may be demonstrated experimentally that some materials do not behave in that manner exhibiting non-asymptotic development of permanent strains. Illustrative examples are presented in Figures 6.44 to 6.46. For the furnace bottom ash, Figure 6.45, some deterioration of the material or of the inter-particle bonds may be responsible for the acceleration in the accumulation of permanent deformations. For slate waste and bottom ash the correlation coefficients obtained were reasonably high, 0.86 and 0.82, respectively. Nonetheless, the quality of prediction as perceived from those figures is not adequate and it is important to examine the results graphically and to judge the quality of fit of the model.

Influence of the stress level

By using parameters p^* and $M = a / b$ (Figure 6.26) from the determined failure lines of the unbound materials, it was possible to determine both constants a and b of Equation *6.39*, and calculate the characteristic permanent strain for the standard stress path. For granite (T1) and slate waste (SW1) the characteristic permanent strain seems to be reasonable taking as a reference the corrected permanent strain after 80000 cycles (Table 6.12). In Figure 6.47 the parameter A_1 is plotted for the unbound slate waste, representing a complete characterisation of the influence of the stress level on the development of permanent strains.

Conversely, the value of A_1 obtained for china clay sand (CC5) is too high. It may be argued that since the specimen failed after approximately 5000 cycles, a high value would necessarily result. This does not explain A_1 totally since the data considered for regression were recorded up to 2500 cycles over which range the model fits well the accumulation of permanent strains with the number of cycles (R^2 = 0.94). In addition, the characteristic permanent strains of the material should be the same independent of the stress path applied, which does not seem to be the case.

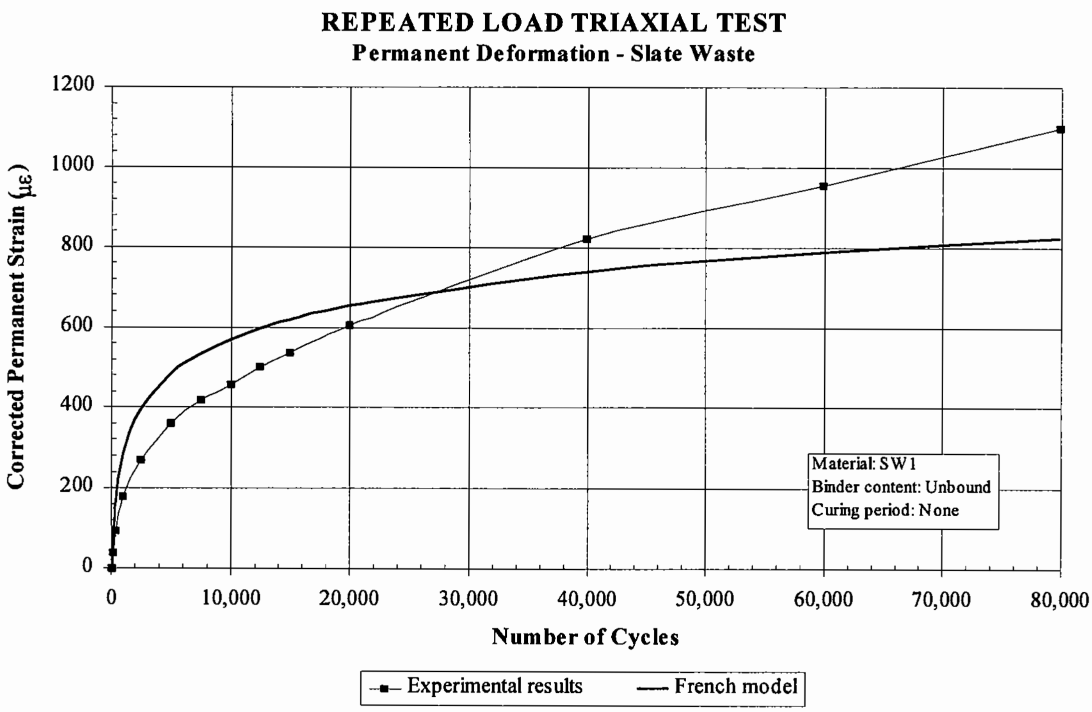

Figure 6.44 Non-asymptotic development of permanent strains (SW1).

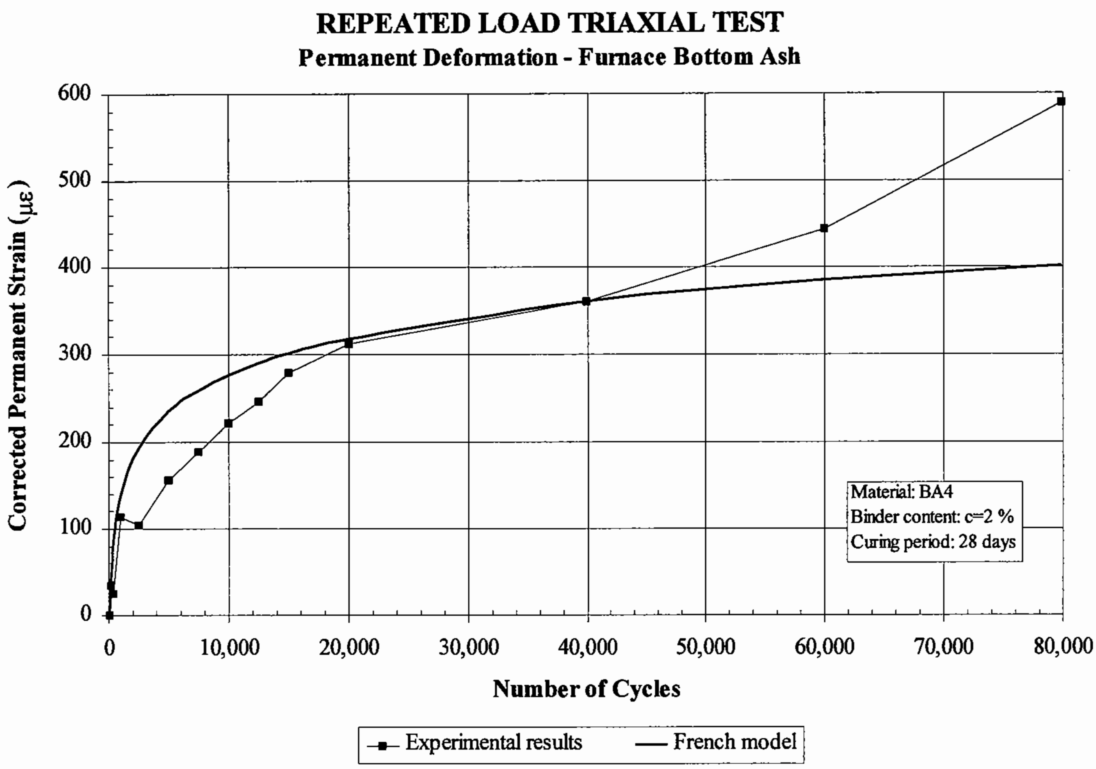

Figure 6.45 Non-asymptotic development of permanent strains (BA4).

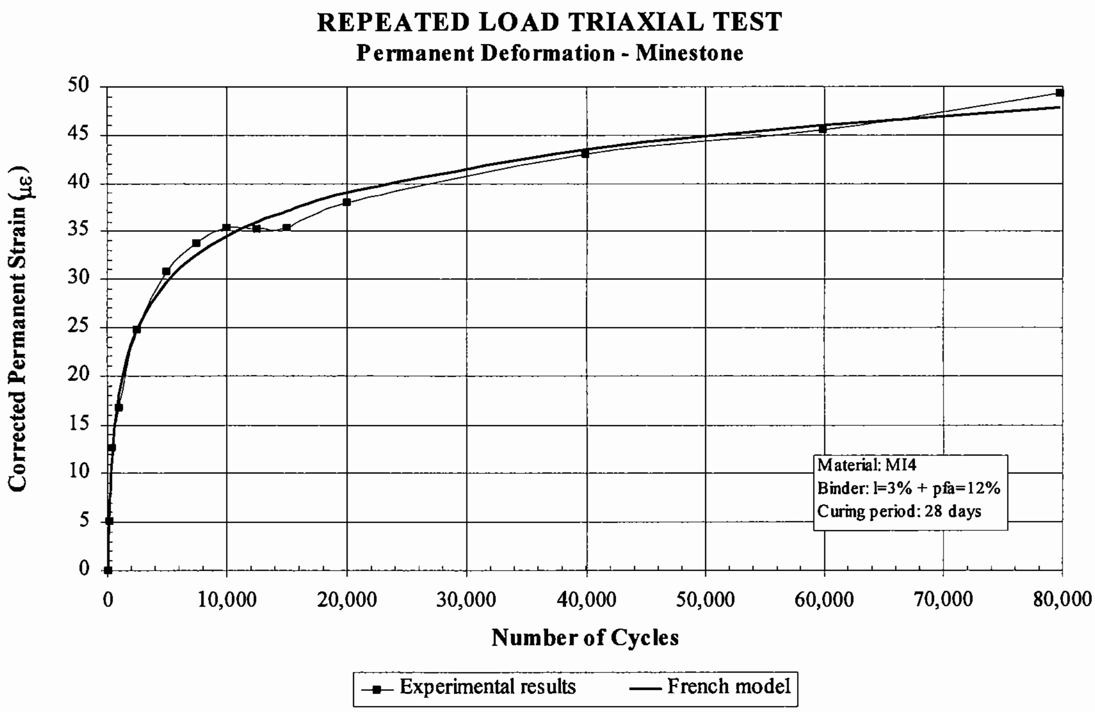

Figure 6.46 Non-asymptotic development of permanent strains (MI4).

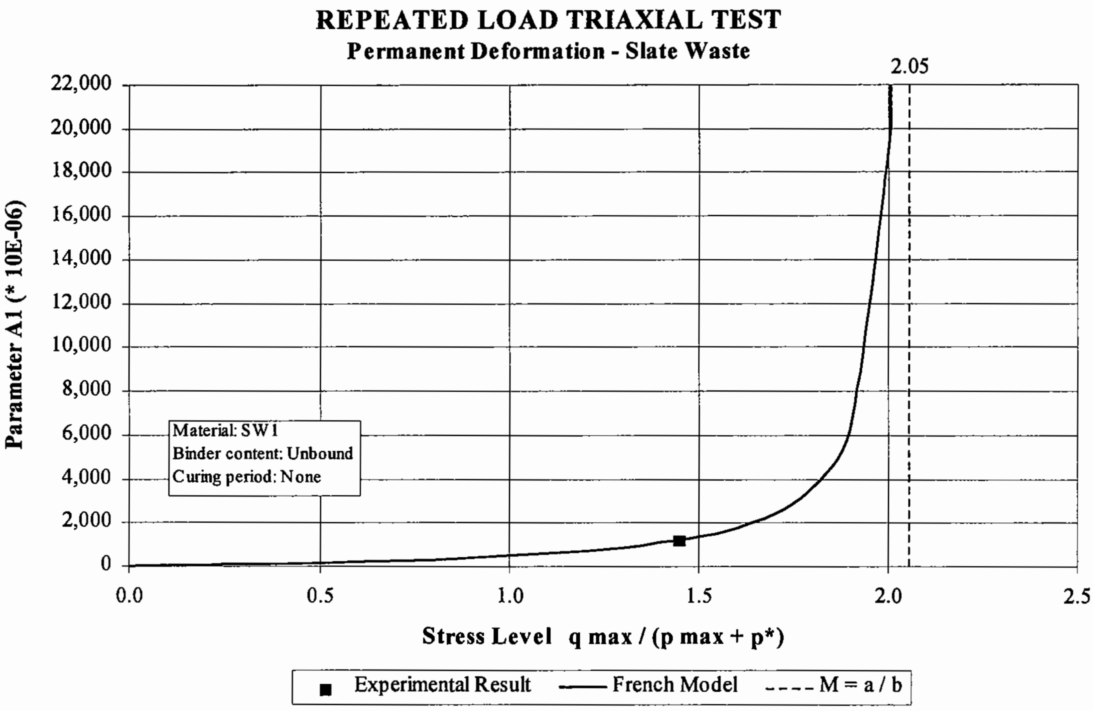

Figure 6.47 Permanent deformation model - influence of the stress level.

A research project to study the behaviour in terms of permanent deformations of five different granular materials was conducted at the University of Nottingham [Lekarp, Richardson & Dawson, 1996]. It has been concluded that this approach using data from failure lines gives origin to high scatter and no evidence of a relationship between permanent strains and static failure, as suggested in the French model, was found. This appears to confirm the results obtained here.

For the reasons presented above it was decided to rank the materials based on the corrected permanent axial strain after 80000 cycles (Figure 6.43), instead of considering their respective characteristic permanent strains. Unbound materials are clearly more susceptible to permanent deformations than treated materials as we would expect. There is no data at 80000 cycles for unbound china clay sand due to the failure of the specimen but its performance is worse than that of the unbound slate waste (SW1). This may be due to its particle size distribution being more uniform than Type 1 sub-base materials and the round shape of the particles.

On the other hand, contrary to what was found in terms of resilient modulus for which treated materials such as furnace bottom ash (c = 2 % and c = 2 % + pfa = 8 %) and minestone (l = 5 %) performed in a similar manner to unbound materials, the permanent deformations are substantially reduced by treatment. See in particular the position of china clay sand with just 2 % of cement (CC4). For the other mixtures, with higher levels of treatment, this issue becomes less and less important.

6.8. Proposed classification system

A classification system based on the results of the repeated load triaxial tests should be based on both resilient and permanent deformation characteristics. The proposed system (Figure 6.48) draws on the classification method proposed in France for unbound granular materials [Dawson, Thom & Paute, 1996] which considers the characteristic values as the basis of the classification. Due to the problems experienced in modelling permanent deformations and the limitations of existing models the corrected measured permanent strains after 80000 cycles were considered instead of the characteristic values.

For the materials tested, there appears to be a negative correlation between the characteristic values. This is reflected by the correlation coefficient obtained of 0.88. Although it is an overall tendency, it is not always verified and materials with high resilient modulus may be susceptible to permanent deformations and vice-versa depending on factors such as the grading of the material, shape of the particles, moisture content and structure of the compacted material.

The classification system incorporates four classes of materials:

- *Class C1* this class was considered to include treated material with very high resilient modulus and very low susceptibility to permanent deformations;
- *Class C2* top quality unbound granular materials;
- *Class C3* medium quality unbound granular materials;
- *Class C4* low quality unbound granular materials.

The class limits are empirically defined based on the experience of testing a wide range of granular materials. Classes C2, C3 and even C4 may also include lightly-stabilised materials, in particular some treated secondary materials, for which the treatment does not improve the material's performance above the levels that could be expected from unbound materials. For those, their behaviour under repeated loading is likely to be of the unbound type.

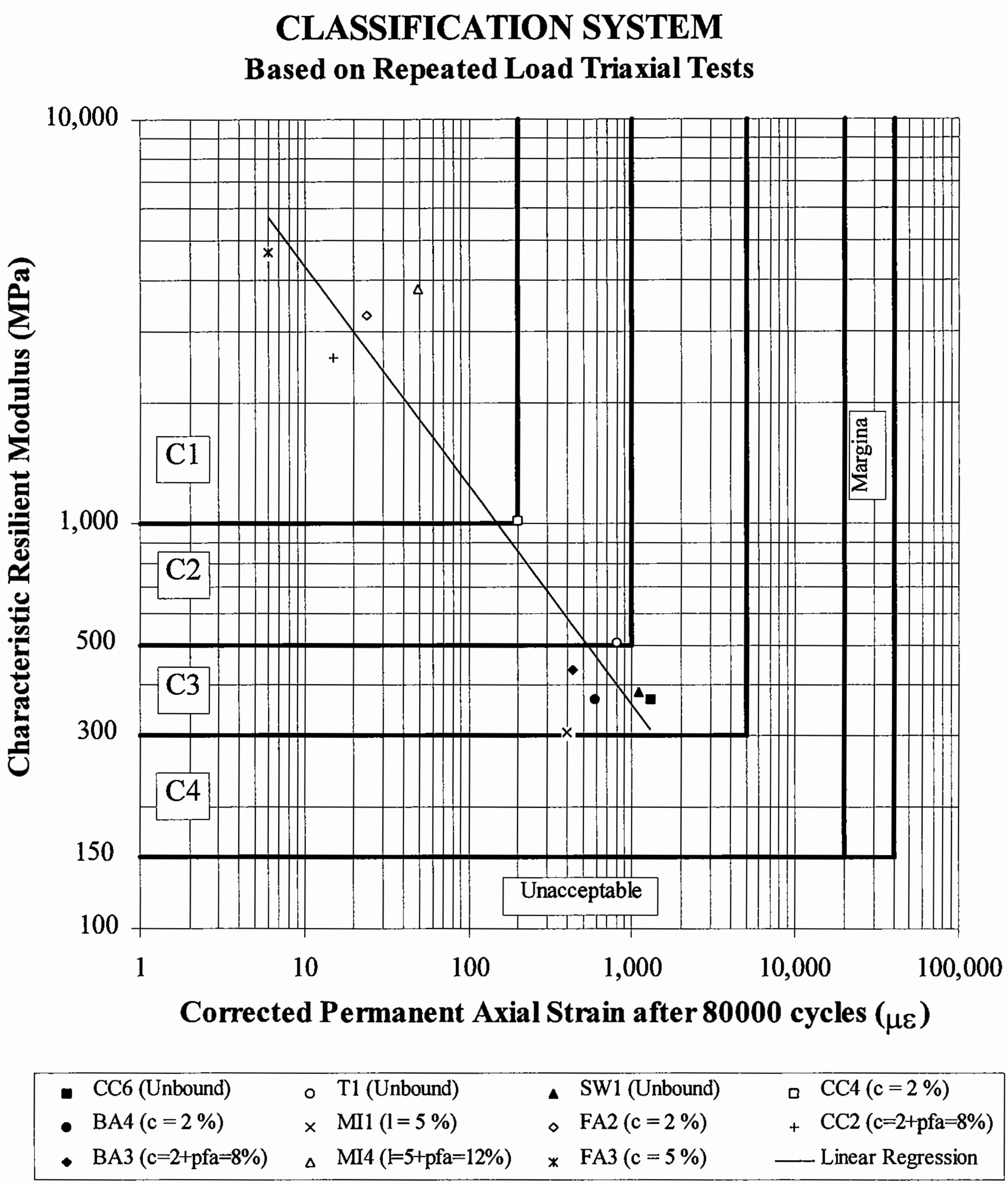

Figure 6.48 Classification system proposed.

6.9. Summary

- During the preliminary testing of unbound slate waste it was possible to identify the limitations of Mr - ν models such as K-θ and Uzan models. The compaction method was also found to be inadequate to replicate the type of densities achieved in the pavement.

- The test technique required for repeated load triaxial tests included all phases: preparation, conditioning, strength assessment, resilient behaviour and permanent deformation tests. The necessary changes to recommended procedures were identified and described. The need to refer the applied stress paths to the failure line of the materials studied was emphasised.

- A compaction technique was developed for the compaction of unbound and stabilised specimens. The material is compacted in five layers, in plastic moulds which are easy to manufacture and inexpensive. The compaction simultaneously uses a vibrating table and a vertically-guided vibrating hammer.

- The stress / strain behaviour of lightly-treated materials does not seem to be very different in type from that of unbound materials. For the higher levels of treatment, however, non-linearity and stress-dependency are reduced.

- With regard to the result of mechanical properties it was possible to identify two different groups: the first comprising unbound materials and mixtures with similar results, was identified as having an 'unbound type of behaviour'; the second of materials corresponding to a 'treated type of behaviour'.

- In general, treatment improves the triaxial strength of the materials in a significant way. One exception identified was minestone stabilised with 5 % of lime. Its strength remained at the top end typical of unbound materials indicating that this mixture is not totally successful in taking full advantage of the binder used.

- Again, treatment enhances resilient modulus but in some cases that improvement just brings the material to the level expected for conventional unbound materials. This is the case of minestone with 5 % lime and furnace bottom ash either with 2% cement or 2 % cement plus 8 % fly ash. However, it is necessary to consider those results in comparison with their performance when they are unbound which was considered to be unacceptable for minestone and poor for bottom ash. The treatment of minestone with lime and fly ash appears to be more appropriate.

- Available models for resilient behaviour were comprehensively reviewed and discussed. Several hypotheses were considered for analysis, namely Pappin's model, Mayhew's model and modified versions of Mayhew's model incorporating the parameter p^* and stress path length dependency.

- The Pappin and modified Mayhew models best described the materials with 'unbound type of behaviour', but their performance clearly declined for treated materials.

- A new K-G model was developed for treated granular materials. Its performance was consistently better for those materials with 'treated type of behaviour'. Indicative boundaries of validity in terms of mechanical properties (resilient modulus, Poisson's ratio and triaxial strength) were presented for Pappin or modified Mayhew models and for the new model.

- The French model for permanent deformations was adopted to analyse the experimental results. In general, the equation reflecting the influence of the number of cycles was found to give good results for unbound materials, but poor to bad results for most of the treated materials. The correlation coefficient alone is insufficient to draw any conclusions on the appropriateness of the model and for this reason a graphical examination is recommended. Often, the materials studied showed a non-asymptotic development of permanent deformations which is not well modelled by the French approach.

- Some limitations were identified with the equation to model the influence of the stress level. In particular, the characteristic values determined by using the data from failure lines do not seem to be consistent, nor independent of the stress level applied.

- Permanent deformations were successfully reduced through treatment for all mixtures. For the higher levels of treatment the deformations obtained were negligible.

- A classification system for granular materials has been proposed developed from the French method. It considers the characteristic resilient modulus and the corrected permanent axial strain after 80000 cycles which may be obtained from repeated load triaxial tests.

CHAPTER 7

REPEATED LOAD INDIRECT TENSILE TESTS

7.1. Introduction

The number of tests that can be used to evaluate the mechanical properties of stabilised materials is considerable. Some of the most commonly used are [Molenaar, 1993]:

- beam flexure test;
- direct tensile test;
- indirect tensile test;
- uniaxial compression test;
- triaxial compression test;
- resonant column test;
- ultrasonic test;
- wheel track test.

These tests can be performed either using monotonic or repeated loading. However, it is generally accepted and understood that for a closer simulation of *in situ* pavement loading conditions the application of repeated loading is essential.

In this research the repeated load triaxial and indirect tensile tests were adopted. The first is widely recognised as a performance test of some excellence for the study of unbound / bound pavement materials. On the other hand, the indirect tensile test is more controversial with a large number of followers but also of opponents. There is a popular English saying that has been used in a discussion on the appropriateness of the indirect tensile test for material evaluation:

> "*Being an Englishman you never believe anything you read and only half of what you see*"

that illustrates the distrust of some towards this form of testing. Incidentally, as we stagger into the information age the proverb may also need some reformulation to a more conservative percentage of what one sees...

A comprehensive laboratory characterisation of stabilised mixtures for road applications involves the study of several properties that bound materials should possess in order to behave adequately in the pavement structure. The following aspects were considered in this study:

- tensile strength;
- stiffness modulus;
- Poisson's ratio;
- resistance to fatigue;
- durability; and
- development of properties with time.

In this chapter the major objective is to present the methodologies for assessing the mechanical properties of pavement materials stabilised with hydraulic binders using indirect tensile tests. Special emphasis will also be given to the validation of the developed techniques and in demonstrating the usefulness of this mode of testing for material evaluation.

7.2. Discussion on the indirect tensile mode of testing

Mixtures considered for stabilisation should be designed and optimised taking into account strength, stiffness and fatigue properties. When studying secondary materials the number of possible mixtures (secondary materials, secondary binders, or a combination of them) is generally higher than when using conventional materials. In addition, experience / information available is more scarce. The need for a simple test to characterise them which is able to produce consistent measures of performance is of major importance. It is for this purpose that the repeated load indirect tensile test may have greater potential. Owing to its expedite nature, other fields of application include quality control and compliance and in the characterisation process for pavement back-analysis techniques.

One of the complaints commonly made by critics of the indirect tensile test is that two halves of a cylinder loaded diametrically along the joint still gives a tensile strength value [Stock, 1992]. This observation misses two points:

- in fact, the tensile strength of such a test specimen is zero and it is impossible to prove / measure otherwise;
- it is wrong to use continuum elastic media analysis to calculate a tensile strength that does not exist between two parts of a split cylinder.

Some of the criticisms towards this test are believed not to be completely justified. There has been a considerable amount of research work carried out using this type of test recently and important improvements have been made since the test was first described in 1953 [Carniero & Barcellus, 1953; Akazawa, 1953].

The strongest laboratory alternatives for this test are the direct tensile test and the beam flexure test. For the evaluation of strength, the direct tensile test has been preferred in the procedures for the characterisation of bound mixtures to be included in the European Standards [CEN, 1995 i] though the indirect tensile test is also allowed as an alternative method. Concerning stiffness, direct tension or compression are recommended. A report of one of the Technical Committees of the Permanent International Association of Road

Congresses [PIARC, 1991] endorses the use of either direct tensile tests or splitting tensile tests for measuring tensile strength and stiffness and, when new materials are to be tested, to complement the study with fatigue behaviour evaluation using flexural tests.

None of these three tests reproduces, in a representative way, the complex tridimensional stress state in the pavement with rotation of principal stresses and simultaneous application of normal and reversed shear stresses.

A summary of advantages and disadvantages of the direct tensile test, beam flexure test and indirect tensile test is presented below. The issues facing the indirect tensile mode of testing are further discussed with the aim of demonstrating its suitability for pavement materials' assessment.

Direct tensile test

Advantages

- using adequate techniques the properties obtained are the best approximation to the real values
- uniform stress distribution in the central part of the specimen
- no assumptions required for tensile strength calculation

Disadvantages

- complexity and time requirements
- possible underestimation of results
- special influence of compaction method
- impossibility of testing specimens from in-service pavements.

The test is reasonably complex and time-consuming. The application of pure tension is very difficult and for this reason special care is necessary when setting up the specimen in the testing rig to ensure its correct alignment and to avoid any eccentricity of the load applied. When grips are used to hold the specimen they also introduce secondary stresses into the specimen.

There are a number of factors that may result in an underestimation of the tensile strength:

- load eccentricity;
- secondary stresses induced by grips;
- the application of the maximum tensile stress to a large volume of the specimen and thus the higher probability of including weak sections [Neville, 1995];

The method of compaction used for direct tensile specimens is extremely important and may affect results in a more serious manner than for other type of tests. Unless the material is compacted in one layer, using vibro-compression techniques that are not readily available, the artificial creation of weak sections is probable. Indeed, failure in those cases is likely to occur in sections of the specimen between compacted layers. When the form of the specimen is such that there are discontinuous changes in the specimen's transverse area, their effect combined with very small eccentricities of the load and the effect of the grips may also result in preferential failure surfaces at those levels.

As far as stiffness is concerned, its value is determined as the slope of the stress / strain line, passing through the origin, and corresponding to a tensile stress level of 30 % of the tensile strength [NF P 98-232, 1992], or 50 % of that value [Shahid, Thom & Peaston, 1996]. Another method is to plot the stress / strain curve, the stiffness being the slope of the line passing through the origin and tangential to that curve [Shahid, 1994]. There are a few remarks worth mentioning:

- perfect elastic behaviour is also assumed which is one of the main criticisms towards the indirect tensile test;
- materials are not conditioned prior to testing for stiffness modulus determination or the conditioning is done at very small stress levels, well under the stresses applied during the test;
- for non-linear materials the stiffness moduli determined are only valid for a particular stress condition;
- the test is performed under static conditions (monotonic loading) failing to simulate the repeated loading due to traffic.

Stiffness in compression and in tension have been reported to agree reasonably well [Schmidt, 1972; Williams, 1986]. In this case, the importance of the direct tensile test for stiffness determination is diminished due to the increased complexity of the test when compared with uniaxial compression tests.

Important developments have been made in recent years to solve some of the problems referred above [Shahid, 1994]. However, the probability of the direct tensile test to become a routine test, especially in the more demanding area of control and compliance of pavement materials, remains low in the short term due to the difficulty in compacting tensile specimens in the field, the impossibility of obtaining specimens with the required shape from an in-service pavement and the need for specialised equipment.

Beam flexure test

Advantages

- simulation to some extent of stress state in the lower part of bound layers (large horizontal tensile stresses combined with small vertical compressive stresses)

Disadvantages

- considerable overestimation of results, in particular strength values
- need to extract slabs from the pavement to assess mechanical properties of finished pavements
- strong influence of moisture content and variability of strength results.

The most commonly used beam flexural test is the third-point loading in which the load is applied in two line loadings spaced at one-third of the beam length and symmetrically located in relation to the centre. On-sample instrumentation may be used improving the overall quality of the test [Correia & Nunes, 1989]. The analysis of results is based on elastic beam theory in which linear stress-strain relationship and linear stress diagram are assumed. Thus, the stresses and strains in the beam are assumed to be proportional to the distance to its neutral axis. This is just an approximation of the reality as it is known that the actual stress diagram for loads approaching failure is parabolic and as a result the tensile strength is overestimated.

Another aspect that may result in overestimation of the results is slower crack propagation due to the materials in the vicinity of the neutral axis which are subject to much lower stresses [Neville, 1995].

The flexural strength test is not recommended for control or compliance purposes due to the strong influence of moisture content and the variability of the tensile strength obtained in this test [Neville, 1995]. For testing specimens from a pavement it is necessary to cut slabs and then manufacture the beams with adequate dimensions. Though this is commonly done the extraction of cores is much easier and less disruptive.

Repeated load indirect tensile test

Advantages

- facility in obtaining specimens in the laboratory and especially in the field through coring represents the main advantage of this test
- using adequate techniques the properties obtained are close to the actual values
- expedite nature of the test
- more uniform results
- the equipment necessary is relatively inexpensive

Disadvantages

- small overestimation of the actual tensile strength
- highly variable stress state induced to the specimen
- impossibility of controlling the ratio of compressive stress to tensile stress applied to the specimen
- formation of wedges under certain conditions

For bound pavement layers the design criterion is usually the maximum tensile strain or stress at the base of the bound layers underneath the load. In this area, the stress state is characterised by longitudinal and transverse tension combined with vertical compression. This is similar to the biaxial stress state in the proximity of the centre of the specimen in indirect tensile mode, though the ratio between compressive and tensile stresses cannot be controlled during test. However, if the specimen is directly cored from the pavement it should be cored horizontally for the directions of the compressive and tensile stresses to coincide in the field and in the laboratory. This type of specimen is more difficult to obtain since *in situ* coring is normally done vertically. Nevertheless, it is possible to cut a slab from the pavement and core specimens from that slab in the appropriate direction. In the laboratory it is also possible to follow this procedure which is common practice for bituminous materials.

It should be emphasised, however, that the direction of testing is more important for materials exhibiting strong anisotropic behaviour. This will depend essentially on the type and shape of the aggregate and whether or not the aggregate orientates preferentially in a particular direction during compaction [Karasahin, 1993]. Contrary to what happens with some unbound materials, there is no evidence that stabilised granular materials behave in a highly anisotropic manner.

One common criticism of this test is the use of linear elastic analysis to interpret the results. This is a valid comment due to the limitations of elastic theory to analyse non-elastic, stress-dependent materials. Yet, this approach is commonly used in the alternative tests and so it is not a unique disadvantage of the indirect tensile test.

In Figures 7.1 and 7.2 the theoretical stress distribution in a cylindrical specimen loaded in indirect tensile mode is presented based on the stress analysis conducted by Hondros [1959]. The sign convention generally used differs from that normally followed in soil mechanics and adopted in Chapter 6. Thus, tension is here considered positive and compression negative.

It can be seen that, on the vertical axis in the central part of the specimen, typically one fourth of the diameter, the stresses do not change significantly and for this reason this is the best area to locate on-sample instrumentation. In particular, the tensile stress along the vertical axis where failure is initiated is approximately constant with the exception of the areas in the vicinity of the load. Here, high horizontal compressive stresses are induced in the specimen simultaneously to high vertical compressive stresses of similar magnitude, thus originating a biaxial compressive stress state.

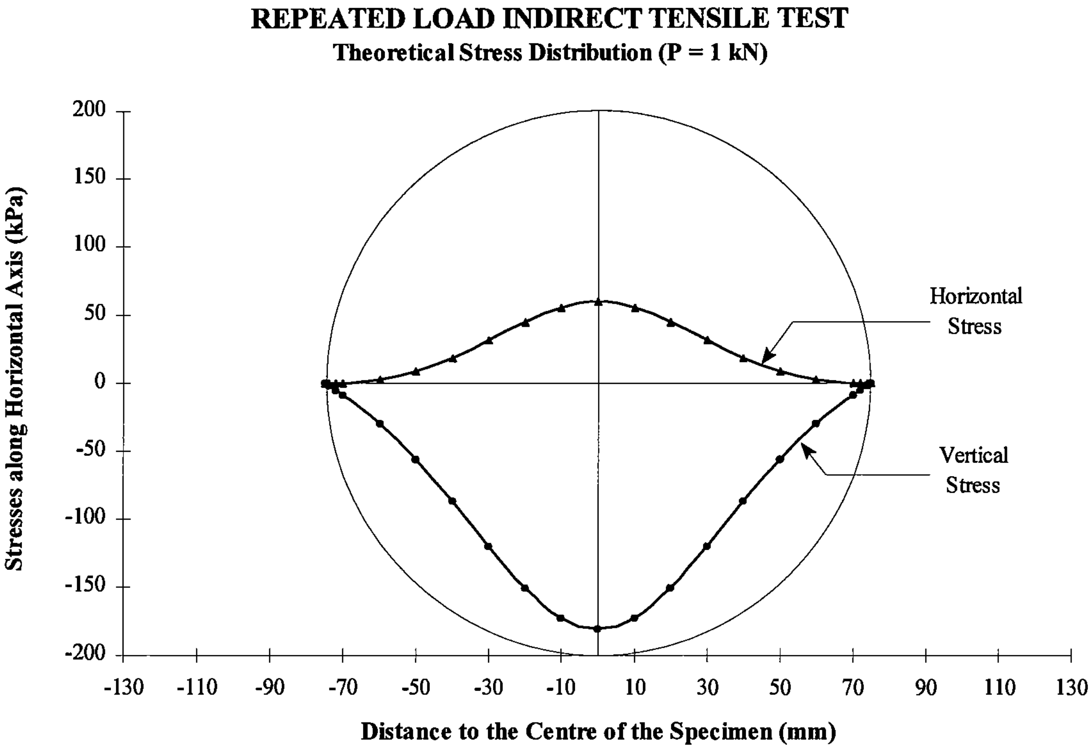

Figure 7.1 Stresses in indirect tensile specimen (horizontal axis).

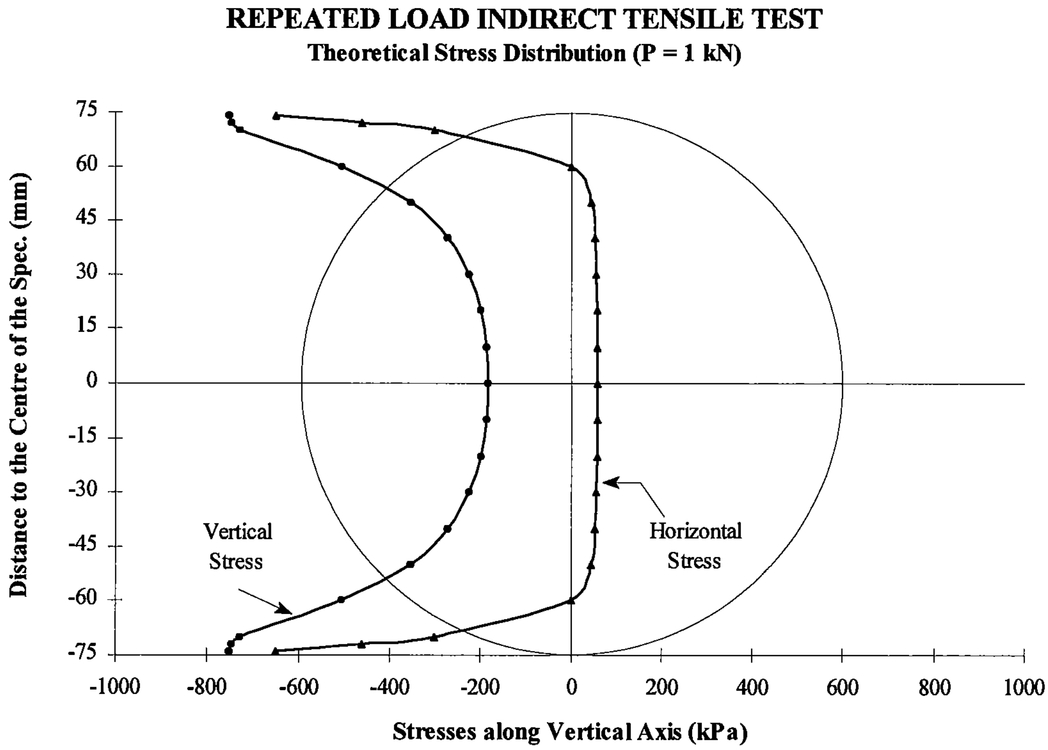

Figure 7.2 Stresses in indirect tensile specimen (vertical axis).

Although shear stresses exist in that area, equal to half the difference between normal stresses and of about twice the magnitude of the shear stresses in the centre of the specimen, no evidence was found in the bibliography that they would initiate failure of bound materials. As a matter of fact, a premature failure due to those stresses would result in underestimation of the tensile strength when all the evidence points to a small (severe, according to those most critical of the test) overestimation of that property.

The reasons why failure is not usually initiated in this area may be understood from the analysis of Figure 7.3. At the centre of the specimen, the ratio between tensile and compressive stresses, $\sigma_{x\,max}$ and $\sigma_{y\,max}$, is of 1 : 3 (Mohr circle A). That ratio in terms of ultimate strengths for stabilised materials is of about 1 : 10 [Williams, 1986] (Mohr circles B and C) and the maximum unconfined compressive stress, which occurs approximately 15 mm from the surface of the specimen down the vertical axis (Figure 7.2), is approximately 8.5 times the magnitude of the maximum tensile stress (Mohr circle D). Closer to the loading heads and the surface of the specimen a biaxial compressive stress state exists, which means that the specimen can resist considerably higher stresses than its compressive strength determined in unconfined compression tests (Mohr circle E).

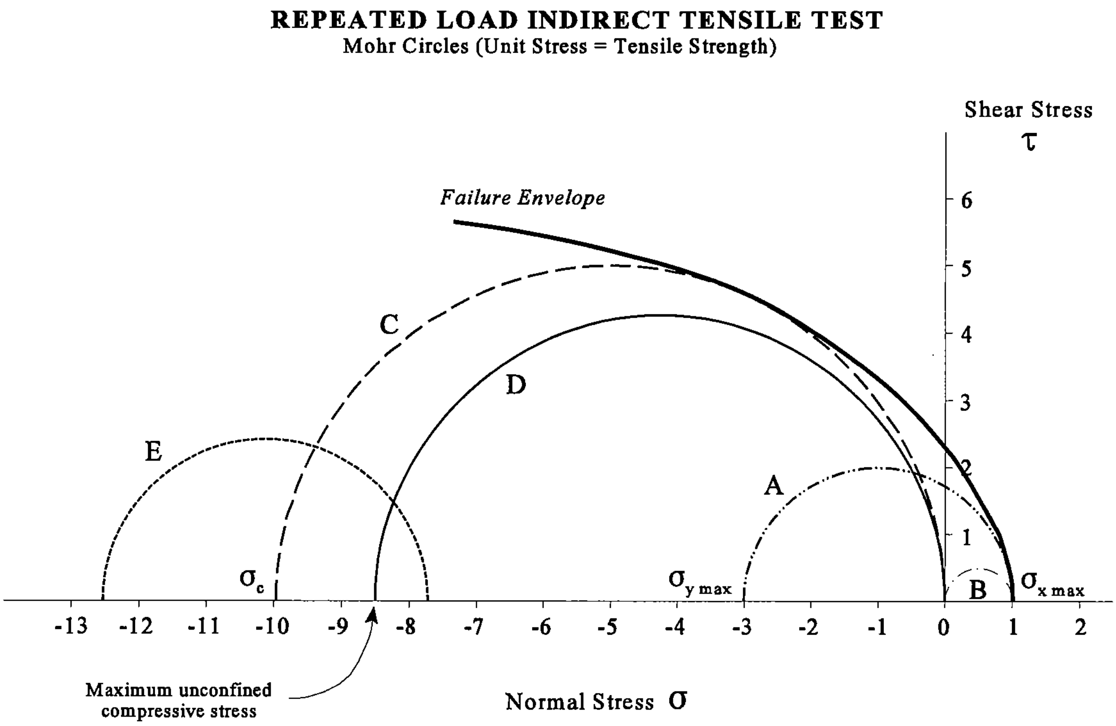

Figure 7.3 Mohr circles of the theoretical stress distribution in indirect tensile mode.

These local effects in the contact of the loading head with the specimen leading to stress concentrations are of some concern. For this reason, Poisson's ratio is generally not well estimated using vertical deformations measured externally on the loading strips. This is similar to what happens in other forms of testing, such as triaxial testing, in which the end effects at the contacts of the specimens with the top and bottom platens may have a significant effect on the results when the vertical deformations are measured externally. To overcome this problem, on-sample instrumentation is used, and this may also be the solution for indirect tensile tests when used in areas of the specimen where the stresses are relatively constant.

The system for the measurement of horizontal deformations using LVDT frames as suggested by Schmidt [1972] may restrain the free deformation of the specimen thus modifying the stress state of the specimen. An adaptation of this system is commonly used nowadays for the determination of the indirect tensile stiffness modulus of bituminous materials [BS DD 213, 1993]. Significant experience has been gained recently with the use of the Nottingham Asphalt Tester in this form of testing which seems to confirm that the LVDT frame does not affect the results obtained substantially. The alternative LVDT frame described in Chapter 5 which is supported in single 'points' on each side of the specimen would decrease this problem. However, no significant difference in results was obtained.

Inadequate alignment or the existence of irregularities in the specimen surface may contribute to inaccuracies in the measurement of deformations due to the rotation of the specimen and difficulties in assessing the implications on the stress level. Clearly, if the load is not applied uniformly this will lead to stress concentrations and the normal analysis based on two-dimensional plane-stress elastic theory is not valid. Specimen frames with vertical guides for the loading head have the advantage of hindering any specimen rotation and contribute to the uniform application of the load. This system has been criticised for reducing the load transmitted to the specimen through friction on the guides [Roque & Buttlar, 1992]. This problem has been solved in this research with the introduction of an internal load cell under the loading heads measuring the actual force transmitted to the specimen.

The indirect tensile test is easy to perform, it is reported to give more uniform results than with other alternative tensile tests and the strength obtained is believed to be close to the strength of the material [Neville, 1995]. This was also confirmed using the finite-element method to explain analytically the differences in results from the three tests in analysis [Raad, Monismith & Mitchell, 1977]. Those researchers concluded that:

- the direct tensile test produces the most reliable tensile strength;
- the beam flexure test may originate tensile strength values that are up to twice the real value;
- the indirect tensile test seems to be the most appropriate in practice producing values that differ by not more than 13 % from the actual tensile strength.

Although some assumptions were considered to conduct the finite-element analysis, such as linear elastic behaviour and plane-stress conditions, other experimental relationships between tensile strength test results found in the bibliography appear to confirm this tendency. A brief summary of the equations is presented in Table 7.1.

Table 7.1 Relationships between tensile strength tests.

Relationship	**Reference**	**Notes**
$\sigma_{td} = 0.8\ \sigma_{ti}$	[CEN, 1995 i]	Fly ash bound materials
$\sigma_{td} = 0.6\ \sigma_{tf}$	[Kolias & Williams, 1978] [1]	Cement bound materials
$\sigma_{tf} = 1.5\ \sigma_{ti}$	[Quaresma, 1990] [2]	Cement bound materials
$\sigma_{td} = 0.9\ \sigma_{ti}$	from [1] and [2]	
$\sigma_{td} = 0.89$ to $0.95\ \sigma_{ti}$	[Neville, 1995]	Concrete
$\sigma_{td} = \sigma_{ti}$	[Dempsey et al., 1984]	Stabilised materials

Key:
σ_{td} tensile strength in direct tension
σ_{ti} tensile strength in indirect tension or Brazilian splitting test
σ_{tf} tensile strength in flexural test

Considering that there are factors that may give rise to an underestimation of the tensile strength in the direct tensile test, the equations presented in Table 7.1 seem to confirm that the strength obtained in the indirect tensile test is very close to the actual values. Hence the possible formation of wedges in the specimen in the areas immediately in contact with the loading strips appears not to result in major overestimations of the tensile strength. Moreover, from the experience gained during this study the problem of wedge generation seems to be essentially confined to some materials combining high tensile strengths with fine aggregate as the main component.

In other types of test also involving failure of the specimen, such as the indirect tensile fatigue test, the importance of wedge generation is diminished when load-controlled tests are performed as adopted throughout this research. Brown [1996] divides the cracking mechanism of failure into two phases: crack initiation and crack propagation. The latter depends on the stress level at the crack ends and so crack propagation in load-controlled tests is very fast when compared to deformation-controlled tests in which the stress is reduced as the stiffness decreases to maintain the rate of deformation constant. Good repeatability and reproducibility results have been obtained with fatigue tests in indirect tensile mode performed on bituminous materials in the Nottingham Asphalt Tester [Read, 1996].

It has been said that highly variable stress states occur within the specimen in indirect tensile mode and as a result the stiffness modulus quantified in indirect tensile mode is an average modulus representing the composite response of the material [Sousa, Taylor & Tanco, 1991]. It has been counter-argued that for bituminous materials this is true for relatively high temperatures when the behaviour of the material is highly non-linear [Roque & Buttlar, 1992]. In stabilised materials this type of behaviour may be possible for some materials with very light treatment or with unbound type of behaviour for which this test has less potential and interest, whereas the non-linearity and stress-dependency clearly decreases as the level of treatment increases as was demonstrated with the repeated load triaxial test results.

In summary, the indirect tensile test is not perfect, it has a number of advantages and disadvantages as possible substitute tests have, but there is reasonable evidence that it is possible to obtain adequate mechanical properties for use in analytical methods of pavement design when using appropriate methods of testing and analysis. Overall, in terms of tensile strength the results should be very close to the actual values with possible small overestimation. As to stiffness the results obtained should be perfectly representative of the material's real value as long as adequate techniques are followed, in particular, the measurement of deformations in the central part of the specimen.

7.3. Method of analysis

The analysis of test results and the assessment of the mechanical properties is based on the theory of elasticity, assuming that the material is homogeneous, linear elastic and isotropic and that the material is subjected to plane stress conditions. A review of the expressions used is conducted below.

Equations giving the stresses along axis x, perpendicular to the load:

$$\sigma_x = \frac{2\,P}{\pi\,d\,t}\left(\frac{d^2 - 4\,x^2}{d^2 + 4\,x^2}\right)^2 \qquad 7.1$$

$$\sigma_y = -\frac{2\,P}{\pi\,d\,t}\left(\frac{4\,d^4}{(d^2 + 4\,x^2)^2} - 1\right) \qquad 7.2$$

where: σ_x horizontal tensile stress along x-axis;
- σ_y vertical compressive stress along x-axis;
- x distance to the origin (centre of the specimen);
- P vertical load;
- d diameter of the specimen;
- t thickness of the specimen.

The maximum normal stresses occur in the middle of the specimen for x = 0:

$$\sigma_{x\ \max} = \frac{2\,P}{\pi\,d\,t} \qquad 7.3$$

$$\sigma_{y\ \max} = -\frac{6\,P}{\pi\,d\,t} \qquad 7.4$$

where: $\sigma_{x\,max}$ maximum horizontal tensile stress at the centre of the specimen;

$\sigma_{y\,max}$ maximum vertical compressive stress at the centre of the specimen.

For plane stress conditions ($\sigma_z = 0$), Hooke's law governing the behaviour of elastic and isotropic materials is:

$$\begin{Bmatrix} \epsilon_x \\ \epsilon_y \end{Bmatrix} = \frac{1}{E} \begin{bmatrix} 1 & -\nu \\ -\nu & 1 \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_y \end{Bmatrix} \qquad 7.5$$

where: ϵ_x, ϵ_y horizontal and vertical strains;

σ_x, σ_y horizontal and vertical stresses;

E Young's modulus;

ν Poisson's ratio.

As referred before, the term Young's modulus is not generally used in pavement engineering being replaced by resilient modulus or stiffness modulus. To differentiate in relation to the resilient modulus obtained from triaxial testing, stiffness modulus will generally be adopted.

Substituting σ_x and σ_y using Equations *7.1* and *7.2* the following equation is obtained:

$$\epsilon_x = \frac{2\,P}{S_m\,\pi\,d\,t} \left(\frac{4\,d^4\,\nu - 16\,d^2\,x^2}{(d^2 + 4\,x^2)^2} + (1 - \nu) \right) \qquad 7.6$$

where S_m is the stiffness modulus.

The maximum horizontal tensile strain ($\epsilon_{x\,max}$) occurs in the middle of the specimen for $x = 0$:

$$\epsilon_{x\ max} = \frac{\sigma_{x\ max}}{S_m} (1 + 3\ \nu) \qquad 7.7$$

Total horizontal deformation (δ) may be obtained integrating Equation *7.6* over the diameter of the specimen:

$$\delta = \int_{-d/2}^{+d/2} \epsilon_x\ dx \qquad 7.8$$

i.e.

$$\delta = \frac{P}{S_m\ t} \left(\nu + \frac{4}{\pi} - 1 \right) \qquad 7.9$$

Solving Equation *7.9* in relation to the stiffness modulus we obtain:

$$S_m = \frac{P}{t\ \delta} \left(\nu + \frac{4}{\pi} - 1 \right) \qquad 7.10$$

The determination of Poisson's ratio may be done using the following equation derived from Hooke's law (Equation *7.5*):

$$\nu = \frac{\sigma_x - \sigma_y \dfrac{\epsilon_x}{\epsilon_y}}{\sigma_y - \sigma_x \dfrac{\epsilon_x}{\epsilon_y}} \qquad 7.11$$

Considering the relationship between compressive and tensile stresses at the centre of the specimen

$$\sigma_{y\ max} = -3\ \sigma_{x\ max} \tag{7.12}$$

the following equation results for the assessment of Poisson's ratio

$$\nu = -\frac{1 + 3\dfrac{\epsilon_{x\ max}}{\epsilon_{y\ max}}}{3 + \dfrac{\epsilon_{x\ max}}{\epsilon_{y\ max}}} \tag{7.13}$$

where: $\epsilon_{x\ max}$ maximum horizontal tensile strain at the centre of the specimen;

$\epsilon_{y\ max}$ maximum vertical compressive strain at the centre of the specimen.

For this calculation it is either necessary to assume that the average values of the horizontal and vertical deformations measured along the gauge length are a reasonable approximation of the maximum values or convert those values into the maximum point strains necessary for the equation.

Roque & Buttlar [1992] performed a three-dimensional finite element analysis on cylindrical specimens loaded in indirect tensile mode. Some differences were found in relation to the two-dimensional plane-stress results:

- variation in the horizontal tensile stress along the z-axis
- considerable and non-uniform specimen bulging. Both vertical and horizontal deformations were found to be affected by the bulging of specimens under load;
- significant effect of Poisson's ratio on the stress distribution.

Consequently, a new measurement and analysis system has been proposed [Roque & Buttlar, 1992; Buttlar & Roque, 1994]. For the measurement of vertical and horizontal deformations a set of two miniaturised LVDTs is used which are located in the central part of the specimen, ideally in both faces of the specimens. This has the following advantages:

- it contributes to a reduction of measurement errors due to bulging of the specimens; and
- the measurement is carried out in an area where the stresses are almost uniform.

Roque & Buttlar [1992] found that the measurements obtained with this system need to be corrected to take into account three-dimensional effects and specimen bulging. Correction factors were then proposed which depend on Poisson's ratio and the geometry of the specimen. These are presented below:

$$\sigma_{x\,\max} = \frac{2\,P}{\pi\,d\,t}\,C_{SX} \qquad 7.14$$

$$C_{SX} = 0.948 - 0.01114\left(\frac{t}{d}\right) - 0.2693\,\nu + 1.436\,\nu\left(\frac{t}{d}\right) \qquad 7.15$$

and

$$\sigma_{y\,\max} = -\frac{6\,P}{\pi\,d\,t}\,C_{SY} \qquad 7.16$$

$$C_{SY} = 0.901 + 0.138\,\nu + 0.287\left(\frac{t}{d}\right) - 0.251\,\nu\left(\frac{t}{d}\right) - 0.254\left(\frac{t}{d}\right)^2 \qquad 7.17$$

where: C_{SX} correction factor (tensile stress); and

C_{SY} correction factor (compressive stress).

Finally, the horizontal and vertical strains which are average values along the gauge length need to be converted to the maximum point strains at the centre of the specimen (correction factors 1.07 and 0.98 respectively) and also corrected to account for three-dimensional effects (correction factors C_{BX} and C_{BY}). The equations proposed by those authors are:

- for the horizontal tensile strain

$$\epsilon_{x\ \max} = \frac{H_M}{GL}\ 1.07\ C_{BX} \qquad 7.18$$

$$C_{BX} = 1.03 - 0.189\left(\frac{t}{d}\right) - 0.081\ \nu + 0.089\left(\frac{t}{d}\right)^2 \qquad 7.19$$

- and for the vertical compressive strain

$$\epsilon_{y\ \max} = -\frac{Y_M}{GL}\ 0.98\ C_{BY} \qquad 7.20$$

$$C_{BY} = 0.994 - 0.128\ \nu \qquad 7.21$$

where: H_M measured horizontal deformation;
Y_M measured vertical deformation;
GL gauge length;
C_{BX} correction factor (horizontal tensile strain);
C_{BY} correction factor (vertical compressive strain).

The Poisson's ratio is then determined using Equation *7.11* and the stiffness modulus using Hooke's law and the corrected values of stresses and strain:

$$S_m = \frac{1}{\epsilon_{x\ max}} (\sigma_{x\ max} - \nu\ \sigma_{y\ max}) \qquad 7.22$$

The influence of these correction factors is shown in Figure 7.4 for a fly ash specimen, part of the results presented in Table 7.3. It can be seen that the effect on Poisson's ratio is noticeable but, on the other hand, the change in stiffness value can be disregarded.

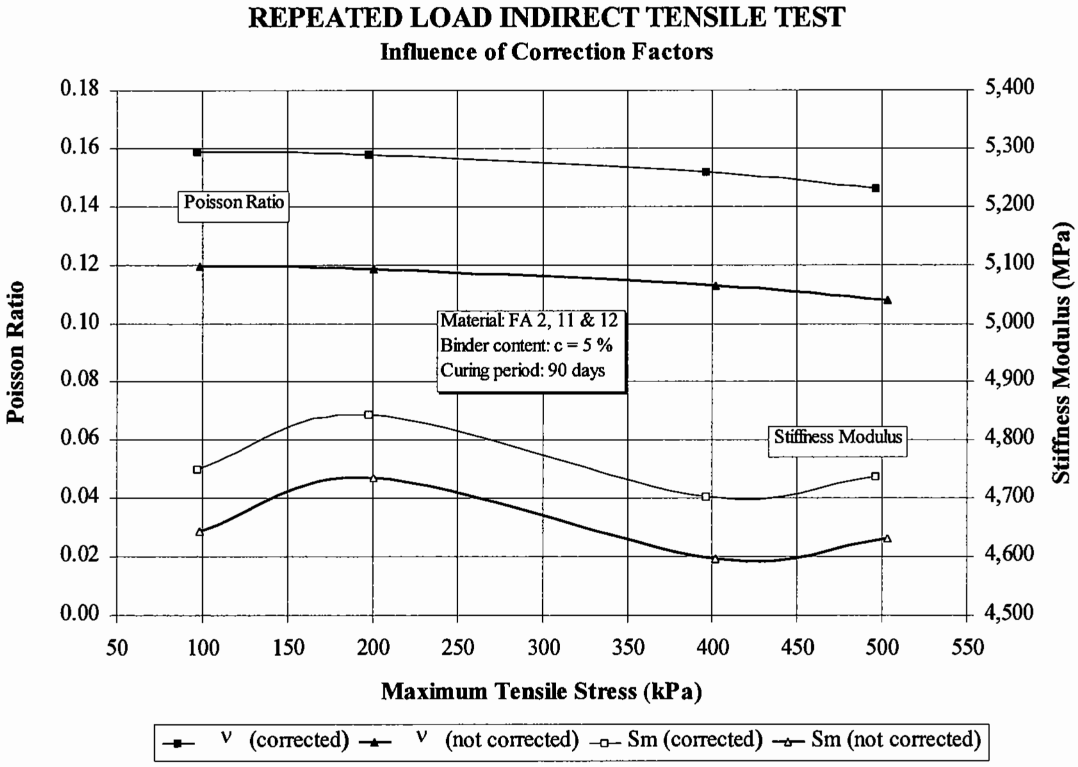

Figure 7.4 Effect of correction factors on Poisson's ratio and stiffness modulus.

7.4. Test specimens

7.4.1. Choice of reference material

The comparison of the indirect tensile test with other performance-based tests such as the repeated load triaxial test is not easy or straightforward due to several factors:

- non-linear behaviour of the materials meaning that the mechanical properties determined are only valid for a particular stress path;
- impossibility of replicating in indirect tensile mode the loading conditions applied in triaxial testing;
- size of the specimens and compaction method.

To attempt this comparison and validate the results obtained in the repeated load indirect tensile tests the following issues would have to be addressed in order to minimise those problems:

- the compacted material should be as homogeneous as possible to minimise the influence of the aggregate structure in the behaviour of the material;
- the material should exhibit a behaviour close to linear, i.e. stress-dependency should not be significant;
- the same compaction procedure should be adopted.

For this purpose the material chosen was pulverised fuel ash stabilised with cement at three different levels of treatment, a material which was to be a focus of the study reported in the subsequent sections. To obtain information on the development of properties with time three different curing periods were also considered: 28, 90 and 360 days. Other materials are referred to so as to illustrate particular issues.

7.4.2. Specimen compaction

The material was compacted in accordance with the technique presented for triaxial testing (Chapter 6), using vibrating table combined with vibrating hammer. The target moisture content and dry density were also the same as determined in the compactibility test.

7.4.3. Storage conditions

After the compaction of the specimens they were left to cure for a pre-determined period according to the testing programme. The method used for curing affects the properties of the material. Therefore, it is important to standardise this procedure in order to simulate the situation in the field in a reasonable manner and enable comparisons between different mixtures.

The method used was as follows:

- immediately after compaction, the specimens (inside their moulds) were carefully sealed to avoid the loss of moisture. This is essential for strength development and may be achieved by using plastic membranes to wrap the specimens tightly;
- sealed specimens were stored in a temperature controlled room at constant temperature of 20 °C;
- after a period of 4 to 7 days the specimens had already developed an initial strength enabling the removal of the moulds, thus releasing them for the compaction of further specimens. The specimen was again adequately sealed and stored at constant temperature (20 °C) until the end of the curing period;
- especially for highly permeable mixtures, it was important to turn specimens up-side down periodically in order to avoid concentration of moisture in the lower half of the specimens which would have resulted in considerable heterogeneity within the specimen.

7.4.4. Sawing and coring of specimens

Before the end of the curing period the triaxial specimens were dry-cut into diametral specimens using a diamond blade. The approximate dimensions of the final specimens were of 150 mm diameter and 70 mm thickness.

The possibility of testing cores from in-service pavements is one of the main advantages of this form of testing. To assess the best technique for coring hydraulic bound materials *in situ*, 100 mm diameter specimens were cored from the middle of 150 mm triaxial specimens and used in the initial stages of this research to conduct a study on the influence of load risetime on the properties of the materials.

The coring is generally more difficult than for bituminous materials but it was successful even for a material as brittle as stabilised fly ash, being easier for other mixtures with an aggregate as a main component. There are two aspects that resulted from this trial:

- principally for the weaker materials the coring, as well as the cutting, should be conducted in dry conditions using appropriate core drills and diamond blades. In some cases, the use of water may result in permanent damage of the core and in any case it is likely to affect the materials' properties measured during test;
- to avoid the damage of the core being drilled it is important to use vacuum (or air pressure) to remove the dust from the cutting surface.

7.5. Indirect tensile strength

7.5.1. Test method

The strength tests are performed using monotonic loading at a constant speed of 10 kPa/s until failure occurs. Values of the vertical load and vertical deformation are constantly recorded into the computer throughout the test. The vertical deformation is obtained from a pair of LVDTs measuring the displacements of the upper loading platen. Due to the effect of stress concentration in the proximity of the loading strips, that deformation is not considered in any analysis to calculate the mechanical properties of the material. Instead, it is used to control the test, via a personal computer, and automatically stop the test when the specimen fails. For test control purposes, failure is defined as corresponding to a vertical deformation of 10 mm.

The indirect tensile strength is calculated using Equation *7.3* and the average of at least 3 specimens (preferably 5) should be considered.

7.5.2. Results

The indirect tensile strength results obtained are presented in Table 7.2 and Figure 7.5, each row in Table 7.2 being derived from 3 specimens.

The drawing of stabilisation curves allows the optimisation of mixtures in accordance with the properties required for the material under study. Figure 7.5 shows the stabilisation curve for each of the curing periods considered. It can be seen that though the tensile strength continues increasing with the percentage of binder, the improvement in strength decreases as the level of treatment increases. Economic analyses of the materials are desirable to evaluate whether or not the extra cost in binder is compensated by, for instance, an economically advantageous reduction in thickness of a pavement layer built from it.

Table 7.2 Results of the indirect tensile strength tests.

Material			**Tensile Strength**	**Ratios** [1]	**Standard Deviation**	**Coefficient of Variation**
Major Component	**Binder Content**	**Curing Period**				
	%	days	kPa		kPa	%
Fly Ash	2	28	213		29.4	13.4
	2	90	350	1.64	14.2	4.1
	2	360	400	1.14	68.4	15.6
Fly Ash	5	28	510		72.9	12.8
	5	90	752	1.47	79.6	11.8
	5	360	779	1.04	131.7	14.6
Fly Ash	9	28	654		80.8	14.0
	9	90	1032	1.58	11.7	1.1
	9	360	1097	1.06	-----	-----

Notes: [1] ratio @ 90 days = $\sigma_{ti\,(90\ days)} / \sigma_{ti\,(28\ days)}$

ratio @ 360 days = $\sigma_{ti\,(360\ days)} / \sigma_{ti\,(90\ days)}$

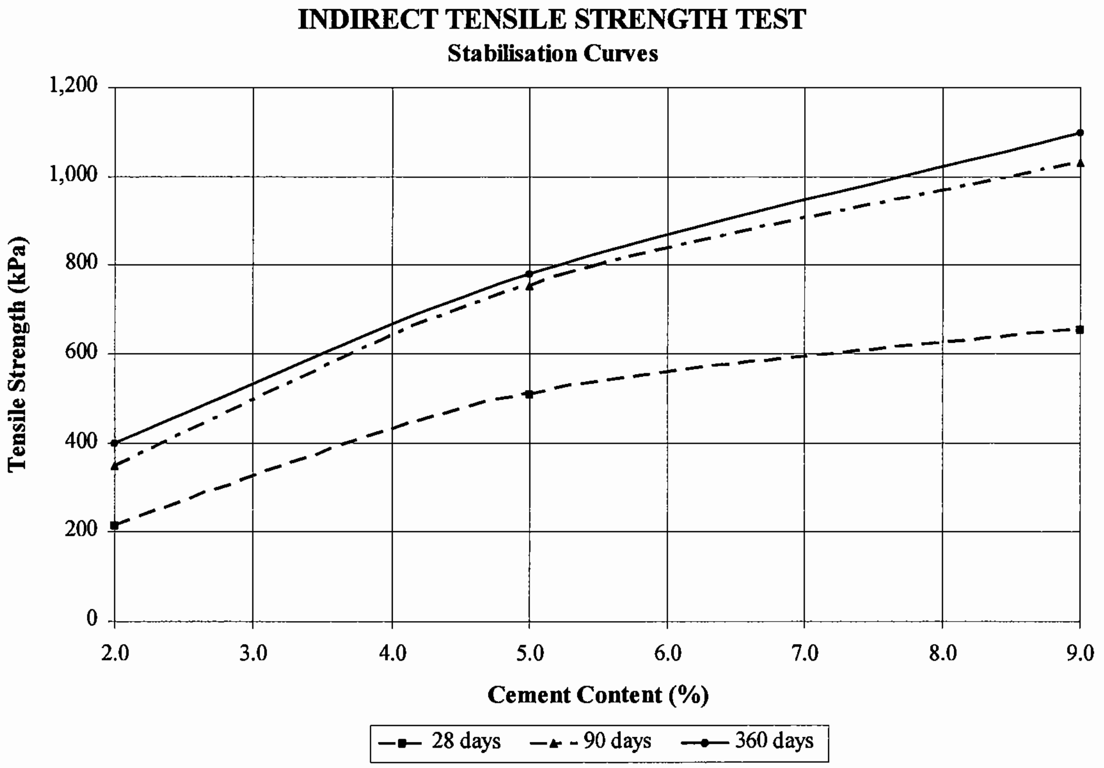

Figure 7.5 Stabilisation curves in terms of indirect tensile strength (fly ash).

The ratio between tensile strength at 360 and 90 days appears to be considerably lower than 1.5 proposed for lime activated fly ash bound mixtures [CEN, 1995 i]. It may be observed from the stabilisation curves that for this particular material the strength development process until 90 days of curing occurs at a significantly higher rate than afterwards from 90 until 360 days. For this reason, a characterisation of this material at 90 days seems to be adequate but one should bear in mind that the strength continues increasing with time.

The coefficients of variation (Cv) obtained, of up to 15 %, are common in laboratory testing taking into account that the values obtained were the average of only three results. Some values deduced from the literature are:

- *Concrete* - Cv of 20 % for 20 or more results and 40 % for less than 20 results are indicated for mix design purposes [BRE, 1988];
- *Concrete* - Cv up to 20 % for the compressive strength of concrete [Neville, 1995];
- *Soils* - Cv up to 20 %, on reproducibility and repeatability testing [Dawson & Bullen, 1991];
- *Cement bound materials (CBM)* - in terms of strength, the DoT requirement for CBM1 is a minimum average 7 day cube compressive strength of 4.5 MPa, with individual minimum of 2.5 MPa [DoT, 1993]. It is numerically possible to comply with this requirement with test results having coefficients of variation up to 40 %. Even higher Cv may be possible if the average value covers the full range of possible CBM1 strengths, i.e. above 4.5 and below 7.0 MPa;
- *Gravels and sands treated with hydraulic binders* - the direct tensile strength is determined in the French Standards [NF P 98-232, 1992] as the average of three results unless one value is lower than the average of the other two by more than 50 %. This limit corresponds to a Cv of 57 %.

It appears from the last two points that with the aim of simplifying the content of Standards for an easier execution of tests and analysis of results, very high coefficients of variation are actually being allowed, in fact much higher than what may be practically achieved. The use of Cv values may be a better method of controlling the variability of material properties, also being derived from a very basic calculation.

7.6. Poisson's ratio

7.6.1. Test method

The technique used for the determination of Poisson's ratio was adopted from the methodology proposed for bituminous materials by Roque & Buttlar [1992] and more directly from the recent research conducted by Read [1996]. A brief summary of the technique is described here. Poisson's ratio was not routinely assessed in this research, but its determination was carried out on selected specimens for test validation purposes.

The measuring system comprises a set of two miniaturised LVDTs to measure both the vertical and horizontal deformations. Their positioning and gauge length had to be chosen taking into consideration aspects such as maximum particle size, relative uniformity of the stresses and at the same time minimise the bulging effects. The LVDTs are located in the central part of the specimen, where the stress and strain distribution is fairly constant, and supported by four aluminium locating pips glued to one of the flat surfaces of the cylindrical specimen. The gauge length considered was of 38.1 mm (1.5 inch) which enables the use of the correction factors proposed by Roque & Buttlar [1992] to account for three-dimensional effects and specimen bulging. Figure 7.6 shows a detail of the system.

The technique was found to work well with stabilised materials and no modifications to the system were considered essential apart from the necessary adaptations to the bigger 150-mm-diameter specimens used in this work.

The analysis is conducted using equation *7.11* or, considering the point strains at the centre of the specimen, equation *7.13* which leads to identical results. As referred before the appropriate correction factors should be used. Since they are a function of the geometry of the specimen and of Poisson's ratio, an iterative process is necessary to determine the final value. However, the convergence process is extremely fast and does not represent a major obstacle. Indeed, the final result is not very sensitive to small variations in the value of Poisson's ratio and one to two iterations are generally enough to obtain the final solution.

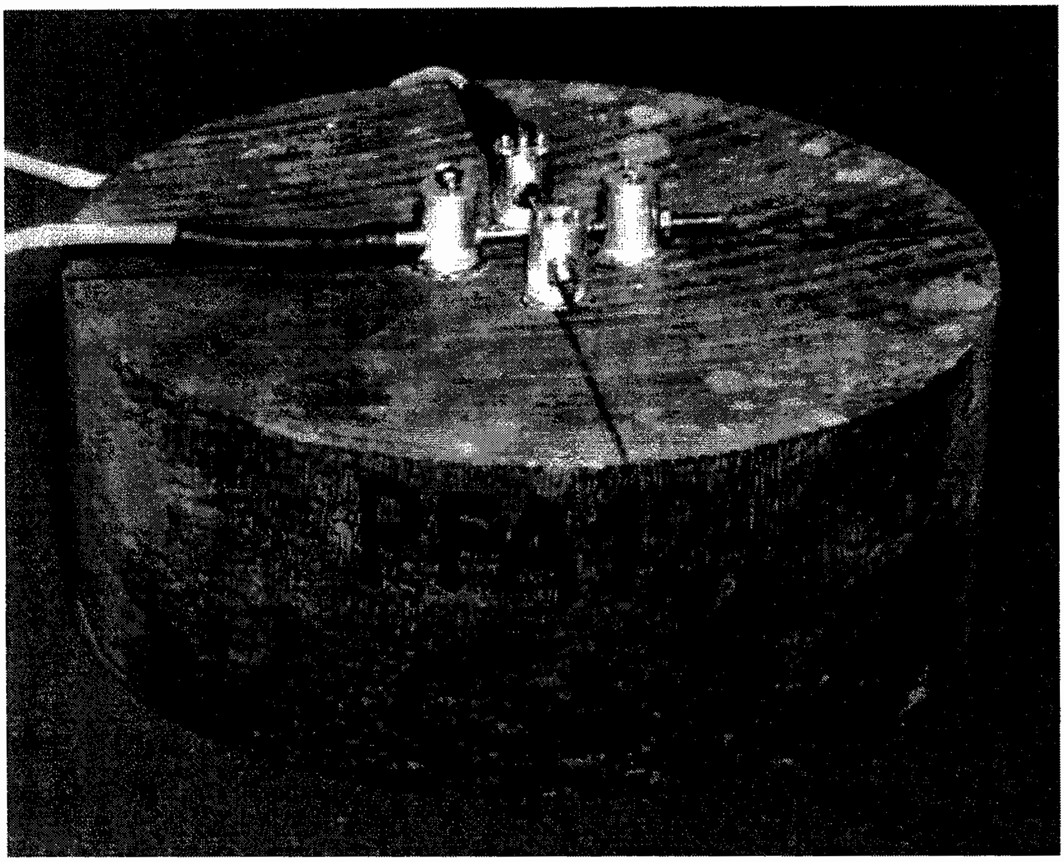

Figure 7.6 Measuring system used for determining Poisson's ratio.

7.6.2. Results

The evaluation of Poisson's ratio was performed for fly ash stabilised with 2 and 5 % of cement after 90 days of curing. The results are shown in Figure 7.7. A comparison with the values obtained in the repeated load triaxial tests, 0.22 and 0.19 for 2 and 5 % respectively (see Chapter 6, Section 6.6.7) seems to indicate a small underestimation of the results. There are a number of factors that may have contributed to this difference:

- triaxial tests were performed at 28 days and slightly lower Poisson's ratio values could have been obtained if tested at 90 days;

- lower reliability of the hoops for measuring very small radial deformations when compared to the miniaturised LVDTs;
- general difficulty in comparing triaxial and indirect tensile results due to the different stress conditions and stress paths applied during the tests;
- although there is some evidence that elastic / resilient properties in compression and in tension are the same this is not yet unanimously accepted.

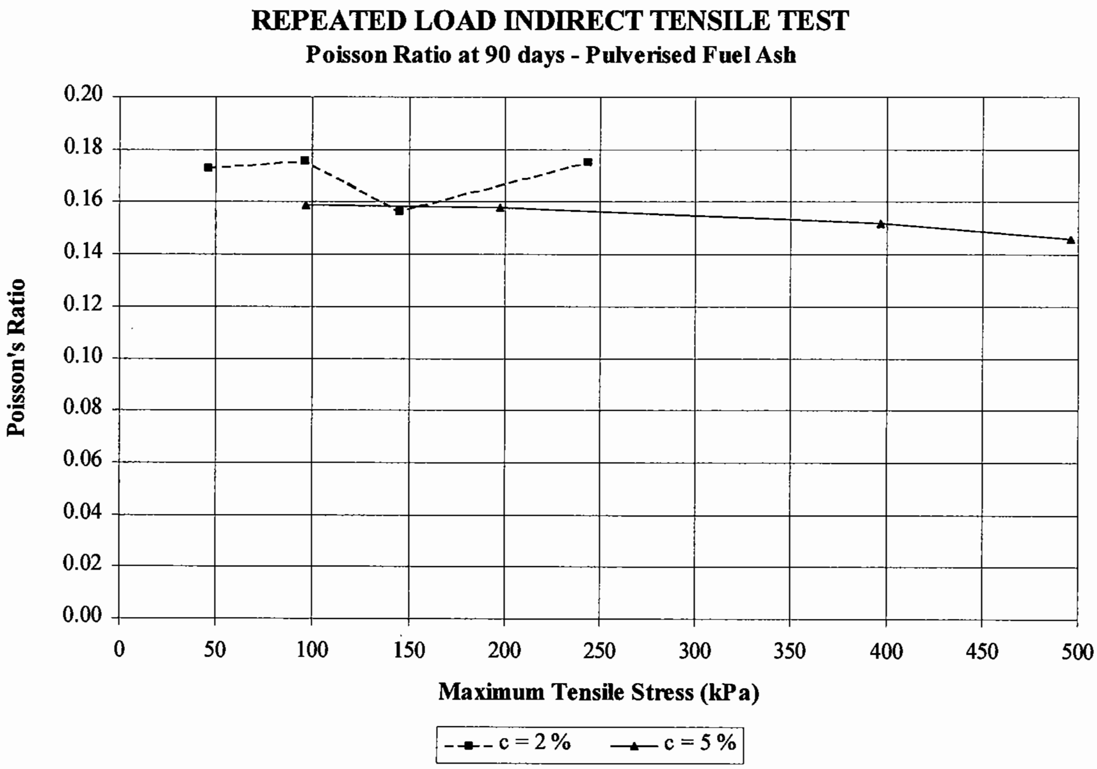

Figure 7.7 Poisson's ratio results (fly ash).

In summary, the values obtained appear to be reasonable and help to validate the technique used and the repeated load indirect tensile test as a performance-based test.

7.7. Stiffness modulus

There are a number of specifications covering indirect tensile testing for the determination of strength [ASTM C496, 1990; BS 1881, 1983], stiffness modulus for bituminous materials [ASTM D4123, 1987; BS DD 213, 1993] and fatigue characteristics of bituminous mixtures [BS DD ABF, 1996]. Although the main criticisms of the indirect tensile test are actually applied to the determination of tensile strength due to the non-linearity and non-elasticity of the material when approaching failure, no standard has yet been developed for the determination of stiffness modulus in indirect tensile mode of materials treated with hydraulic binders. This represents an important lacuna in the present specifications that this research will try to cover.

7.7.1. Test method

The evaluation of stiffness is carried out after performing a failure test on one specimen of the same mixture to quantify the ultimate tensile strength of the material. For the stiffness modulus test the maximum load to be applied is limited to approximately 60 % of that failure value, for assurance that the material will be performing in the elastic range. For some of the weaker mixtures the value of 70 % generally accepted for unbound granular materials seems to cause permanent deformations in the specimen.

Before testing for stiffness modulus the material should be conditioned using the highest stress level that will be used during the test. This conditioning is conducted by the application of 50 to 200 load repetitions. This phase was found to be important for a resilient response of the material and virtually eliminated the accumulation of permanent deformations during stiffness testing. This aspect is illustrated in Figures 7.8 and 7.9. It can be seen from these figures that the stiffness during the conditioning phase changes by a varying amount, depending on the characteristics of the material, but remains broadly constant during subsequent stiffness testing.

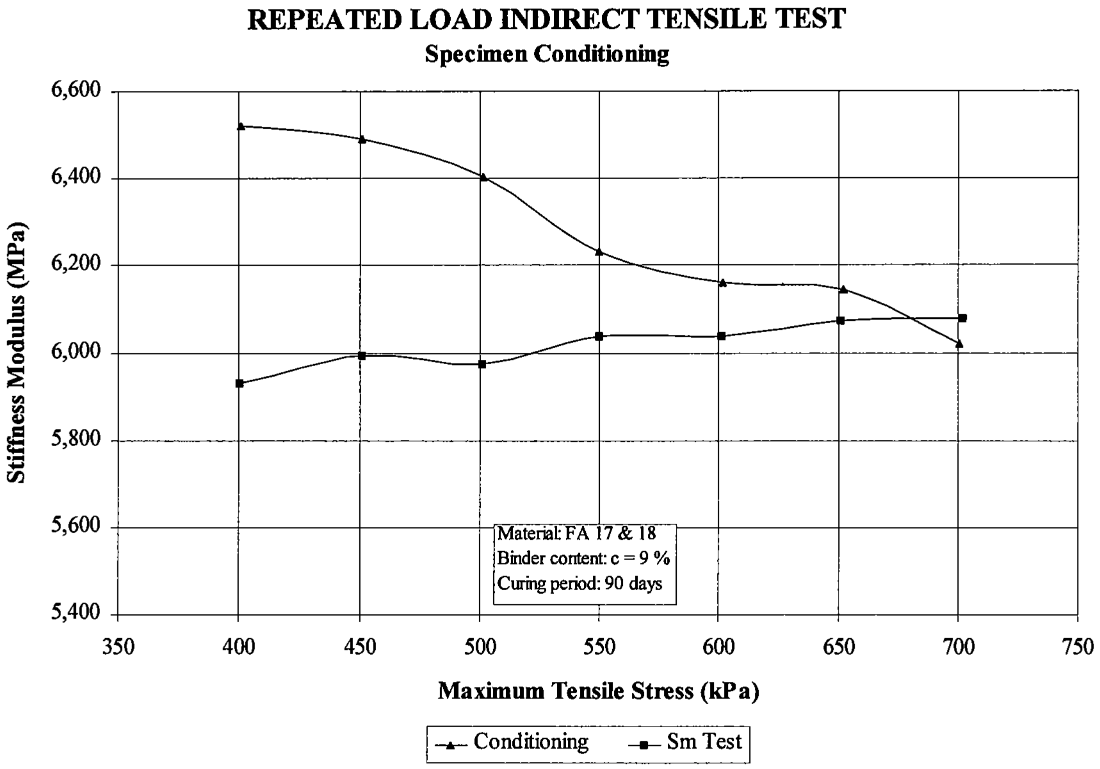

Figure 7.8 Conditioning phase (pulverised fuel ash).

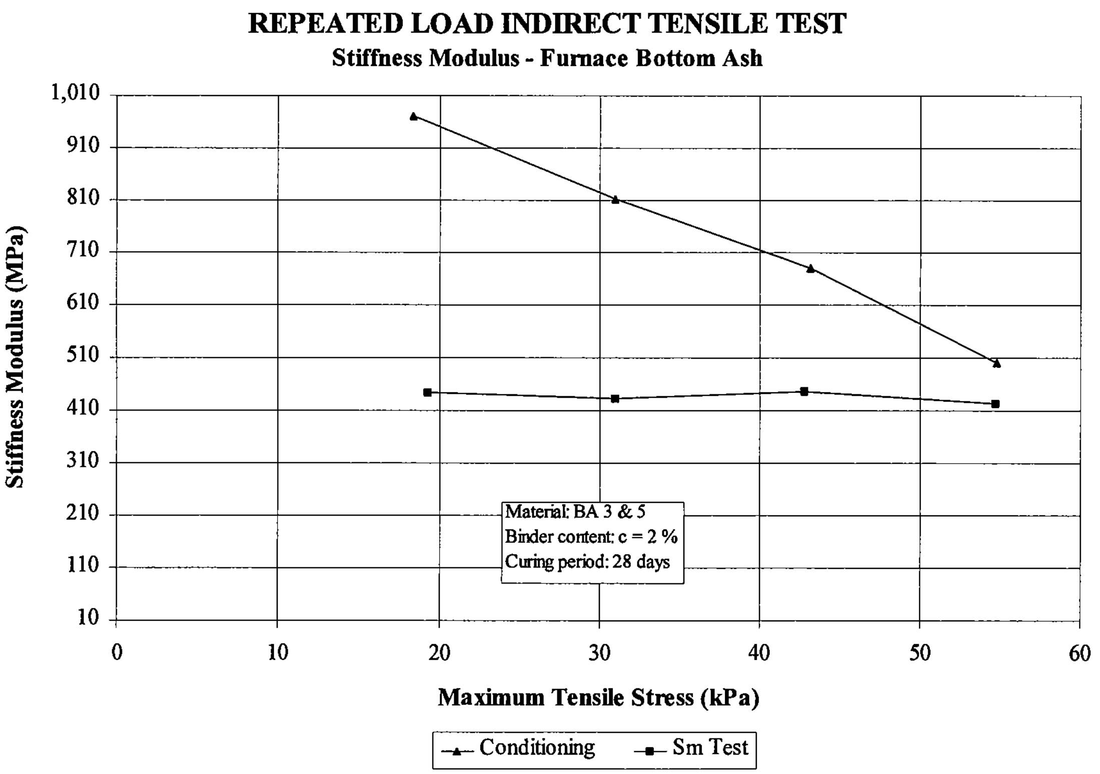

Figure 7.9 Conditioning phase (furnace bottom ash).

After conditioning, five to six increasing stress levels are used and for each one the load is applied repeatedly for a total of 15 cycles. The values of vertical load and horizontal deformation are constantly recorded throughout the test. Indicative values of the stress levels to be applied are: 20, 30, 40, 50 and 60 % of the tensile strength previously obtained. A second test is then carried out after rotating the specimen through approximately 90°.

For the analysis of results two methods are considered depending on the method used to measure the deformations:

- *Horizontal and vertical deformation measured using miniaturised LVDTs located in the central part of the specimen* (as Figure 7.6)
  Theoretically, this is the preferred method since the deformations are measured in an area where the stresses are broadly constant. The stiffness modulus is calculated using equation *7.22*;
- *Total horizontal (diametral) deformation measured using two diametrically opposed LVDTs mounted in an aluminium frame*
  This was found to be an acceptable alternative that may be used when the first system is not available or to simplify and accelerate the tests. Its main disadvantage is not allowing the evaluation of Poisson's ratio which will need to be assumed. Owing to the similar stiffness modulus results obtained by both procedures, it is believed that the best operational method is to use the system of miniaturised LVDTs to make an initial evaluation of the Poisson's ratio for a particular material and further assessments of stiffness modulus to be conducted using the LVDTs in aluminium frame. The stiffness modulus is calculated using equation *7.10.*

The results of the stiffness modulus are plotted against the maximum tensile stress at the centre of the specimen. If the material is stress dependent then the characteristic stiffness modulus is defined by interpolation as the one corresponding to a tensile stress level of 50 % of the tensile strength, which is not the 'real' stiffness but representative of performance at likely stress levels in pavements. If stiffness remains approximately constant then an average value is calculated. This procedure is followed for the tests performed along both axes and their average is reported as the stiffness modulus of the material.

Effect of frequency of loading

In an initial stage, a study was conducted in the conventional Nottingham Asphalt Tester (NAT) using 100 mm specimens and varying the risetime within the range allowed by the software controlling the test, i.e. 60 to 160 ms. The results are reproduced in Figure 7.10. It can be seen that for the range of risetimes considered the stiffness modulus is not affected significantly. Nevertheless, it is apparent that for the lower risetimes the scatter increases and thus risetimes above 120 ms (value currently used for bituminous materials) should be used. Limitations of the instrumentation to measure the load and deformations at faster rates of load applications may be responsible for the increase of scatter.

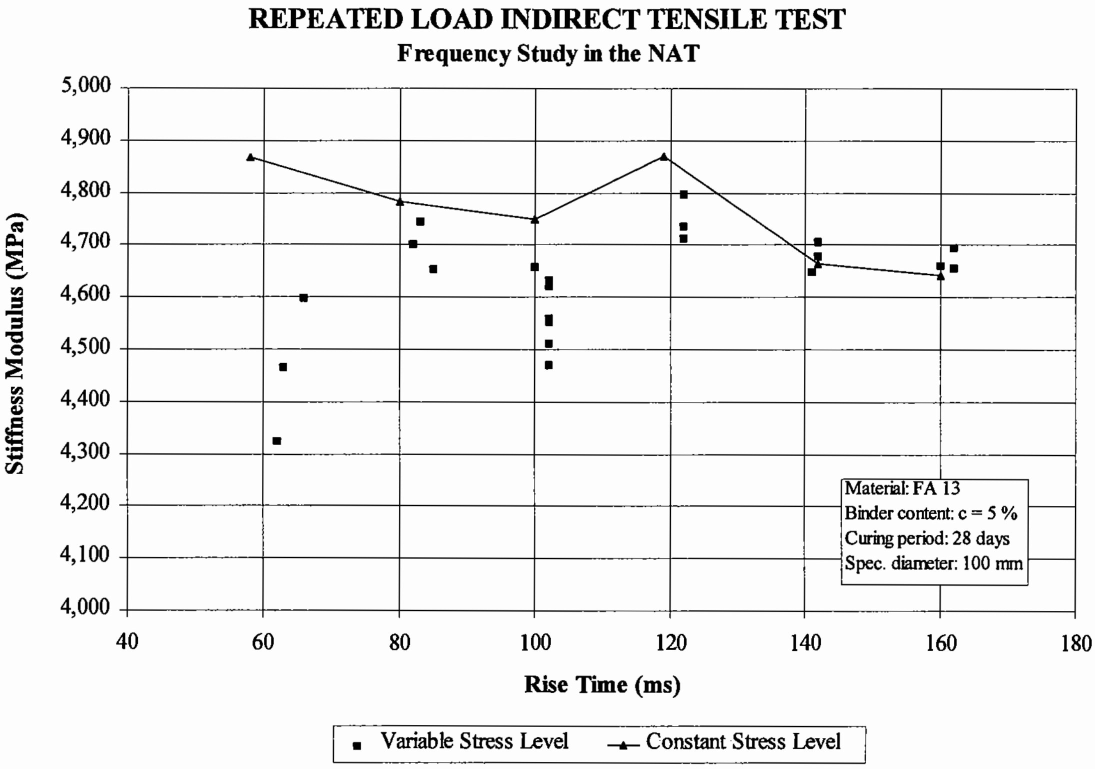

Figure 7.10 Frequency study in the Nottingham Asphalt Tester.

An extension of the previous study was conducted with 150 mm specimens in the modified NAT where a broader range of frequencies is possible. The frequencies considered were 0.1, 0.5 and 5 Hz and the results are shown in Figure 7.11. It is evident that even for this wider range the effect on stiffness modulus can be disregarded and the variations observed are compatible with the ones observed in the repeatability study (Chapter 5).

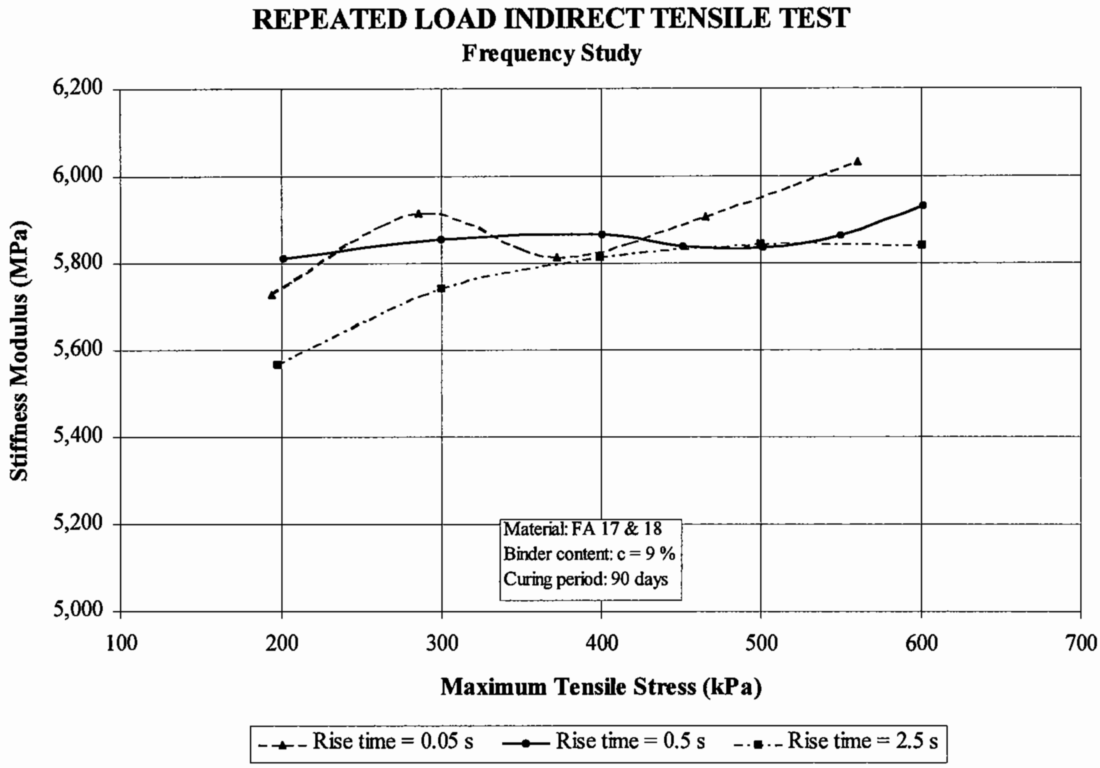

Figure 7.11 Effect of frequency of loading on stiffness modulus.

The choice of frequency of loading should consider the following aspects:

- it should be representative of transient pavement loading;
- it should be as high as possible to reduce the duration of testing, in particular for fatigue testing (described later in this chapter);
- it needs to permit a reliable data acquisition and assessment of properties. This implies lower frequencies than desirable to satisfy the other two points.

In light of these considerations, and taking into account the results obtained in the frequency studies, the risetime considered for testing was 125 ms (corresponding to a frequency of 2 Hz) which is similar to the one generally considered for bituminous materials (120 ms).

7.7.2. Results

An illustration of the behaviour of the materials stabilised with hydraulic binders under repeated loading in indirect tensile mode is presented in Figures 7.12 to 7.14. The deformations plotted in these figures were measured in the central part of the specimen using the system of miniaturised LVDTs. It can be seen that in this area where the stresses are broadly constant and for the stress levels used for stiffness modulus determination the behaviour of the material is resilient and almost linear and therefore the application of the equations derived from the theory of elasticity is acceptable. This behaviour is observed both in tension and compression and for all five stress levels applied (Figure 7.14) although higher scatter exists for lower stress levels and lower deformations.

The results obtained for the stiffness modulus are given in Table 7.3 and the corresponding stabilisation curves plotted in Figure 7.15 (when Poisson's ratio was not experimentally determined a constant value equal to 0.15 was assumed).

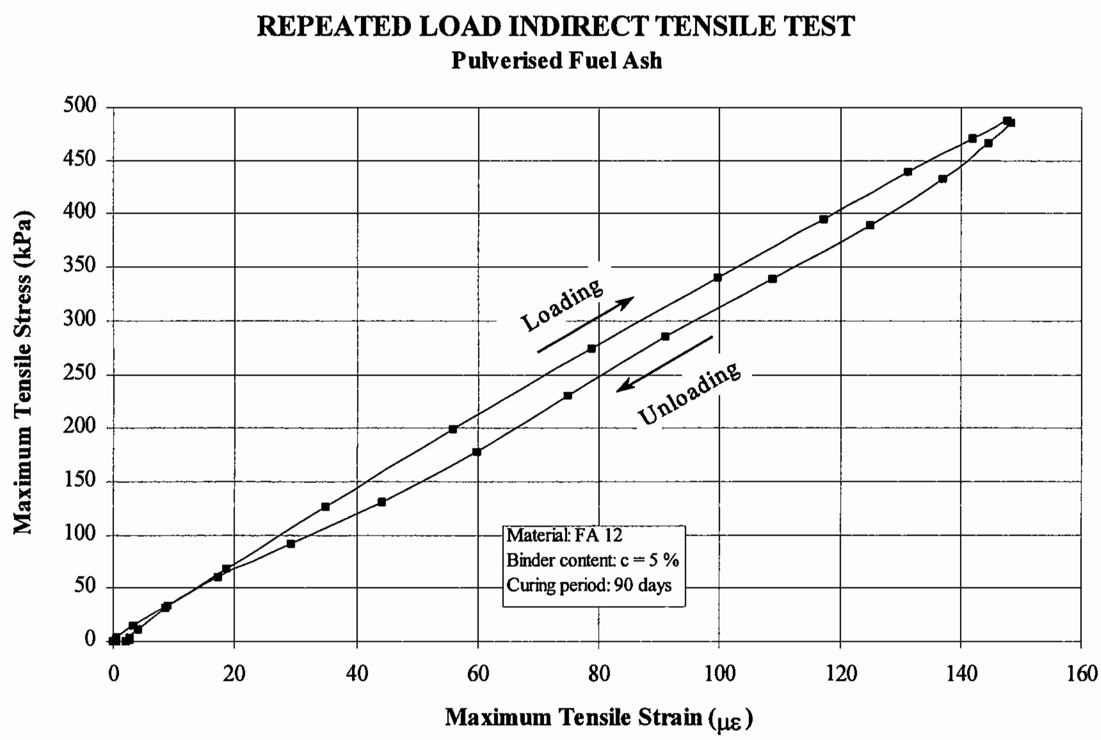

Figure 7.12 Typical resilient behaviour in tension (indirect tensile mode).

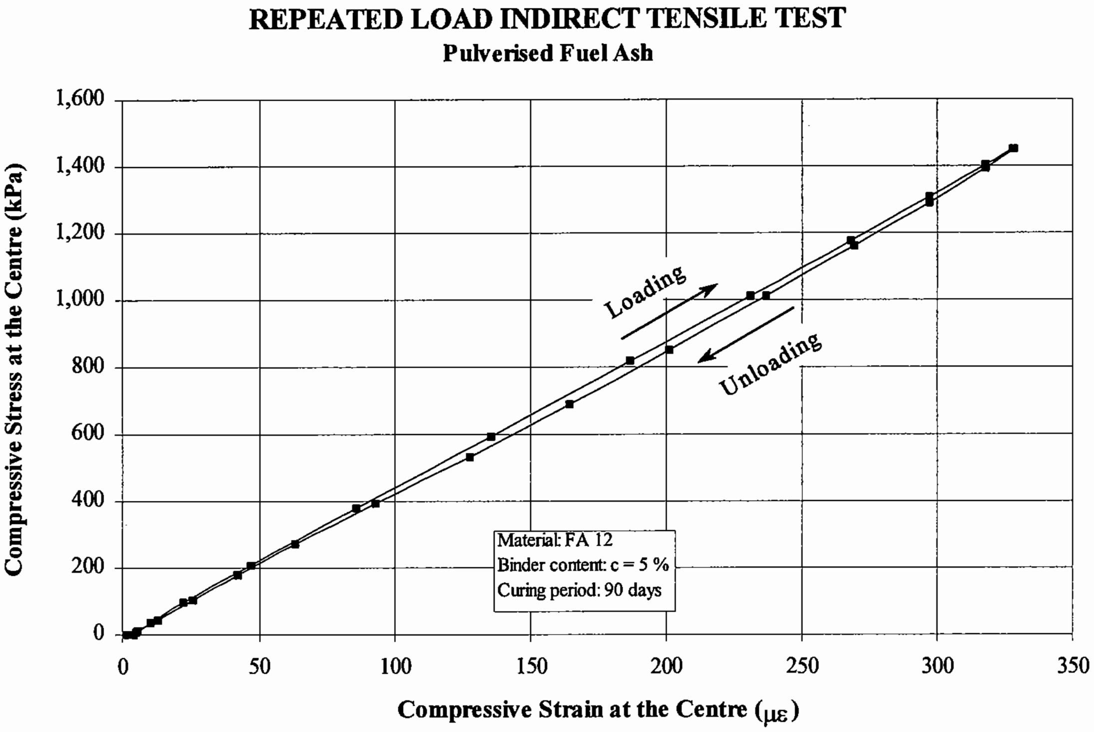

Figure 7.13 Typical resilient behaviour in compression (indirect tensile mode).

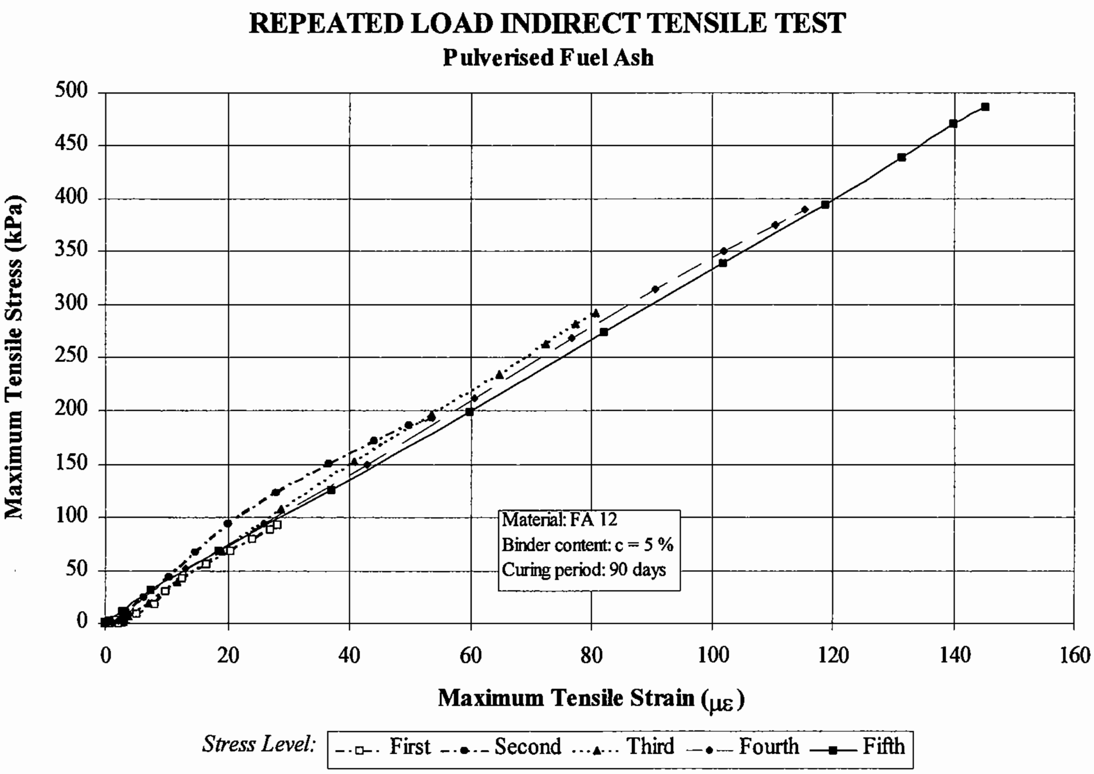

Figure 7.14 Tensile stress / strain behaviour at different stress levels.

Stiffness continues to increase with the percentage of binder but with diminishing levels of improvement similar to what was observed for tensile strength (Figure 7.5), though in this case that is more evident at 360 days. The effect of an increase in binder content is stronger on tensile strength than on stiffness modulus.

A comparison of the results obtained for the same material at 28 days in the repeated load triaxial tests (see Section 6.6.7) is summarised in Table 7.4. First, it should be emphasised that the values obtained in indirect tensile mode agree reasonably well with the ones obtained in triaxial mode, the results being of the same magnitude. Although the stiffness values obtained in the indirect tensile tests are less than those obtained in the triaxial tests, a second computation of the indirect tensile stiffness using the Poisson's ratio obtained in the triaxial tests gives a very close agreement between both tests. The difference in Poisson's ratio was already discussed in the previous section. These results seem to further validate the repeated load indirect tensile test as a performance-based test capable of determining the mechanical properties of pavement materials stabilised with hydraulic binders for design purposes.

Table 7.3 Results of the indirect tensile stiffness modulus.

Material			Stiffness Modulus	Ratios [(1)]	Standard Deviation	Coefficient of Variation
Major Component	**Binder Content**	**Curing Period**				
	%	days	MPa		MPa	%
Fly Ash	2	28	2881		299.0	11.7
	2	90	3402	1.18	474.7	14.0
	2	360	4510	1.33	518.8	11.5
Fly Ash	5	28	4277		416.9	9.7
	5	90	4631	1.08	467.7	10.3
	5	360	5838	1.26	504.4	8.6
Fly Ash	9	28	5975		710.4	11.9
	9	90	6322	1.06	319.1	6.7
	9	360	6609	1.05	513.1	7.8

Notes: [(1)] ratio @ 90 days = $S_{m\ (90\ days)} / S_{m\ (28\ days)}$
ratio @ 360 days = $S_{m\ (360\ days)} / S_{m\ (90\ days)}$

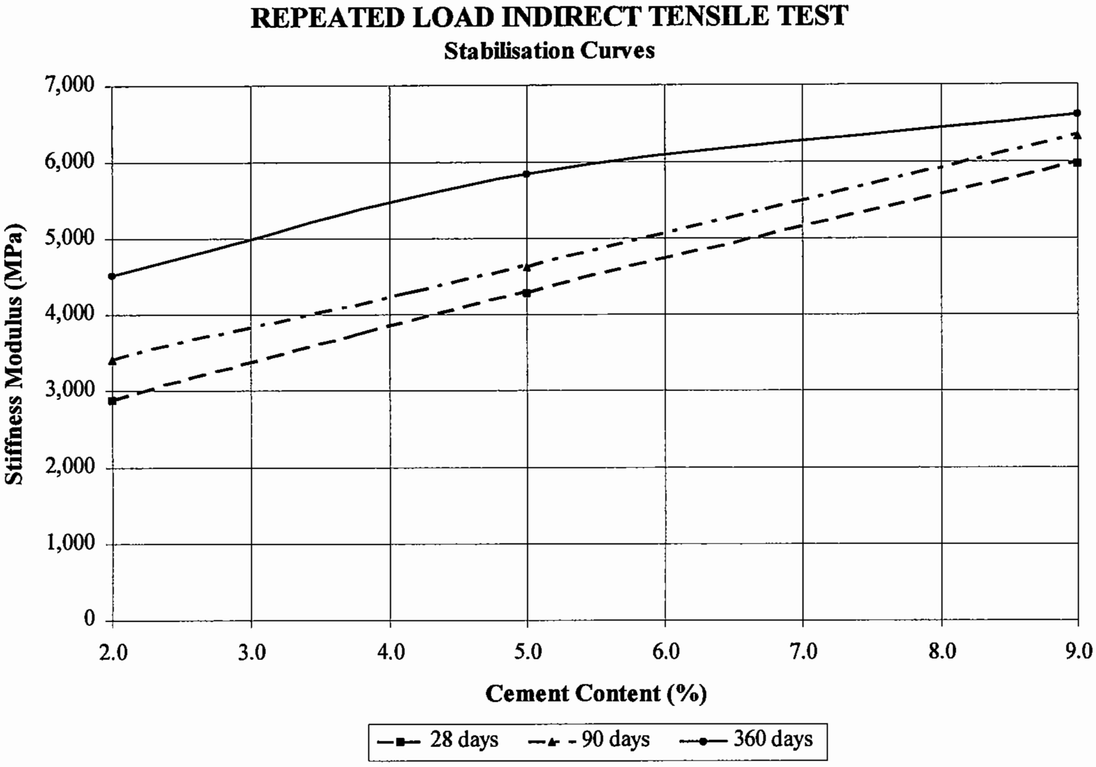

Figure 7.15 Stabilisation curves in terms of stiffness modulus (fly ash).

Table 7.4 Comparison of results obtained in triaxial and indirect tensile testing.

Material			Indirect Tensile Testing			Triaxial Testing	
Major Comp	Binder	Curing	Sm	ν	Sm$_2$ [(1)]	Sm	ν
	%	days	MPa		MPa	MPa	
Fly Ash	2	28	2881	0.17	3206	3296	0.22
Fly Ash	5	28	4277	0.15	4681	4711	0.19

Notes: [(1)] Sm$_2$ computed using Poisson's ratio obtained in the repeated load triaxial tests.

7.8. Durability

7.8.1. Test method

With respect to cement bound materials, the Specification for Highway Works [DoT, 1993] stipulates the determination of the effect of immersion in water on the compressive strength as a way of evaluating the durability of the materials. It is thought that this test should be able to detect the presence of any harmful component within the stabilised material and its effect quantified by the index R_i, defined as the compressive strength of the immersed specimens as a percentage of the compressive strength of the control specimens.

This is not always the case and, for example, specimens of cement bound minestone were shown not to stop expanding at the end of the immersion period [Thomas, 1986; Thomas, Kettle & Morton, 1989]. On account of this, it was concluded that the determination of the complete effect of changes in moisture content after short periods of immersion was improbable. Nonetheless, despite its inability to evaluate the full extent of the expansion process, the presence of detrimental components causing significant expansion was clearly identified. Whenever necessary, once the presence of those components is initially detected, it will be possible to assess the material in more depth by further testing. Furthermore, the immersion test seems to be more representative of the conditions in the pavement than other durability tests used such as wetting-drying and freeze-thaw tests. These tests have been criticised as being excessively conservative for many geographic areas as a basis for specifying a design strength [Saylak, Taha & Little, 1988].

Another approach is specified in relation to lime stabilised materials [BS 1924, 1990]. While the normal curing period for cement stabilised materials is considered to be 7 days, the period for materials stabilised with lime is considered to be 28 days and, therefore, the immersion tests should be performed on specimens cured for 28 days at a constant temperature of 20 °C followed by 7 days immersion in water (compared with 35 days standard curing).

When the use of secondary materials and binders is being studied, some of the mixtures will develop strength at a slow rate. Consequently, those mixes will not have developed enough strength after 7 days of curing. Additionally, the immersion in water in the early age may be harmful for the strength development process or for the specimen integrity.

Thus, for this project the normal durability test for cement bound materials is considered to be inadequate for the study of secondary aggregates and binders. Instead, it was proposed that durability tests be performed either on

- specimens cured at 20 °C for 21 days + immersion in water at 20 °C for 7 days. This hypothesis will allow testing at 28 days and direct comparison with specimens cured in a temperature controlled room at 20 °C; or
- specimens cured at 20 °C for 83 days + immersion in water at 20 °C for 7 days. In this case tests will be performed at 90 days allowing direct comparison with specimens cured in a temperature controlled room at 20 °C for the same period.

A question still remains as to whether testing at 28 days is completely fair to all types of possible mixtures including secondary aggregates and binders and whether it will evidence their true potential. If the answer is negative then it would probably be better to test them after longer curing periods. This is also in accordance with the type of curing period proposed for the European Standards [CEN, 1995 i & ii] where the mechanical performance of fly ash bound mixtures is assessed in the laboratory using strength and/or stiffness modulus values at 360 days. These may be estimated from the 90 days results by using empirical relationships (mainly valid for lime activated mixes) proposed in the same standard drafts.

There are also two other variables that can be introduced in the formulation of a durability test. These are the period of immersion and the temperature used. Strength after long-term (21 days) and short-term (8 days) immersion have been considered as alternatives to other durability indicators such as the vacuum-saturation test and freeze-thaw test [Allen, Currin & Little, 1977].

Different temperatures have also been used to achieve an accelerated ageing of the specimens:

- by rapid cure at 49 °C for 30 hours as a substitute for a normal cure [Allen, Currin & Little, 1977];
- in swelling tests on steel slags where the material is immersed in water at 50 °C [CRR, 1993];
- in disintegration tests performed on steel slags involving the immersion in water at 60 °C [CRR, 1993] or 70 °C [Gorlé, Verhasselt, Thijs & Kerkhof ,1992];
- to assess whether a particular soil is suitable for stabilisation or not a test has been developed in France involving curing at 90 °C, for 24 hours, in a saturating atmosphere [PIARC, 1991].

A first study on durability was then conducted on cement-stabilised fly ash and lime-stabilised minestone. The following conditions were considered:

- curing period extended to 360 days for a full evaluation of the materials' true potential (at 20 °C);
- two periods of immersion, 7 and 21 days, using two different temperatures for each period (20 and 50 °C);
- the control specimens continued curing at 20 °C for the same curing periods (7 and 21 days respectively).

To evaluate the effect of immersion on properties more related to the performance of pavements the conventional compressive stress test is substituted by the indirect tensile test and the effect of immersion is assessed as follows:

- quantify the ratio of tensile strength of immersed specimens over tensile strength of the control specimens;
- quantify the ratio immersed / control stiffness modulus;
- the occurrence of visible cracking and measurable expansion.

The ratios of 'immersed' / 'control' both in terms of tensile strength and stiffness modulus should not drop below 75 to 80 %. This compares with the value of 80 % specified for the immersion test [DoT, 1993].

The assessment of the soaked specimens is performed immediately after removing the specimens from water and drying the free surface water. In this research it has been observed that leaving the specimen for a period of time, for instance two hours, may be enough for the specimen to start recovering from the period of immersion and its full effects not properly quantified.

7.8.2. Results

The results obtained are presented in Figures 7.16 to 7.19. The number of specimens was kept to a minimum due to the extremely long curing period considered and not all binder levels were tested at all testing conditions. Also just one set of control specimens was considered per material which continued curing at 20 °C.

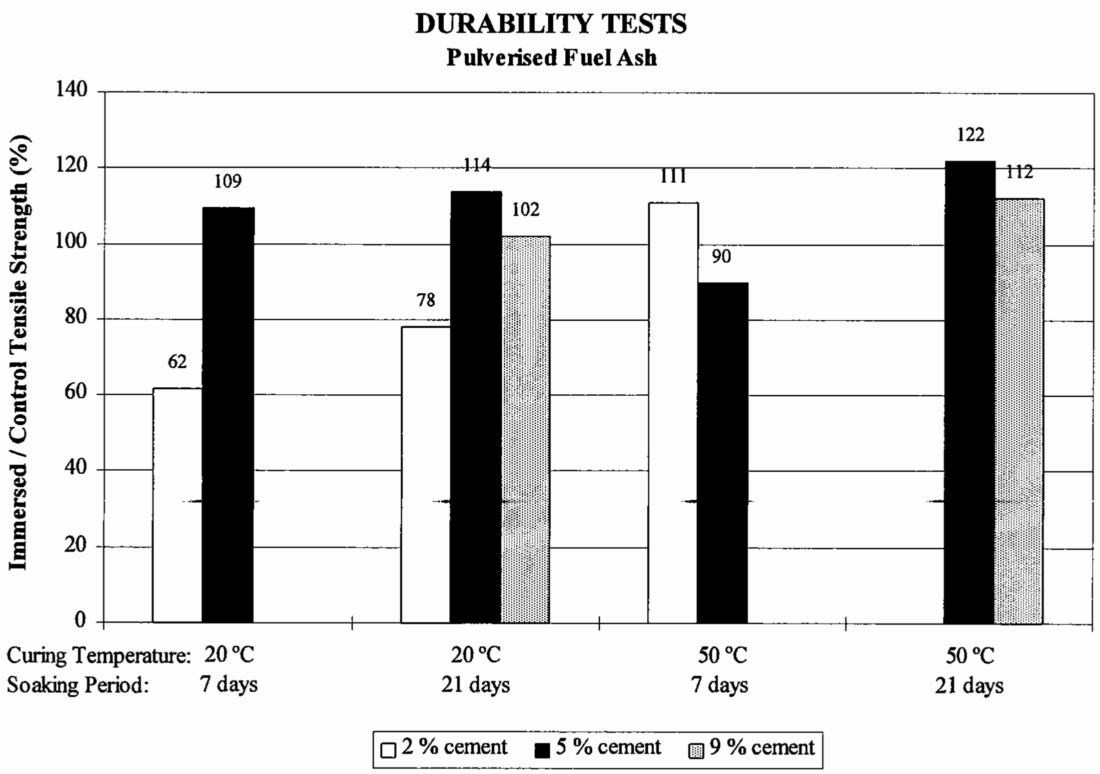

Figure 7.16 Durability results of cement-stabilised fly ash (tensile strength).

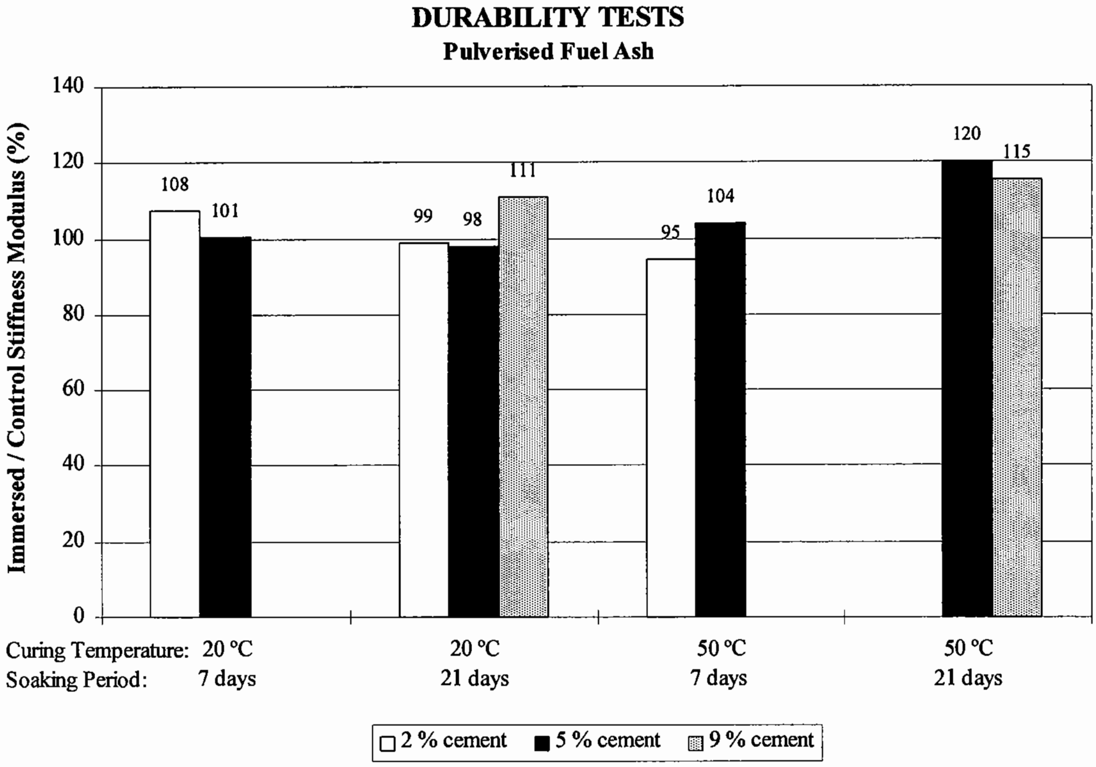

Figure 7.17 Durability results of cement-stabilised fly ash (stiffness modulus).

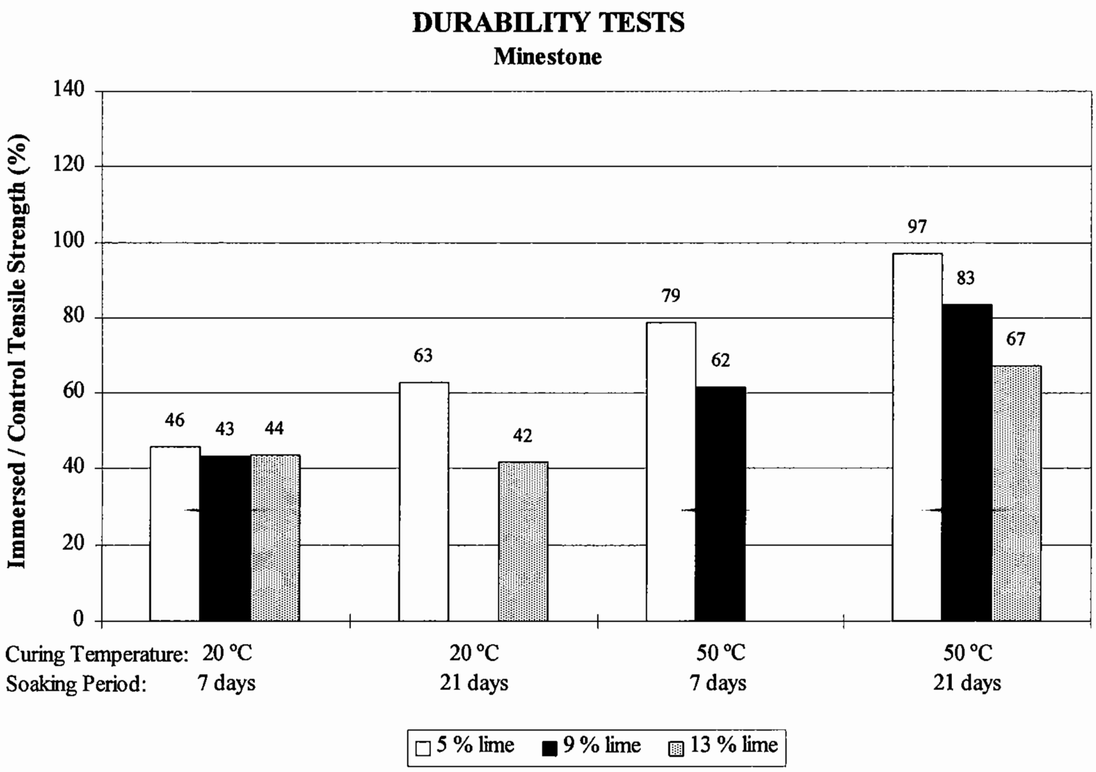

Figure 7.18 Durability results of lime-stabilised minestone (tensile strength).

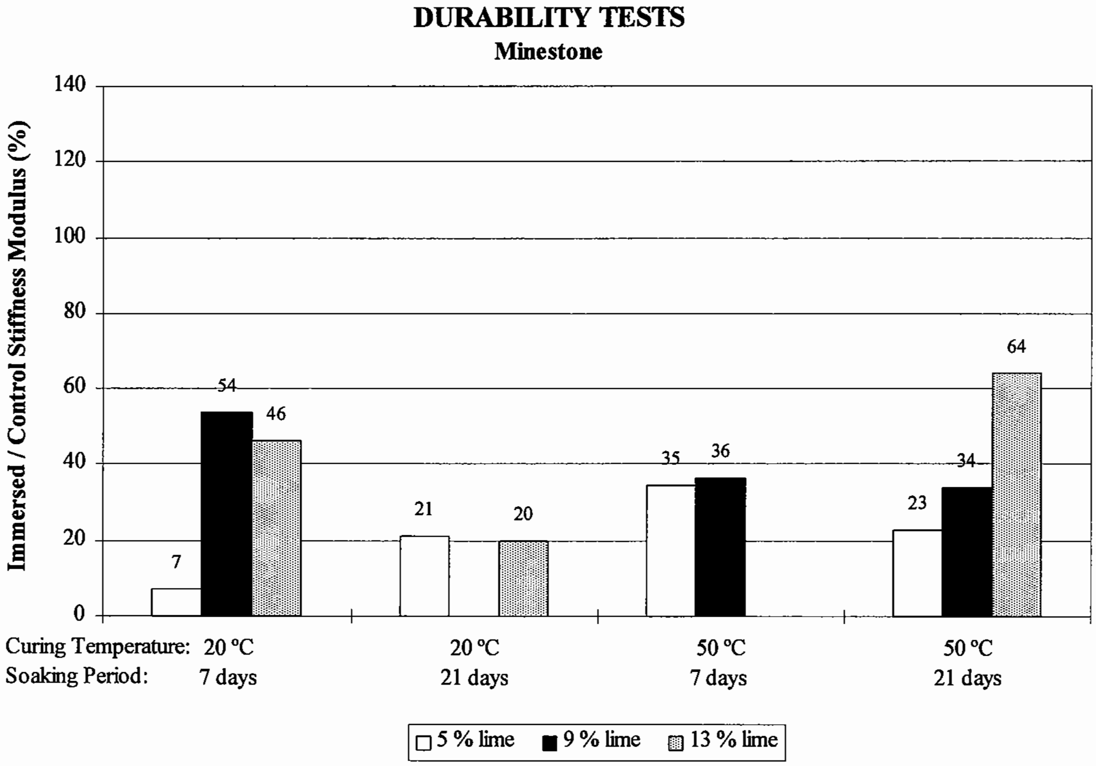

Figure 7.19 Durability results of lime-stabilised minestone (stiffness modulus).

For pulverised fuel ash the results are reasonably high indicating a good durability of the material. The exception here was fly ash with 2 % cement in which the immersed tensile strength dropped to 62 % of the control specimens. The water susceptibility seems to be higher for this level of treatment than for the other two, raising some concern on the durability of the material when stabilised with such a small percentage of binder. However, the prolonged immersion during 21 days produced a lower reduction to 78 %. Of course, this improvement is due to hydration while the specimens are submerged (which may also account for ratios higher than 100 %) but prolonged contact with water is also likely to occur when the material is used in pavements and the possibility of that contact producing extra strength is a positive characteristic to bear in mind.

The consideration of two immersion periods on durability studies seems to be advantageous to double check results and for the extra information that it provides in relation to long-term immersion. This aspect will be studied later in Chapter 8.

The results obtained with fly ash seem to confirm that tensile strength is more affected by variations in moisture content than is the stiffness modulus.

As far as minestone is concerned the results are in clear contrast with the ones obtained with fly ash. First, the deterioration of mechanical properties after immersion is high raising serious concern as to the durability of the mixtures. Second, there is an increase in the variability of strength and stiffness results which may result from the particles of the aggregate whose size was up to 28 mm and / or from the deterioration being amplified in weak sections of the specimens.

There appears to exist a tendency for strength improvement (Figure 7.18) at higher temperatures and for longer periods of immersion. Curing temperature affects the strength and stiffness of lime stabilised materials. Higher temperatures activate pozzolanic reactions accelerating the strength development process. However, by altering the curing temperature to unrealistic values when *in situ*, there is a risk of facilitating production of chemical reactions that would not occur otherwise. Further data are listed in Table 7.5 where there is no clear evidence of strength enhancement with time when the material was cured at normal temperature (20 °C).

Table 7.5 Strength development with time (minestone).

Material			**Tensile Strength**
Major Component	**Lime Content**	**Curing Period**	
	%	days	kPa
Minestone	5	28	121
	5	90	102
	5	360	154
Minestone	9	28	127
	9	90	111
	9	360	113
Minestone	13	28	110
	13	90	109
	13	360	101

For this reason it will be necessary to conduct a comprehensive research on the effects of using high curing temperatures before they can be incorporated safely in durability tests. Those effects will not be the same for all combinations of aggregate / binder and any generalisation from a particular material is not possible. On this count, at the present moment the use of 20 °C is thought to be a more realistic and conservative option.

Overall, the tests considered seem to be able to discriminate between materials according to their susceptibility to the immersion in water. With the findings reported here a second study on durability was carried out on a wide range of mixtures which will be presented in Chapter 8.

7.9. Resistance to fatigue

7.9.1. Test method

Fatigue testing is conducted by applying repeated loading to the specimen until failure occurs at tensile stress levels lower than the tensile strength of the material. This procedure is repeated with several specimens at different stress levels so that fatigue lines can be determined by plotting the indirect tensile stresses against the number of cycles to failure (on a log scale). Linear regression using the least squares method is applied to fit a straight line to the experimental results. The equation can be presented in the following forms:

$$\sigma_N = a - b \log_{10} N \qquad 7.23$$

or

$$\frac{\sigma_N}{\sigma_0} = 1 - a \log_{10} N \qquad 7.24$$

where: σ_N indirect tensile stress resulting in failure after N cycles;
σ_0 indirect tensile strength;
N number of cycles to failure;
a, b material constants.

The second method of presentation is here preferred since it allows a more direct comparison of the fatigue characteristics of materials by using the slope of the fatigue line or the value of σ_N / σ_0 after a specified number of cycles, for example 1,000,000 cycles.

Other methods of analysis exist. For example the analysis in terms of strains against the number of cycles rather than stress is commonly used for bituminous materials [BS DD ABF, 1996]. This was not adopted here for the following reasons:

- the analysis in terms of strains requires the assessment of stiffness modulus at the stress levels used for fatigue testing. This gives rise to some problems
    - the determination of stiffness using indirect tensile mode is a reliable technique in so far as the stress levels are maintained well below failure values (up to approximately 60 % of the tensile strength). But for fatigue testing the range of stress levels used is from 50 - 60 % up to 100 %;
    - extrapolation outside the range of stresses used for stiffness determination may be unreliable, notably if that extrapolation is done for higher stress levels approaching failure;
- analyses conducted in terms of strains, which were carried out in the initial stages of this project, produced lower correlation coefficients.

The stress levels should be related to the monotonic tensile strength of the materials and may be chosen down to approximately 60 % of the ultimate strength value, depending on the material tested. As an example 90, 80, 70, and 60 % may be enough to give a reasonable idea of the fatigue line and then further tests may be conducted to confirm values and improve the fatigue line definition.

The values of the vertical load, cycle number and vertical deformation are recorded at specified times during the test. The main purpose of monitoring the vertical deformation is to permit the test to be stopped when the specimen fails.

Effect of frequency of loading

The influence of the frequency of loading on fatigue testing was studied by testing at 2 different frequencies: 0.1 Hz and 2 Hz. The results are shown in Figure 7.20. It can be seen that a large increase in frequency causes an increase in the fatigue life of the material for a given stress level. Yet, the slope of the fatigue line does not change significantly.

The frequency of loading used was 2 Hz, corresponding to a risetime of 125 ms. This value is very similar to the one used for bituminous materials (120 ms), though the resting time is considerably decreased to 150 ms, reducing the length of the test.

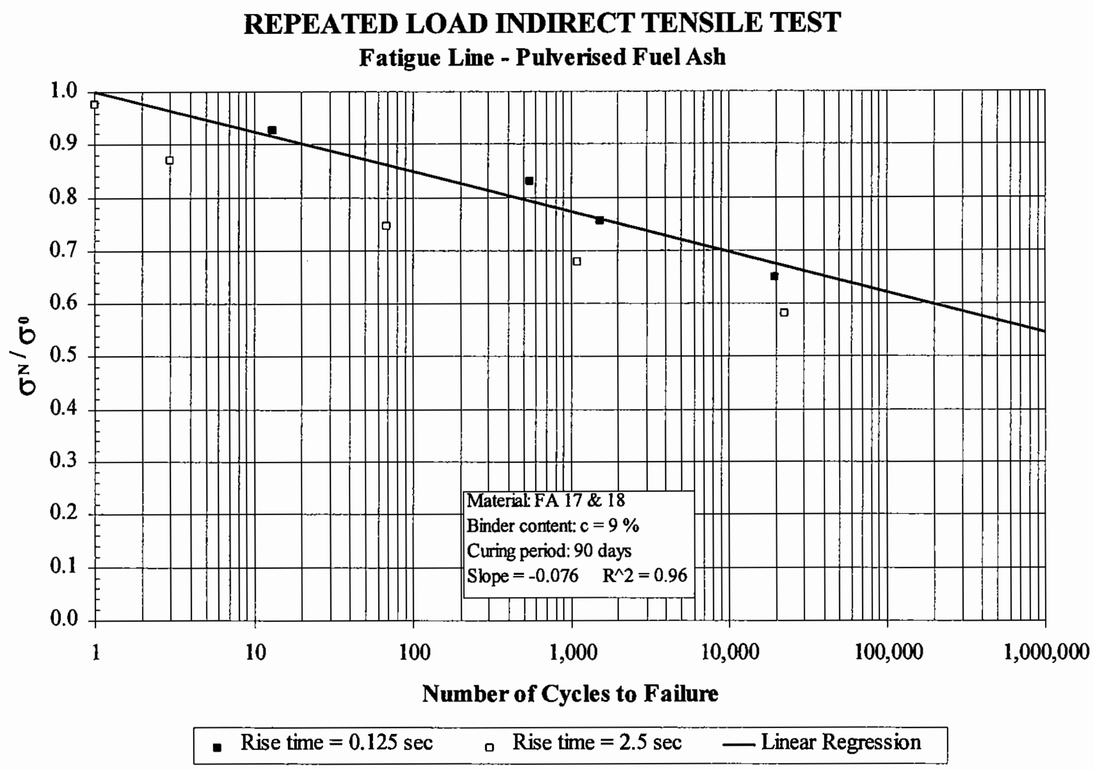

Figure 7.20 Effect of the frequency of loading on the resistance to fatigue.

7.9.2. Results

The fatigue lines obtained for each one of the three stabilisation levels and for each curing period studied are presented in Figures 7.21 to 7.29. In some cases, testing a higher number of specimens as recommended previously would improve regression results and the definition of the straight line. However, it was necessary to limit their number because of the extensive number of tests and materials studied in this research.

On the one hand, there is no clear tendency of the fatigue susceptibility to increase with length of curing. On the other hand, there is some evidence suggesting a trend for an improvement in fatigue characteristics with an increase in binder content. An exception to this would be fly ash stabilised with 9 % of cement and tested at 90 days (Figure 7.28).

The specimens of this mixture were sub-divided into two groups to be tested at different frequencies (for the frequency study reported above) and for this reason the number of specimens is low and the slope of the line may be less reliable.

Overall, the test appears to produce consistent fatigue lines and reasonable correlation coefficients. However, it is necessary to bear in mind that the inherent variability of the materials in terms of strength will affect the results in fatigue testing and in the same way as a minimum of 3 to 5 specimens is used for strength evaluation, 'ideally' the same number should be used for each stress level considered. From the experience gained in this work, it is considered that an absolute minimum of 4 different stress levels and 2 specimens per stress level in a total of 8 specimens should be considered for the assessment of fatigue characteristics.

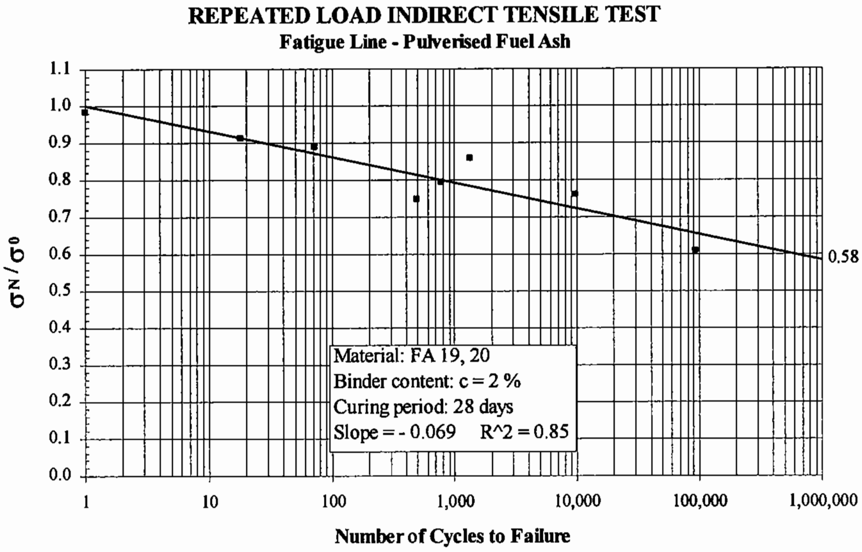

Figure 7.21 Fatigue results of cement-stabilised fly ash (c = 2 % @ 28 days).

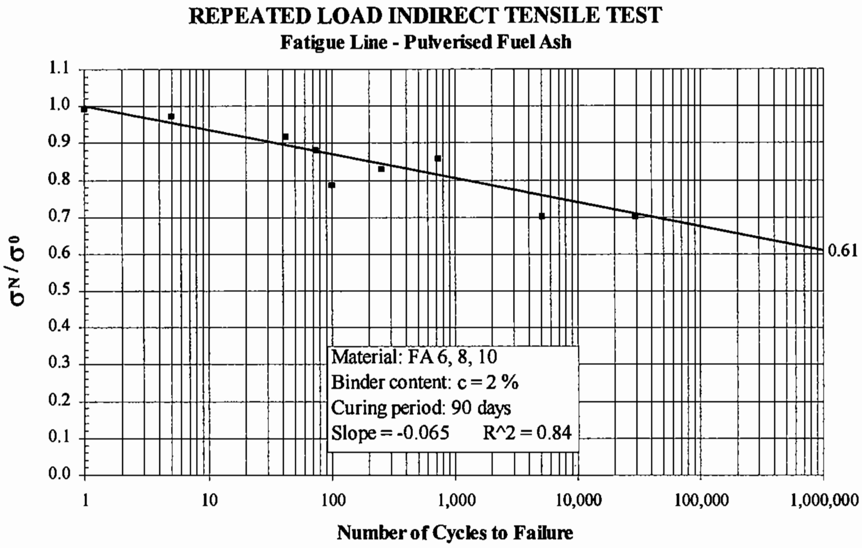

Figure 7.22 Fatigue results of cement-stabilised fly ash (c = 2 % @ 90 days).

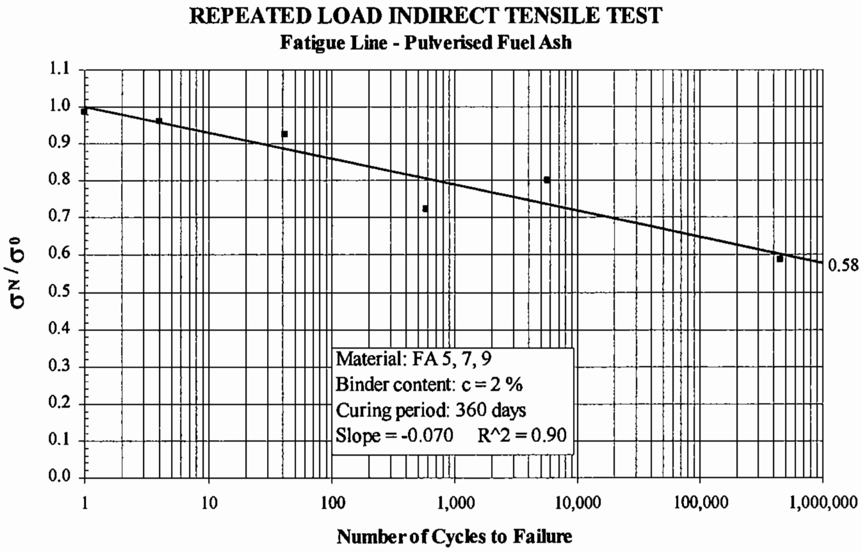

Figure 7.23 Fatigue results of cement-stabilised fly ash (c = 2 % @ 360 days).

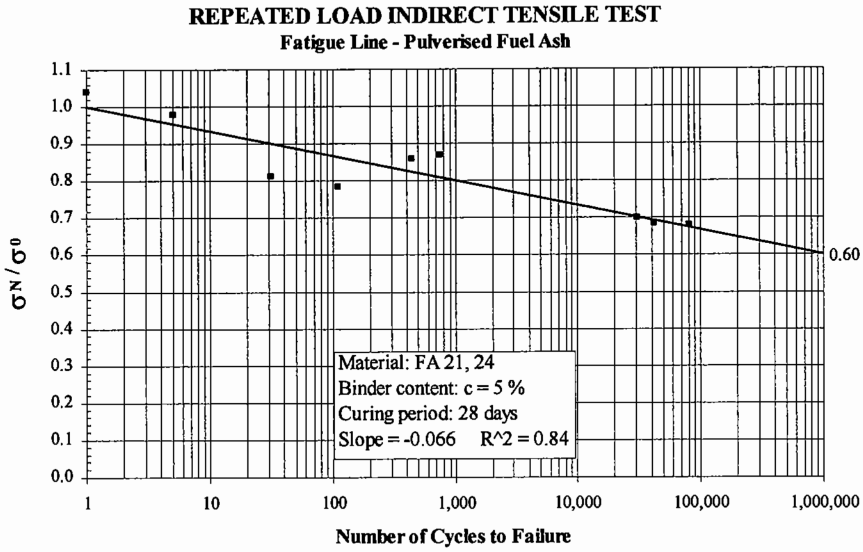

Figure 7.24 Fatigue results of cement-stabilised fly ash (c = 5 % @ 28 days).

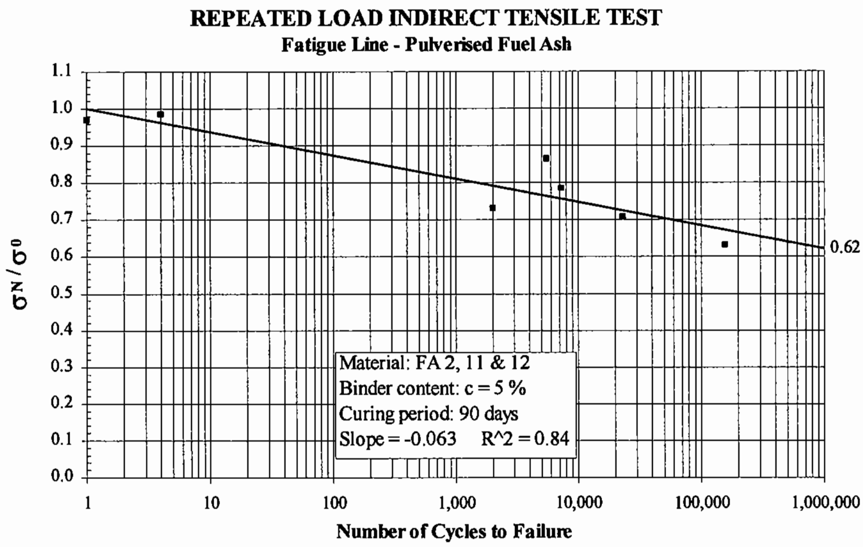

Figure 7.25 Fatigue results of cement-stabilised fly ash (c = 5 % @ 90 days).

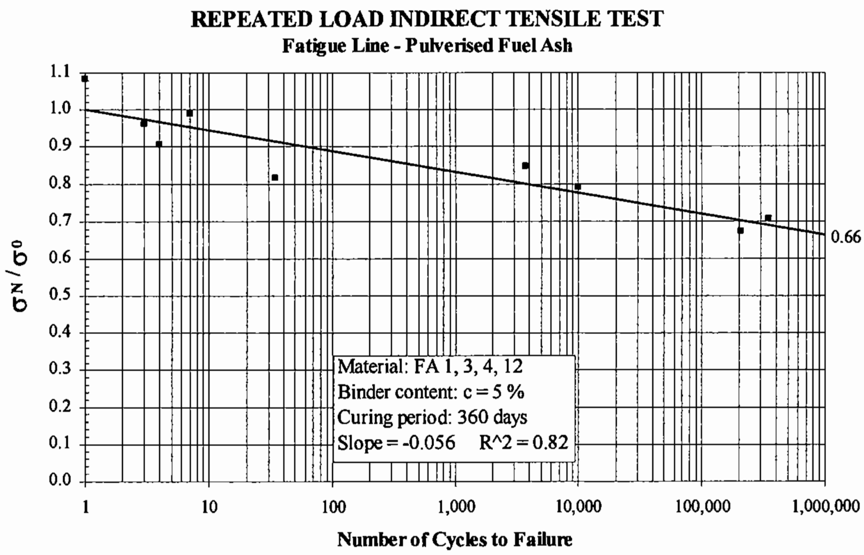

Figure 7.26 Fatigue results of cement-stabilised fly ash (c = 5 % @ 360 days).

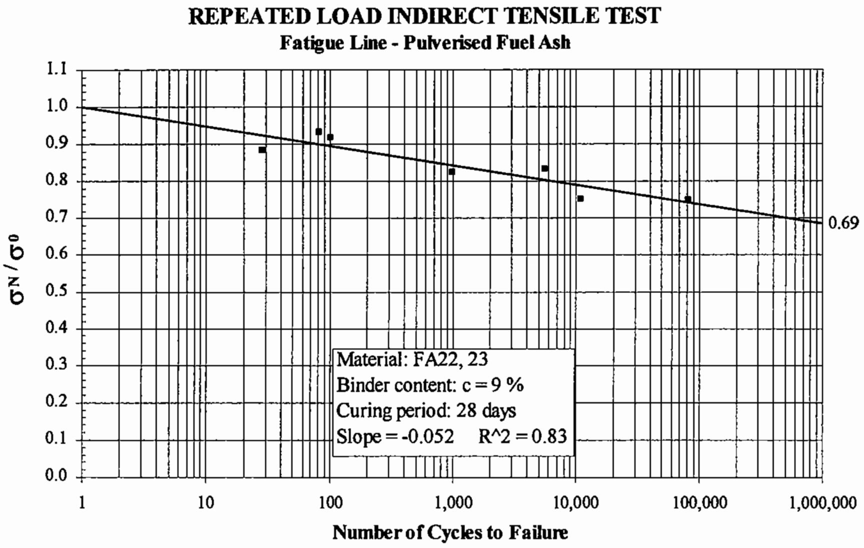

Figure 7.27 Fatigue results of cement-stabilised fly ash (c = 9 % @ 28 days).

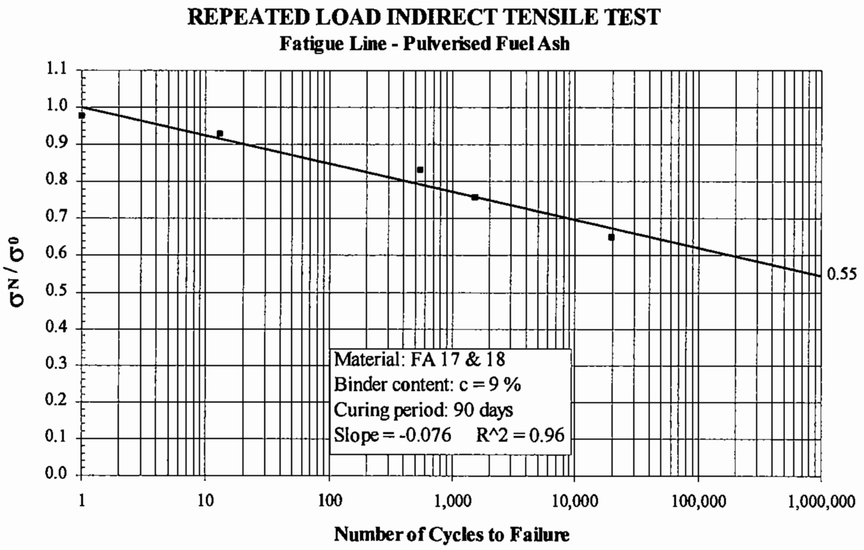

Figure 7.28 Fatigue results of cement-stabilised fly ash (c = 9 % @ 90 days).

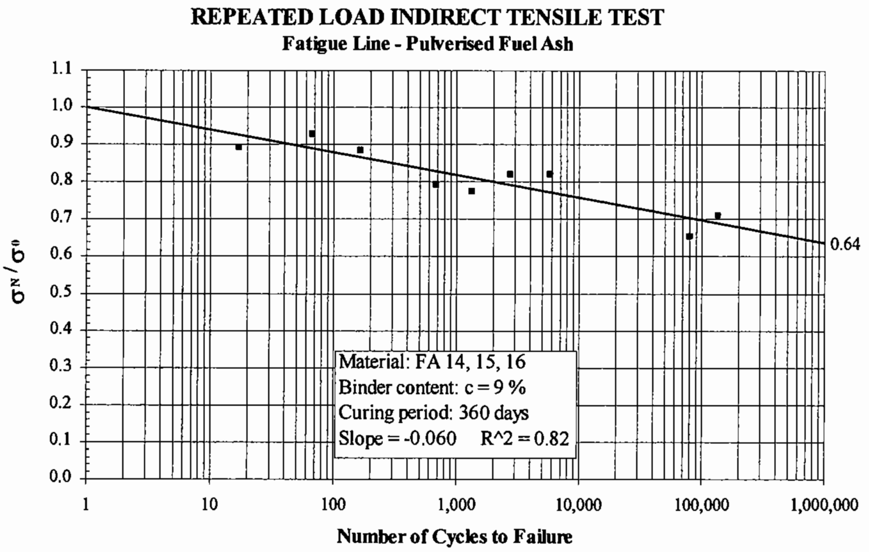

Figure 7.29 Fatigue results of cement-stabilised fly ash (c = 9 % @ 360 days).

7.10. Summary

- The issues involved in indirect tensile testing were comprehensively discussed. The advantages and disadvantages were identified together with those of the main alternative tests: direct tensile test and beam flexure tests. Overall the test has considerable potential for the study of secondary materials and binders and for evaluation of mechanical properties for use in analytical methods of pavement design.

- A method of analysis was presented which is based on two-dimensional plane-stress elastic theory. This was complemented with correction factors to take into account three-dimensional effects and specimen bulging. Those factors were developed based on three-dimensional finite element analysis of indirect tensile specimens and were found to be more important for the calculation of Poisson's ratio than for stiffness modulus.

- A number of specifications cover this form of testing but, with the exception of indirect tensile strength, they are applicable to bituminous materials and do not cover materials treated with hydraulic binders. This represents a lacuna in the present specifications' coverage.

- The research conducted encompassed the study of tensile strength, stiffness modulus, Poisson's ratio, resistance to fatigue, durability and the development of properties with time. The methodology used and proposed for the assessment of each of these properties is described in detail with special reference to the results obtained with cement-stabilised fly ash.

- Empirical relationships proposed in the bibliography for lime activated fly ash bound mixtures, to estimate 360-day properties from 90-day results, were found not to be applicable for the cement stabilised fly ash. This reinforces the importance of expeditious tests such as the repeated load indirect tensile test for material evaluation.

- For stiffness modulus and Poisson's ratio evaluation, on-sample instrumentation systems comprising miniaturised LVDTs located in the central quarter of the specimen should be used to measure the vertical and horizontal deformations directly in an area where stresses and strains remain broadly constant. However, for stiffness modulus determination, LVDTs mounted in an aluminium frame to measure the total horizontal deformation were found to be an acceptable alternative.

- Coefficients of variation (Cv) up to approximately 15 % were obtained for both tensile strength and stiffness modulus. These coefficients are in accordance with the variability obtained in other laboratory tests. The use of Cv values seems to be an adequate method for controlling the variability of properties in laboratory testing.

- The test considered for durability assessment seems to be able to discriminate materials according to their water susceptibility. On the one hand, the consideration of two differing periods of immersion appears to be advantageous providing extra information regarding long-term immersion. On the other hand, higher than normal temperatures should not be used before further research is carried out.

- The resistance to fatigue is affected by the frequency of loading. Fatigue characteristics seem to improve for higher binder contents.

- The results seem to agree reasonably well with those from repeated load triaxial tests and to contribute to the validation of the techniques developed and of the repeated load indirect tensile test as a performance-based test. Overall, the repeated load indirect tensile test seems to produce reliable and consistent measures of material performance.

CHAPTER 8

HEAVY STABILISATION OF SECONDARY MATERIALS

8.1. Introduction

In previous chapters a range of alternative materials was studied both in unbound and lightly-stabilised forms. Another possible approach to the use of secondary materials is their stabilisation using higher levels of binder (treatment level III, see Section 4.4). This will produce stronger and more durable materials that may find applications at several levels in the pavement structure, perhaps as high as the roadbase.

In this chapter, the last approach is considered and the methodologies developed and described before are applied to the study of thirteen mixtures. This work was conducted for the 'LINK II' project "*Enabling the Use of Secondary Materials and Binders in Pavement Foundations*" being carried out by the Transport Research Laboratory (TRL) with the University of Nottingham and involving the Department of Transport and the following companies generating secondary materials: British Cement Association, British Steel, Buxton Lime Industries Ltd, Camas Aggregates, National Power PLC, Powergen, Wimpey Minerals Ltd [Nunes, Dawson, Chaddock & Kennedy, 1995; Nunes, Dawson, Chaddock & Atkinson, 1996]

The principal objectives of the project are:

- to establish appropriate, rapid, practical, laboratory-based means of identifying candidate materials and mixtures for use in the sub-base and capping layers of the pavement structure;

- development of a generic specification defining acceptable performance criteria for secondary aggregates and binders, for use in various layers of the pavement foundation;
- application of this specification to a limited number of materials or material combinations, to evaluate their structural equivalence to conventional materials in full-scale trials carried out under controlled conditions of wheel loading and subgrade conditions.

The main objective of the element of the work described here was to conduct a comprehensive laboratory assessment of thirteen different mixtures in order to select six which will be further studied and optimised. Ultimately, the six selected mixtures together with a control will be used for full-scale testing in the Pavement Test Facility of the Transport Research Laboratory.

8.2. Selection of mixtures

The secondary materials considered in this work were most of the materials selected in Chapter 2, with the exception of minestone, slate waste and furnace bottom ash that were already the object of substantial work presented in previous chapters. A complete list is shown in Table 8.1 where their sources are identified.

Table 8.1 Materials studied and respective sources.

Material	**Company**	**Source**
Aggregates		
China clay sand	Camas Aggregates	Lee Moor Quarry
Air-cooled blastfurnace slag	Wimpey Minerals Ltd.	Llanwern - Cambrian Stone
Air-cooled steel slag	Wimpey Minerals Ltd.	Llanwern - Cambrian Stone
Pulverised fuel ash	Powergen	Ratcliffe-on-Soar Power Stn
FGD Gypsum	Powergen	Ratcliffe-on-Soar Power Stn
Binders		
Cement (OPC)	British Cement Association	Hope Works - Blue Circle Ind.
Cement (C_3A < 5 %)	British Cement Association	Weardale - Blue Circle Ind.
Lime (quicklime)	Buxton Lime Industries Ltd.	Tunstead Quarry
Pulverised fuel ash	Powergen	Ratcliffe-on-Soar Power Stn
Granulated blastfurnace slag	Wimpey Minerals Ltd.	Llanwern - Cambrian Stone
Ground granul. blastfurnace slag	Wimpey Minerals Ltd.	Llanwern - Cambrian Stone
FGD Gypsum	Powergen	Ratcliffe-on-Soar Power Stn
Cement kiln dust	British Cement Association	Westbury - Blue Circle Ind.

The cement with low percentage of tricalcium aluminate (C_3A < 5 %) was used for the stabilisation of FGD gypsum in accordance with the technical guidelines presented in Chapter 2.

The potential number of mixtures to consider in this study is vast. Apart from the number of secondary aggregates that were selected for study, one has to bear in mind the large number of possible mixtures resulting from combining primary and secondary aggregates which in turn may be stabilised using primary or secondary binders, or combinations of these. In addition, the proportions of the mixture components may vary widely, further increasing the number of potential mixtures.

The mixtures considered and the proportioning of their components were selected by a committee of members of the 'LINK II project'. Initially, a short-list of 18 mixtures incorporating secondary materials was developed together with an additional 3 reference mixtures based on:

- known successful performance in the UK in the past;
- known successful performance outside the UK;
- suggestions from the industry involved;
- best estimates of other possible successful mixtures.

The initial list of materials is reproduced in Table 8.2. Further reduction of the number of mixtures had to be conducted to a proposed list of 13 mixtures. The narrowing was based on the following principles:

- final material should have a reasonable chance of performing successfully;
- final material should have some chance of being acceptable to a, possibly, sceptical clientele;
- only one of two broadly similar mixtures should be subjected to detailed testing.

The final materials selected were assigned to three 'families' as indicated below:

- *Ash or gypsum-based mixtures* M1, M2, M4, M6 and M7;
- *China clay sand-based mixtures* M8, M9, M10 and M12;
- *Coarse aggregate-based mixtures* M15, M17, M18 and M19.

The thirteen candidate materials listed above represent a reasonable spread of different types and the selection of the final six materials may be conducted by choosing two from each family without undue overlap as far as their components are concerned.

Table 8.2 Initial list of mixtures.

Code	Major Component	Principal Binder		Activator		Notes
M1	Pulverised Fuel Ash	Cement				reference
M2	Pulverised Fuel Ash	Cement kiln dust	Cement			
M3	Pulverised Fuel Ash	Lime		Sodium Carbonate		
M4	Pulverised Fuel Ash	Gypsum	Lime			proven
M5	Pulverised Fuel Ash	Cement kiln dust	Gypsum			
M6	Pulverised Fuel Ash	Cement kiln dust	Granulated slag			
M7	Gypsum	Pulverised fuel ash	Cement			
M8	China Clay Sand	Cement				reference
M9	China Clay Sand	Cement kiln dust	Cement			
M10	China Clay Sand	Pulverised fuel ash		Lime		≈ proven
M11	China Clay Sand	Pulverised fuel ash	Granulated slag	Lime		
M12	China Clay Sand	Ground granul. slag		Lime	Gypsum	
M13	China Clay Sand	Cement kiln dust	Ground granul. slag			
M14	China Clay Sand	Cement kiln dust	Granulated slag			
M15	Conventional aggregate	Cement				reference
M16	Conventional aggregate	Cement kiln dust		Cement		
M17	Conventional aggregate	Pulverised fuel ash		Lime	Gypsum	proven
M18	Blastfurnace slag	Granulated slag		Lime		
M19	Blastfurnace slag	Granulated slag		Steel slag		
M20	Blastfurnace slag	Cement kiln dust	Granulated slag			
M21	Conventional aggregate	Cement kiln dust	Granulated slag			

Selection of mixture proportions

The estimate of the percentages of each component of the mixtures was based on:

- the literature recommendations presented in Chapter 2;
- past experience on the use of these materials of the members of the 'LINK II project' Committee;
- the indispensable input from the companies involved.

The resulting materials will need to demonstrate an acceptable structural performance both in the short and in the long term. In the third family (the coarse aggregate-based mixtures) both the strength development process due to stabilisation and the aggregate matrix will contribute to the structural performance of the materials. The choice of an adequate grading curve of the mixture will contribute to its mechanical stability, producing better early age trafficability with the load being transmitted through the points of contact between particles and the fines filling the voids between coarse particles to prevent movement. On the other hand, the first family (the ash or gypsum-based mixtures) will need to compensate for the lack of an aggregate matrix with faster development of strength and stiffness to enable trafficking after short periods of time. This family relies very largely on the binder to produce a material with successful performance. The situation for the second family is intermediate with the existence of an aggregate structure as a major feature of the mixture, but lacking many of the coarser particles.

As a result of these considerations Table 8.3 has been developed showing the suggested mixtures and respective proportions. It is not the objective of the work described in this chapter to conduct an optimisation of the mixtures. That is the purpose of a subsequent phase of the project which immediately precedes the full-scale pavement trials and which is in progress at the time of writing. As a result, it is expected the mixtures presented may be further improved through small variations in the proportions of binder and also in the moisture content used for compaction.

Table 8.3 Selected mixtures and proportions.

Material Code	Major Component %	Binder / Activator % (by weight)					
Pulverised Fuel Ash							
M1	93	cement	7				
M2	90	cement kiln dust	7	cement	3		
M4	91	FGD gypsum	5	lime	4		
M6	70	cement kiln dust	20	granulated slag	10		
FGD Gypsum							
M7	82	pulverised fuel ash	10	cement	8		
China Clay Sand							
M8	93	cement	7				
M9	90	cement kiln dust	7	cement	3		
M10	80	pulverised fuel ash	15.5	lime	4	sodium carbonate	0.5
M12	92.5	ground granul. slag	6	lime	1	FGD gypsum	0.5
Granite Type 1							
M15	96	cement	4				
M17	92	pulverised fuel ash	6	lime	1.5	FGD gypsum	0.5
Blastfurnace Slag							
M18	84	granulated slag	15	lime	1		
M19	70	granulated slag	20	steel slag	10		

8.3. Testing programme

The final version of the testing programme is presented in Table 8.4. The programme also included an initial assessment in terms of compactibility tests to determine the optimum values of moisture content and dry density for the compaction of the mixtures. The relevant properties that need to be assessed when using the materials in pavement construction are:

in the short term

- early age trafficability. This is analysed through the values of compressive strength of cubes at 7 days. A guide requirement of a minimum of 2 MPa at 7 days was considered based on the strength requirements presented in Section 4.5;

in the long term

- tensile strength;
- stiffness modulus;
- resistance to fatigue; and
- durability.

The long term assessment was based on repeated load indirect tensile tests performed at different curing periods. The evaluation of the durability is based on the comparison of the performance of specimens immersed for a specified period ('wet cure' in Table 8.4) with that of control specimens.

Cube crushing tests were also performed at 7, 28, 90 and 360 days. Apart from giving an indication of early age trafficability, the information obtained enables an estimation of fatigue resistance at ages when it was not directly determined. This is also the main test currently used for the characterisation of stabilised materials [DoT, 1993] and considerable experience exists on the results obtained. By including this type of test in this project a comparison can be maintained with standard procedures.

The results presented here concern tests performed on specimens cured for periods up to

90 days. The 360-days testing indicated in Table 8.4 is not reported because the testing dates coincided with the submission of this thesis, preventing the processing and analysis of results in due time. Nevertheless, the data is unlikely to affect the choice of the six mixtures to be considered for further investigation which is the main purpose of the study presented in this chapter.

Table 8.4 Final testing programme.

Mixture	CRUSHING TESTS					REPEATED LOAD INDIRECT TENSILE TESTS							
	Cubes at days				TOTAL	Strength at days			Stiffness at days			Fatigue	TOTAL
						normal	wet cure		normal	wet cure			
	7	28	90	360		28	28	90	28,90,360	28	90	@ 90 days	
1 FA/C	3	3	3	3	12	3	2	1	3	2	1	6	15
2 FA/CdC	3	3	3	3	12	3	2	1	3	2	1	6	15
4 FA/GyL	3	3	3	3	12	3	2	1	3	2	1	6	15
6 FA/CdGs	3	3	3	3	12	3	2	1	3	2	1	6	15
7 GY/FaC	3	3	3	3	12	3	2	1	3	2	1	6	15
8 CC/C	3	3	3	3	12	3	2	1	3	2	1	6	15
9 CC/CdC	3	3	3	3	12	3	2	1	3	2	1	6	15
10 CC/FaL+	3	3	3	3	12	3	2	1	3	2	1	6	15
12 CC/GgsLGy	3	3	3	3	12	3	2	1	3	2	1	6	15
15 T1/C	3	3	3	3	12	3	2	1	3	2	1	6	15
17 T1/FaLGy	3	3	3	3	12	3	2	1	3	2	1	6	15
18 BS/GsL	3	3	3	3	12	3	2	1	3	2	1	6	15
19 BS/GsSs	3	3	3	3	12	3	2	1	3	2	1	6	15
					156								195

KEY: ***Aggregates*** **FA**=pulverised fuel ash **GY**=gypsum **CC**=china clay sand **T1**=granite Type 1 **BS**=air-cooled blastfurnace slag
Binders C=cement L=lime **Fa**=pulverised fuel ash **Gs**=granulated blastfurnace slag **Ggs**=ground granulated blastfurnace slag **Cd**=cement kiln dust
Gy=FGD gypsum Ss=steel slag + =sodium carbonate (Na2CO3)

8.4. Compactibility tests

To permit a fair comparison between mixtures and to evaluate the maximum potential of stabilised secondary materials it was necessary to conduct the compaction of the specimens at optimum conditions. Accordingly, to determine the optimum values for moisture content and dry density, compactibility tests were performed on all thirteen mixtures. Some changes were made in relation to what is specified by the British Standards [BS 5835, 1980; BS 1924, 1990]. First of all, the number of portions taken for each level of initial moisture content was reduced to one, or two portions when the first result appeared unreliable. This was an important practical measure, since the number of mixtures to be studied made with full compliance with the referred standard an impractical proposition.

In addition, most of the current standards for pavement materials, including the ones referred above, were developed for conventional aggregates and direct application to secondary materials may cause errors as far as the evaluation of the properties of the material is concerned. In this particular case the standards specify that representative portions of aggregate for testing should be prepared from the bulk sample having an oven-dry mass in the range of 2.4 to 2.6 kg [BS 5835, 1980; BS 1924, 1990]. This amount of material will result in a compacted specimen of around 70 mm height for a conventional granite Type 1 for sub-bases. Secondary materials have a wide range of particle densities, with materials such as pulverised fuel ash and gypsum being at one end of the scale and steel slags at the other end. The use of a common quantity of material irrespective of their relative particle density will result in the energy of compaction being applied to completely different volumes of materials. This may result in significant under / overestimation of the optimum values for compaction for light / heavy materials respectively.

Thus it was decided that the compaction should be carried out on amounts of materials resulting in specimens of roughly the same volume after compaction (70 mm height in the mould gives a reasonable value). It may be argued that to estimate the amount of material to use we will need the optimum values for compaction which is the result of the test. However, it is easy to do that estimation based on past results / experience or by compacting one or two specimens and then adjusting the amount of material necessary for the test.

The results obtained using this procedure are presented in Table 8.5. The optimum values seem to be well defined. Figure 8.1 shows an example in which the compaction curve reaches a peak readily defining the optimum moisture content (OMC) and maximum dry density (MDD) values. Another type of compaction curve found was the one presented in Figure 8.2 in which the dry density of the specimen continues increasing for higher levels of water content. In this case, the effective optimum values considered are the ones corresponding to the point in which the difference between the initial moisture content and the residual one equals 2 %. Above this level further water tends to drain from the specimen, not being used in the compaction process. This generally happens with the most permeable materials such as china clay sand and blastfurnace slag.

The compaction of these mixtures did not present any special difficulty and the same is expected to happen when producing the materials for the Pavement Test Facility trials. However, special precaution should be taken when mixing materials including very small percentages of binder. It is known that the production of homogeneous mixtures is difficult when some components are in percentages lower than 2 or even 4 %. In this study, mixes M10, M12 and M17 have one component in percentages as low as 0.5 % which may represent a problem for the production of uniform mixes. To overcome this difficulty it is recommended that all binders should be pre-mixed together before being mixed with the major component. Following this procedure, what is certainly a very small percentage in total terms, significantly increases when considered only in relation to the total binder volume.

Table 8.5 Results of the compactibility tests.

Material Code		Major Component	Binder									OMC	MDD
			C	L	Fa	Gs	Ggs	Cd	Gy	Ss	+		
		%	%									%	Mg/m^3
Pulverised Fuel Ash													
M1	FA/C	93	7									19.1	1.43
M2	FA/CdC	90	3					7				19.4	1.51
M4	FA/GyL	91		4					5			21.2	1.46
M6	FA/CdGs	70				10		20				17.9	1.58
FGD Gypsum													
M7	GY/FaC	82	8		10							14.1	1.36
China Clay Sand													
M8	CC/C	93	7									9.8	2.01
M9	CC/CdC	90	3					7				9.3	2.03
M10	CC/FaL+	80		4	15.5						0.5	9.2	1.95
M12	CC/GgsLGy	92.5		1			6		0.5			9.2	1.96
Granite Type 1													
M15	T1/C	96	4									6.5	2.14
M17	T1/FaLGy	92		1.5	6				0.5			6.5	2.29
Blastfurnace Slag													
M18	BS/GsL	84		1		15						7.0	2.08
M19	BS/GsSs	70				20				10		8.0	2.18

KEY:
Aggregates
FA=pulverised fuel ash **GY**=gypsum **CC**=china clay sand **T1**=granite Type 1 **BS**=air-cooled blastfurnace slag
Binders
C=cement **L**=lime **Fa**=pulverised fuel ash **Gs**=granulated blastfurnace slag **Ggs**=ground granulated blastfurnace slag
Cd=cement kiln dust **Gy**=FGD gypsum **Ss**=steel slag **+** =sodium carbonate (Na_2CO_3)

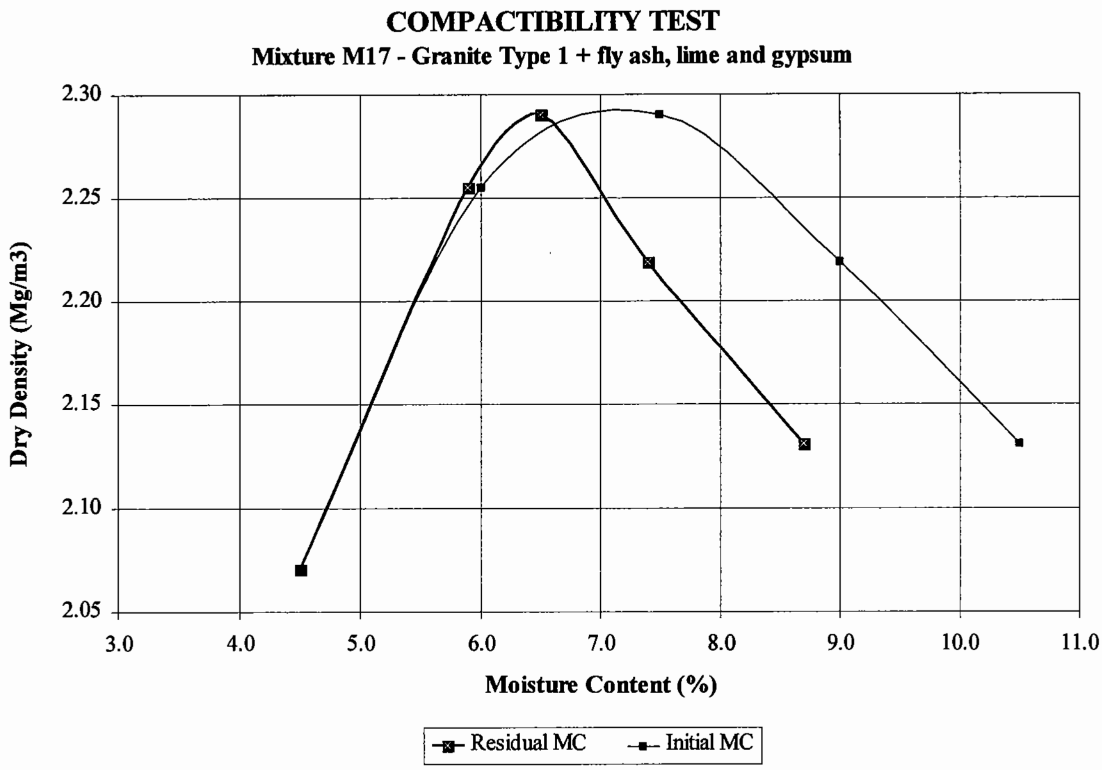

Figure 8.1 Compaction curve of mixture 17.

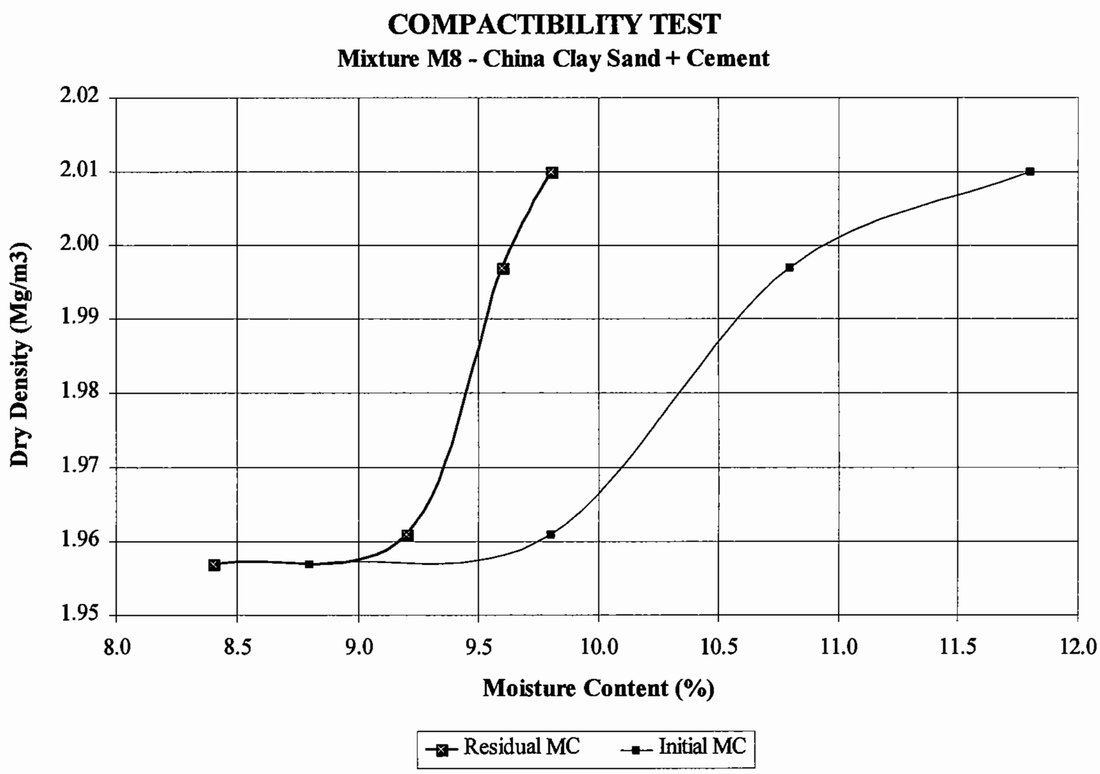

Figure 8.2 Compaction curve of mixture 8.

8.5. Compressive strength of cubes

8.5.1. Methodology

The compressive strength was determined on 100 mm-cubic specimens at different curing periods following the procedure specified in the British Standard [BS 1924, 1990]. Using a vibrating hammer, specimens were compacted at the optimum values of moisture content and dry density previously determined. According to the referred standard, the stabilised material should be compacted and left in the moulds at a constant temperature of 20 °C until the following day, when specimens may be removed from the moulds for further curing. For most of the mixtures this procedure was not possible since the development of strength occurs at a slower rate in comparison with conventional stabilised materials. Hence, they had to be left in the moulds for longer periods, typically two to three days before removal. Unfortunately, this results in corrosion of the moulds and the creation of links between mould and specimen which makes it difficult for the extraction of a complete cube to be achieved. This was a problem observed in particular for the fly ash and gypsum family and for some with china clay sand as a major component. For the other mixes with coarser aggregates the moulds' removal was more straightforward and there were no cube losses during that procedure.

In some cases the cubes were extracted from the moulds using a standard extraction machine to minimise cube losses and preserve the integrity of the specimen. For future compactions of mixes involving secondary binders the following recommendations are thought to be beneficial:

- use of a liner on the cube surfaces during compaction. This will inhibit material from sticking to the mould (although the liner material will need careful selection to avoid interference with the compaction process);
- extension of the period before the removal of the mould to two or three days.

After compaction, specimens are stored for curing observing a procedure similar to the one described earlier in Chapter 7 which includes the following steps:

- immediately after compaction, the specimens (inside their moulds) should be carefully sealed to avoid the loss of moisture. This is essential for strength development and may be achieved by using plastic membranes to double-wrap the specimens tightly;
- sealed specimens should be stored in a temperature controlled room at constant temperature of 20 °C;
- after a period of 2 to 3 days the specimens should have already developed an initial strength enabling the removal of the moulds, thus releasing them for the compaction of further specimens. The specimen should again be adequately sealed and stored at constant temperature (20 °C) until the end of the curing period.

Other methods of storage could also be used. As an example, specimens may be left to cure under either sealed conditions or in a 100 per cent humidity room. After this first phase they could be stored in laboratory conditions, for 2 days before testing, allowing them to air-dry. Alternatively, they may be submersed in water for 2 days before testing. Some disadvantages have been found by Saylak, Taha and Little [1988] in relation to these procedures. The first method leads to higher strengths by allowing the specimen to reduce its moisture significantly, and it does not characterise moisture susceptibility. On the other hand, the humidity room and submersion procedures may be detrimental for specimen integrity or for the strength development process and, when new mixtures with secondary aggregates and binders are studied, it is desirable to make use of methodologies which will not by themselves compromise the acceptability of the materials.

8.5.2. Results

The results obtained in the compressive strength tests are presented in Figures 8.3 to 8.5 as a function of time for each family of material. In Figure 8.6 the materials are ranked according to their compressive strength. Since some of the materials develop strength at a rate slower than conventional materials, their full potential will not be realised unless they are left to cure for longer periods. For comparison purposes the materials were ranked according to their 90 days compressive strength.

8.5.3. Classification and discussion of results

In order to analyse the results and fully understand the strength or weakness of materials they may be classified into the four classes of cement bound materials considered in the Specification for Highway Works [DoT, 1993]. For a complete classification following that specification it is not enough to consider the compressive strength at 7 days, other properties being specified such as grading, type of material, effect of immersion in water, absence of cracking or swelling after immersion and soaked ten per cent fines value (TFV). However, those extra requirements will be disregarded for the purpose of making a first classification of these materials, which is shown in Table 8.6.

From the results obtained it is possible to draw the following conclusions:

- there is no clear relationship between compressive strength after 7, 28 and 90 days curing for the materials studied. Even considering materials within the same family, with the same major component, that relationship cannot be deduced from the results;
- low early strength does not give a clear indication of strength with time. It may be either due to weakness of the material or slow cementation and strength development process, in which case the strength at higher ages will improve significantly (see in particular mixtures M4 and M17).

A summary of the compressive strength results is presented below [1]:

Mixtures

- best performance: M4-FA/GyL, M17-T1/FaLGy, M10-CC/FaL+, M1-FA/C
- worst performance: M18-BS/GsL, M7-GY/FaC, M12-CC/GgsLGy

Aggregates (major component)

- best performance: FA, T1
- worst performance: BS, GY

Binders

- best performance: FaLGy, FaL+, FaC
- worst performance: GsL, GgsLGy

Early-age strength development

- Fast rate: M9-CC/CdC, M1-FA/C, M15-T1/C, M2-FA/CdC, M6-FA/CdGs
- Slow rate: M10-CC/FaL+, M4-FA/GyL, M17-T1/FaLGy, M7-GY/FaC

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1 **KEY:**
Aggregates
FA=pulverised fuel ash **GY**=gypsum **CC**=china clay sand **T1**=granite Type 1 **BS**=air-cooled blastfurnace slag
Binders
C=cement **L**=lime **Fa**=pulverised fuel ash **Gs**=granulated blastfurnace slag **Ggs**=ground granulated blastfurnace slag
Cd=cement kiln dust **Gy**=FGD gypsum **Ss**=steel slag **+** =sodium carbonate (Na2CO3)

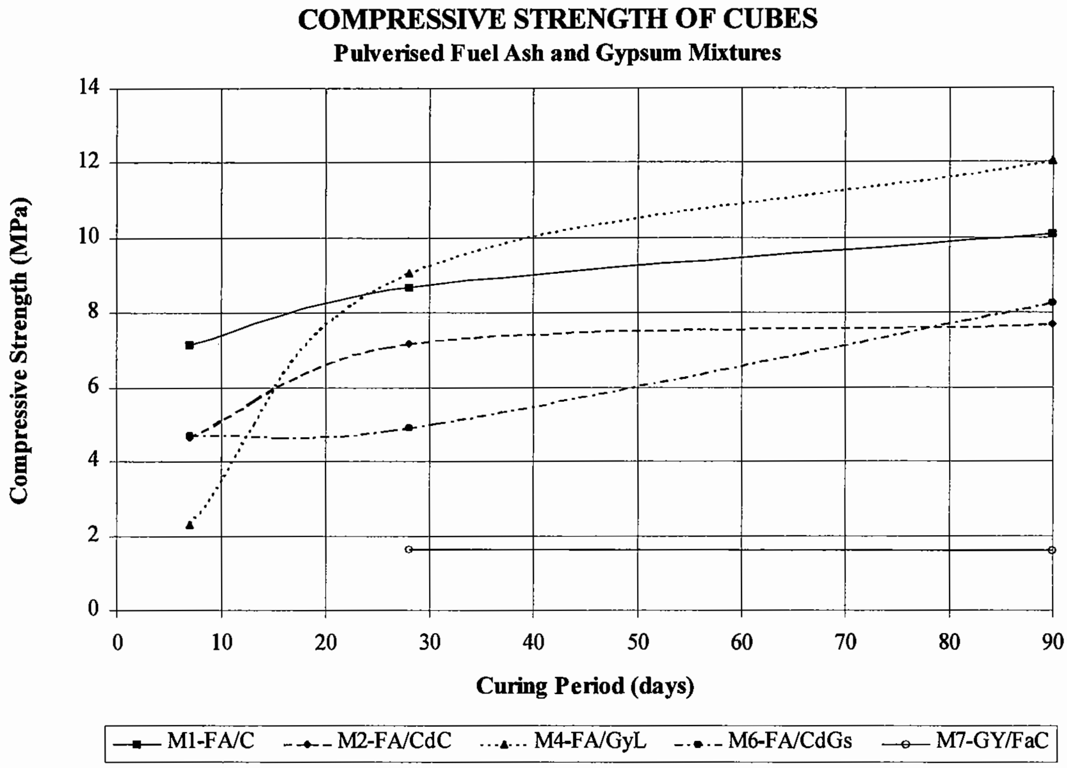


Figure 8.3 Compressive strength of the mixtures with fly ash or gypsum as a major component.

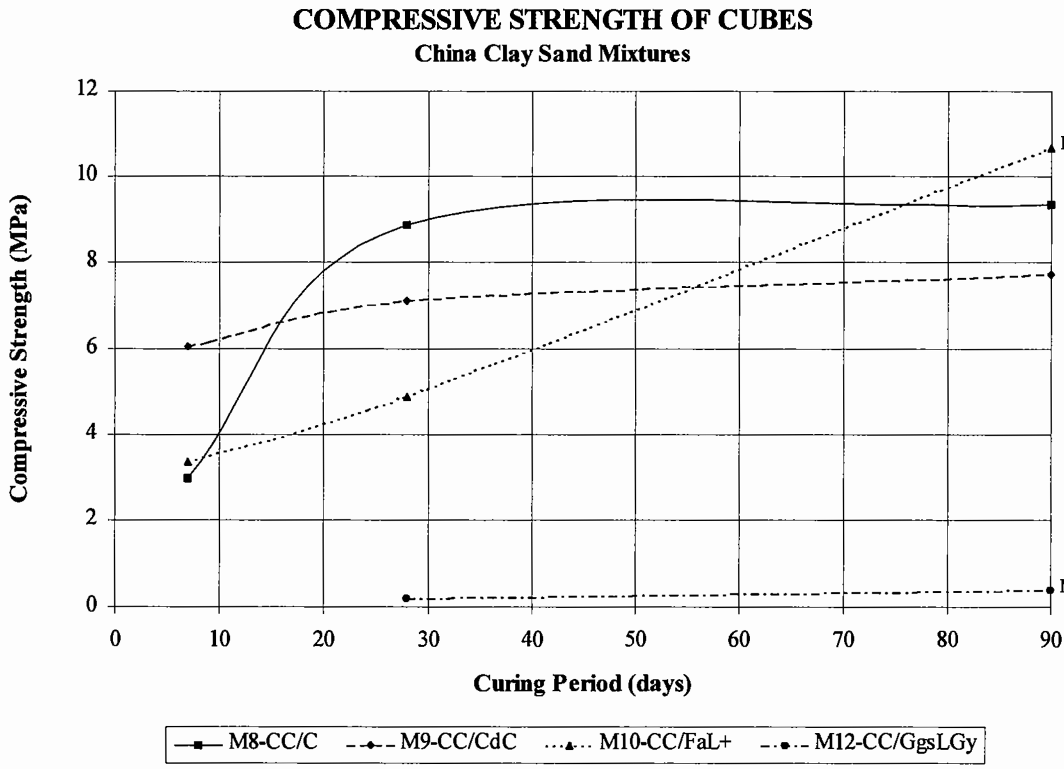


Figure 8.4 Compressive strength of the mixtures with china clay sand as a major component.

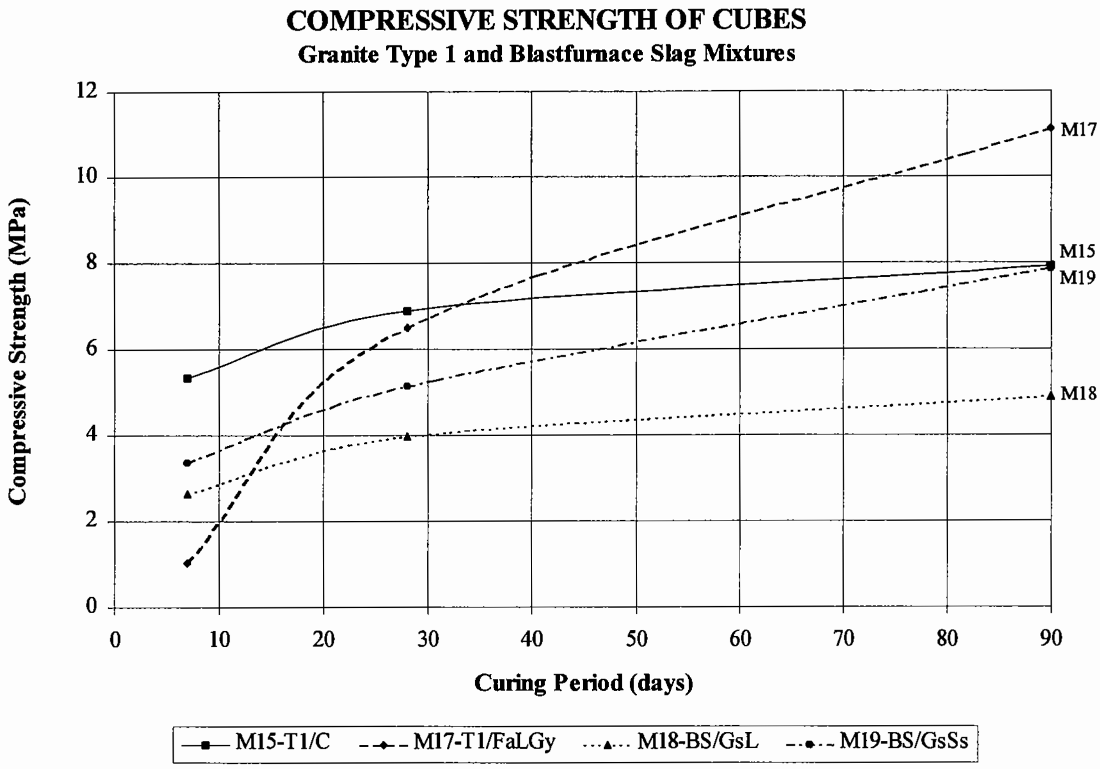


Figure 8.5 Compressive strength of the mixtures with Type 1 aggregate or blastfurnace slag as a major component.

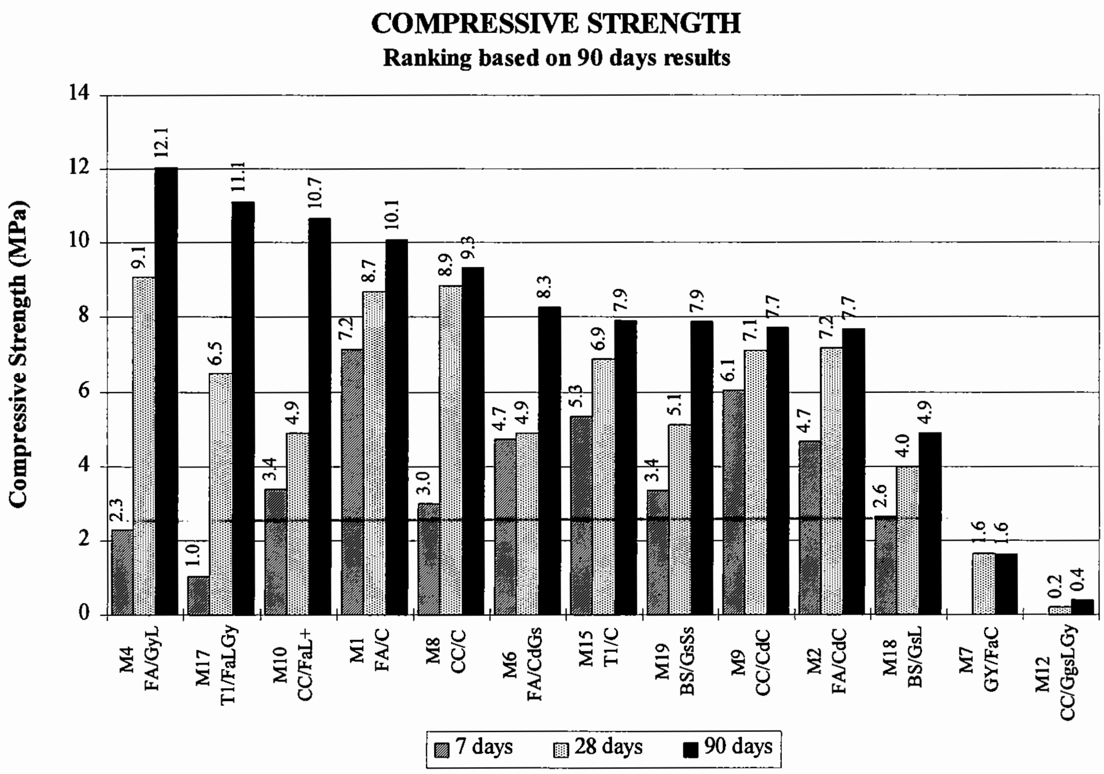


Figure 8.6 Ranking of materials based on the compressive strength of the mixtures at 90 days.

Table 8.6 Simplified classification of the materials based on compressive strength at 7 days.

Material Code		Major Component	Binder									Simplified Classification
			C	L	Fa	Gs	Ggs	Cd	Gy	Ss	+	
		%	%									CBMs
Pulverised Fuel Ash												
M1	FA/C	93	7									CBM 2
M2	FA/CdC	90	3					7				CBM 1
M4	FA/GyL	91		4					5			None
M6	FA/CdGs	70				10		20				CBM 1
FGD Gypsum												
M7	GY/FaC	82	8		10							None
China Clay Sand												
M8	CC/C	93	7									None
M9	CC/CdC	90	3					7				CBM 1
M10	CC/FaL+	80		4	15.5						0.5	None
M12	CC/GgsLGy	92.5		1			6		0.5			None
Granite Type 1												
M15	T1/C	96	4									CBM 1
M17	T1/FaLGy	92		1.5	6				0.5			None
Blastfurnace Slag												
M18	BS/GsL	84		1		15						None
M19	BS/GsSs	70				20				10		None

KEY:
Aggregates
FA=pulverised fuel ash **GY**=gypsum **CC**=china clay sand **T1**=granite Type 1 **BS**=air-cooled blastfurnace slag
Binders
C=cement **L**=lime **Fa**=pulverised fuel ash **Gs**=granulated blastfurnace slag **Ggs**=ground granulated blastfurnace slag
Cd=cement kiln dust **Gy**=FGD gypsum **Ss**=steel slag **+** =sodium carbonate (Na2CO3)

8.6. Repeated load indirect tensile tests

The testing techniques used for indirect tensile testing were essentially as presented in Chapter 7. In the cases where small variations were considered these are identified in the next sections. All data from this type of test are analysed and discussed together in Section 8.6.3 after presentation of the results in Section 8.6.2.

8.6.1. Development of compaction technique and equipment

The use of the same compaction technique for both triaxial and indirect tensile tests (Chapters 6 & 7) had the advantage of enabling a more direct comparison of results. That procedure worked well for triaxial testing because the specimens were compacted in their final form. On the other hand, to obtain indirect tensile specimens with adequate dimensions it was necessary to dry-cut the triaxial-size specimens obtained using the same technique. This raised some health concerns related with noise and the production of dust during cutting (the latter was more evident for mixtures having fine materials as a major component, for example pulverised fuel ash). In addition, the need to manufacture a large number of specimens in a relatively short period of time raised the need to develop a more expeditious compaction procedure that would directly produce specimens with the testing dimensions.

For this purpose, it was decided to use the standard rig for the compactibility test [BS 5835, 1980]. This has the advantage of permitting the compaction to be carried out in exactly the same conditions as during the determination of the optimum values of moisture content and dry density. In particular, the same compaction energy can be applied by controlling the time of compaction, currently of 3 minutes for the compactibility test. Although the rig frame and vibrating hammer could be used for this purpose, the compaction mould was not adequate. A new mould was specially designed to fulfil the requirements mentioned above. In Figures 8.7 and 8.8 details of both the compaction rig and mould are shown.

The three-piece metallic mould enables the positioning of a thin plastic mould and extension within its boundaries (Figure 8.9). A schematic representation is also presented in Figure 8.10. The inner mould is made of plastic tube enabling an easy and economic manufacture of a large number of moulds with the dimensions of the final specimen. This is particularly important since some of the materials cement slowly and they need to be kept in the mould for longer periods than would normally be necessary.

The materials are compacted in one layer, using the standard vibrating hammer [BS 5835, 1980] for a period of three minutes. The size of the completed specimen is 150 mm diameter by 70 mm thickness which compares favourably with the dimensions of the compacted material in the compactibility test. In this manner, using the pre-determined amount of material necessary and the same energy of compaction as during the compaction test, this technique should generate indirect tensile specimens compacted at the optimum values.

After compaction the specimens are stored for further curing until they are tested following the curing periods specified in the programme. The procedure used for specimen storage during curing for the indirect tensile specimens is similar to the one used for cubes which was discussed in the previous section. However, in this case the specimens are left in their moulds for longer periods. This is a good practical way to keep the specimen, preventing moisture losses and the possibility of accidental damage, and it is economically viable since the inner plastic moulds are relatively inexpensive and easy to manufacture. In addition, it enables the specimen to further develop strength which facilitates extraction.

The removal of the specimens from the moulds is done using an extraction machine operated at constant speed. Contrary to what happened with the cubes, this operation is highly successful due to the relatively low adherence of specimen / mould surfaces (and certainly no mould corrosion) and the absence of corners where the material tends to adhere.

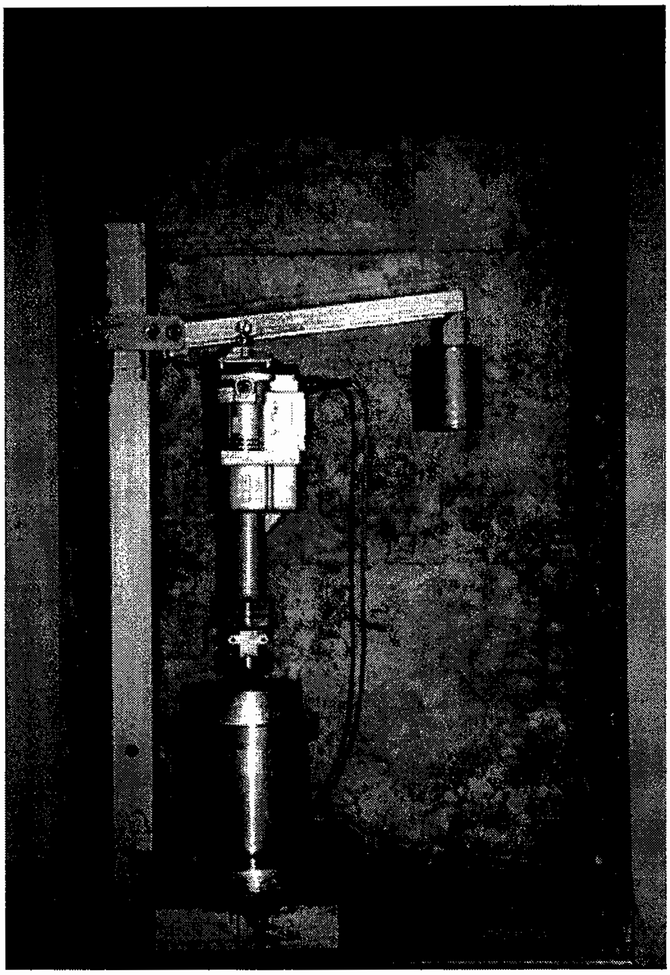


Figure 8.7 Compaction rig.

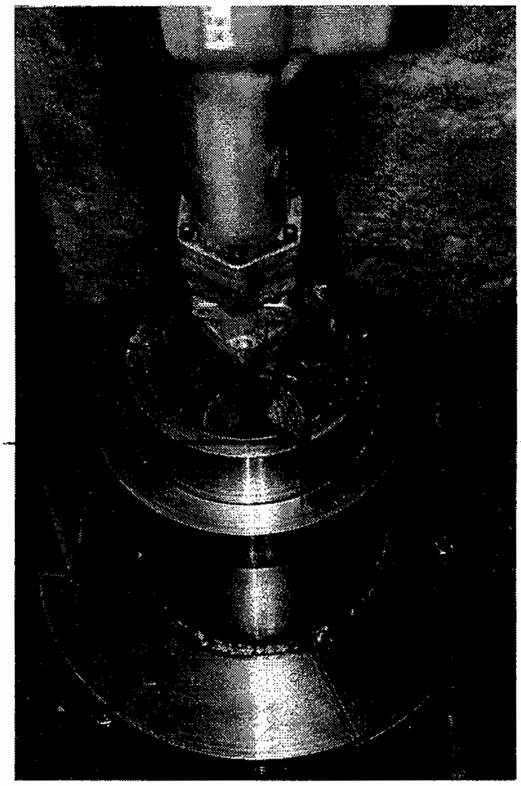


Figure 8.8 Compaction mould.

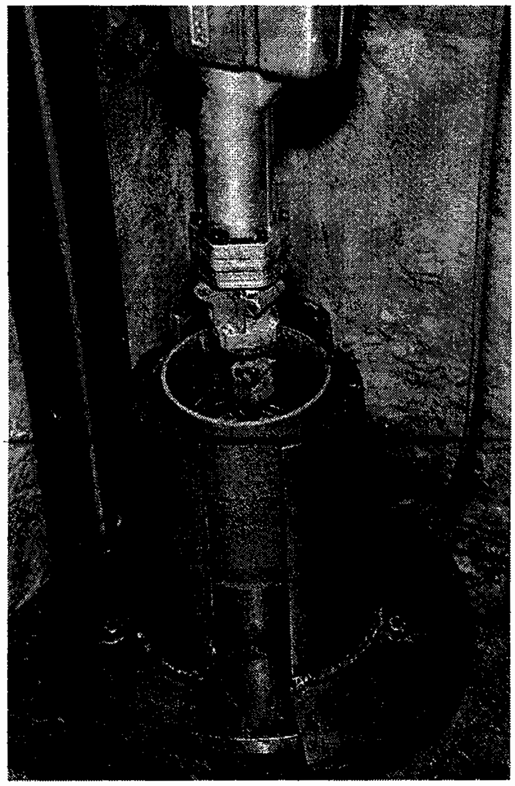


Figure 8.9 View of inner specimen mould.

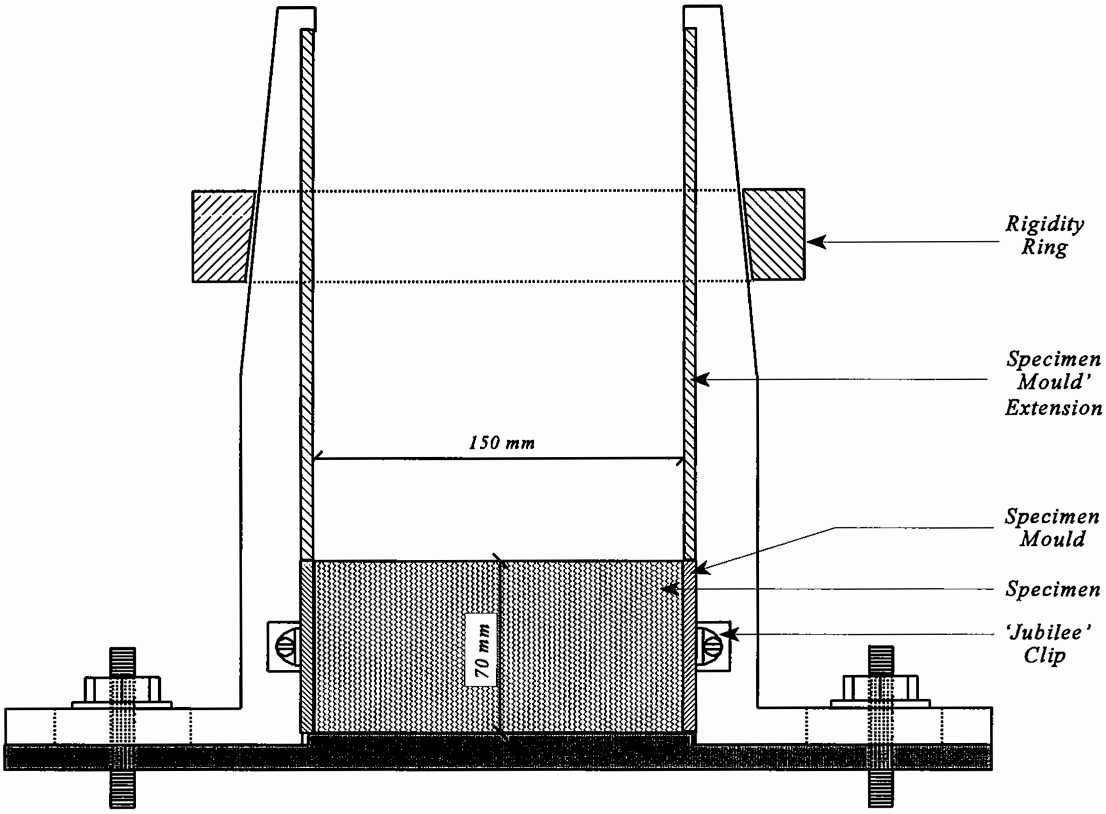


Figure 8.10 Scheme of the developed compaction mould.

8.6.2. Results

8.6.2.1. Indirect tensile strength

The results of the indirect tensile strength are presented in Figures 8.11 to 8.13 as a function of time for each family. All materials are ranked according to their tensile strength at 90 days (Figure 8.14).

8.6.2.2. Stiffness modulus

To quantify the stiffness modulus a constant Poisson's ratio of 0.15 was assumed. This value was based on the results obtained for other hydraulically bound mixtures which were presented in Chapters 6 and 7. This is also corroborated by values reported in the literature [Kolias & Williams, 1978].

The results of the stiffness modulus are introduced in Figures 8.15 to 8.17 as a function of the curing period. In Figure 8.18 the materials studied are ranked according to their indirect tensile stiffness at 90 days.

8.6.2.3. Durability

With regards to durability, the recommendations that resulted from the first study reported in Chapter 7 were taken into account, namely the consideration of two periods of immersion to provide extra information on the resistance to long term immersion. Owing to the large number of mixtures in the study a strong emphasis had to be put on the reduction of the number of specimens. Hence, for this project the following procedure was adopted which represents a compromise between the issues discussed above:

- curing of three specimens at 20 °C for 21 days followed by immersion in water at 20 °C for 7 days;
- all specimens tested for stiffness modulus at 28 days, in indirect tensile mode, using repeated loading;
- first set of two specimens tested for indirect tensile strength at 28 days;
- remaining specimen immersed again until completing 90 days curing;
- specimen tested for both stiffness and strength at 90 days.

This procedure was set after verifying that it was possible to compact a maximum of 15 diametral specimens from a mixture batch within two hours (limit for use of the uncompacted material).

The results obtained are presented in Table 8.7 in the form of a ratio between the mechanical properties assessed on the immersed specimens and on the control specimens cured at a constant temperature of 20 °C.

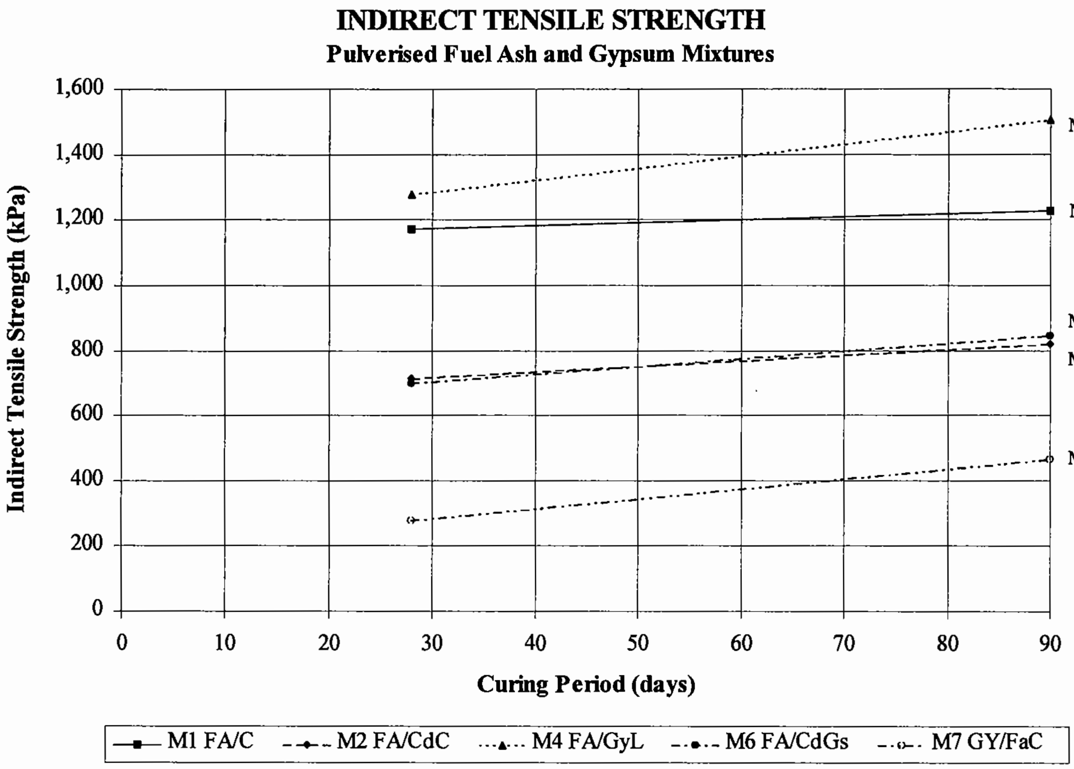


Figure 8.11 Indirect tensile strength of the mixtures with fly ash or gypsum as a major component.

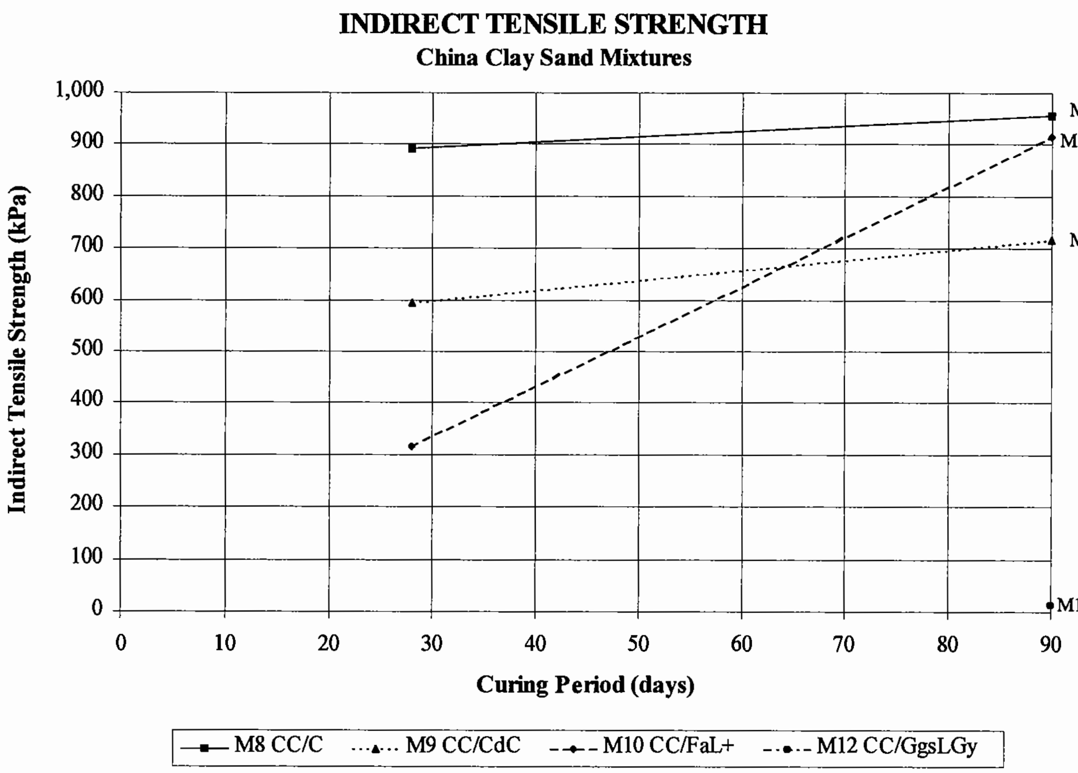


Figure 8.12 Indirect tensile strength of the mixtures with china clay sand as a major component.

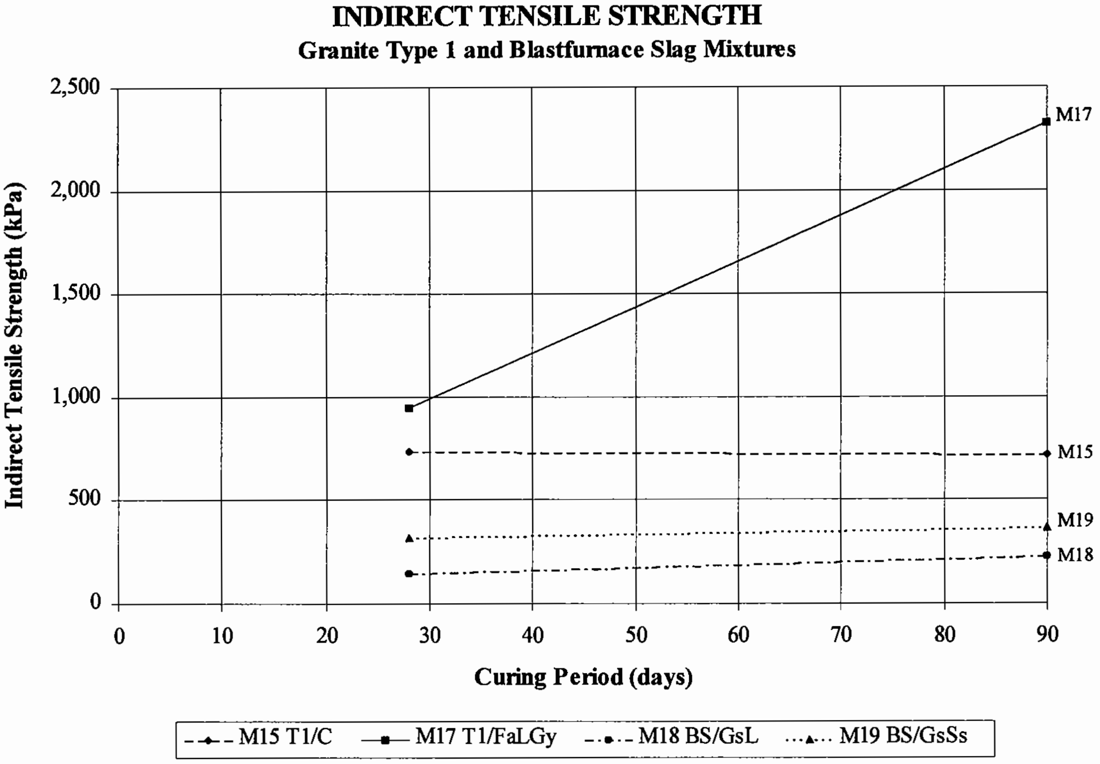


Figure 8.13 Indirect tensile strength of the mixtures with Type 1 aggregate or blastfurnace slag as a major component.

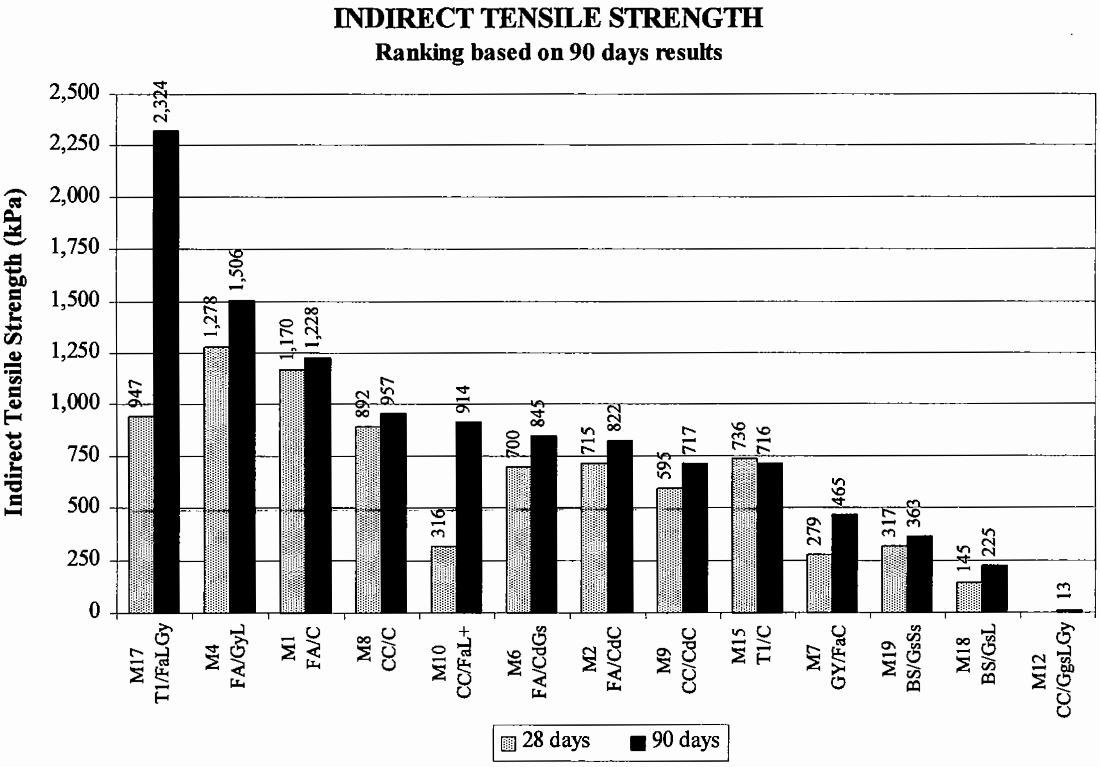


Figure 8.14 Ranking of materials based on the indirect tensile strength of the mixtures at 90 days.

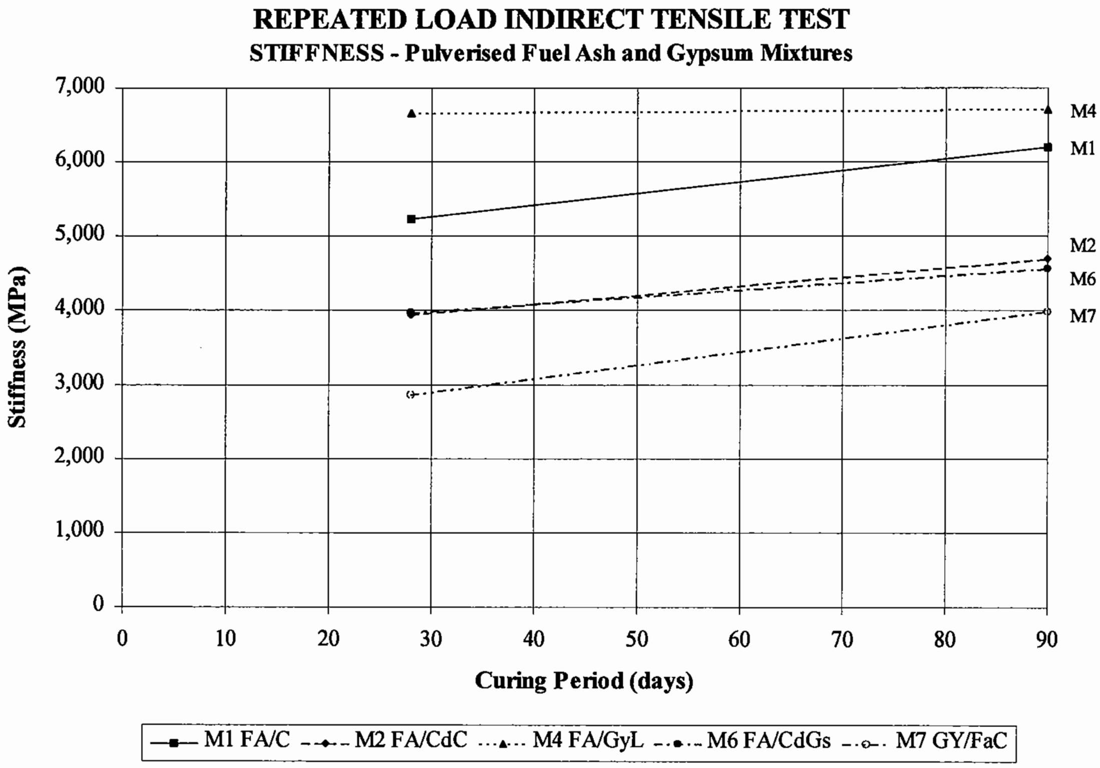


Figure 8.15 Stiffness of the mixtures with fly ash or gypsum as a major component.

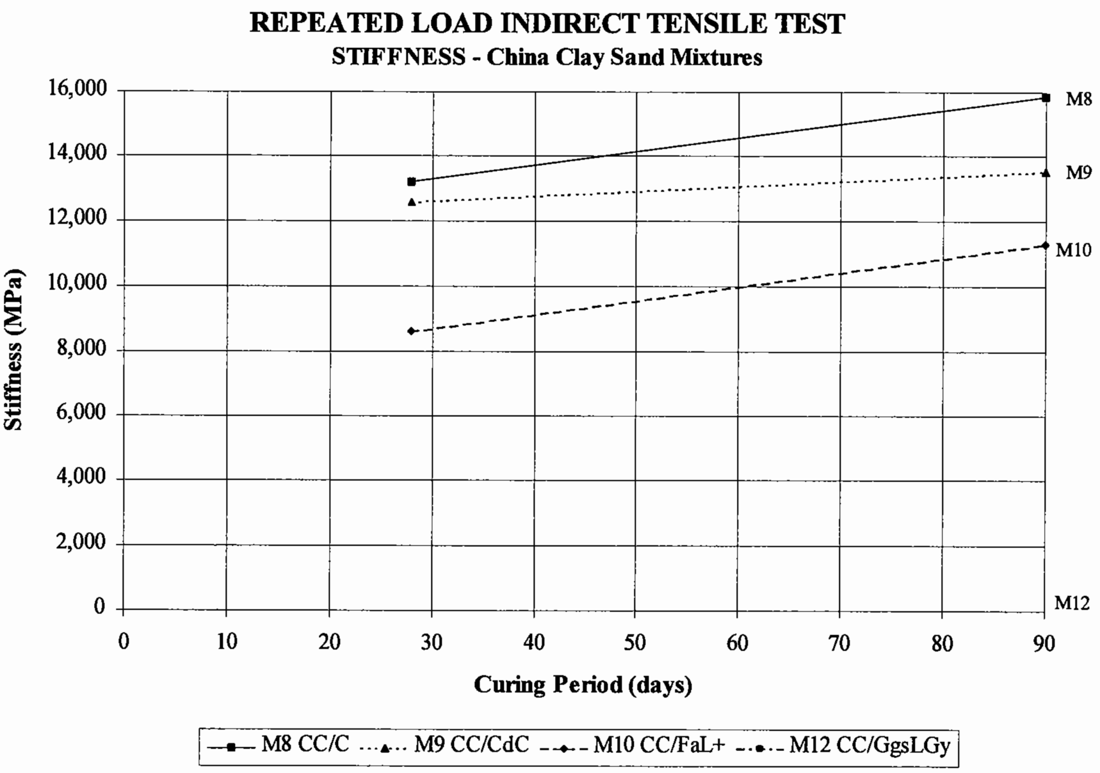


Figure 8.16 Stiffness of the mixtures with china clay sand as a major component.

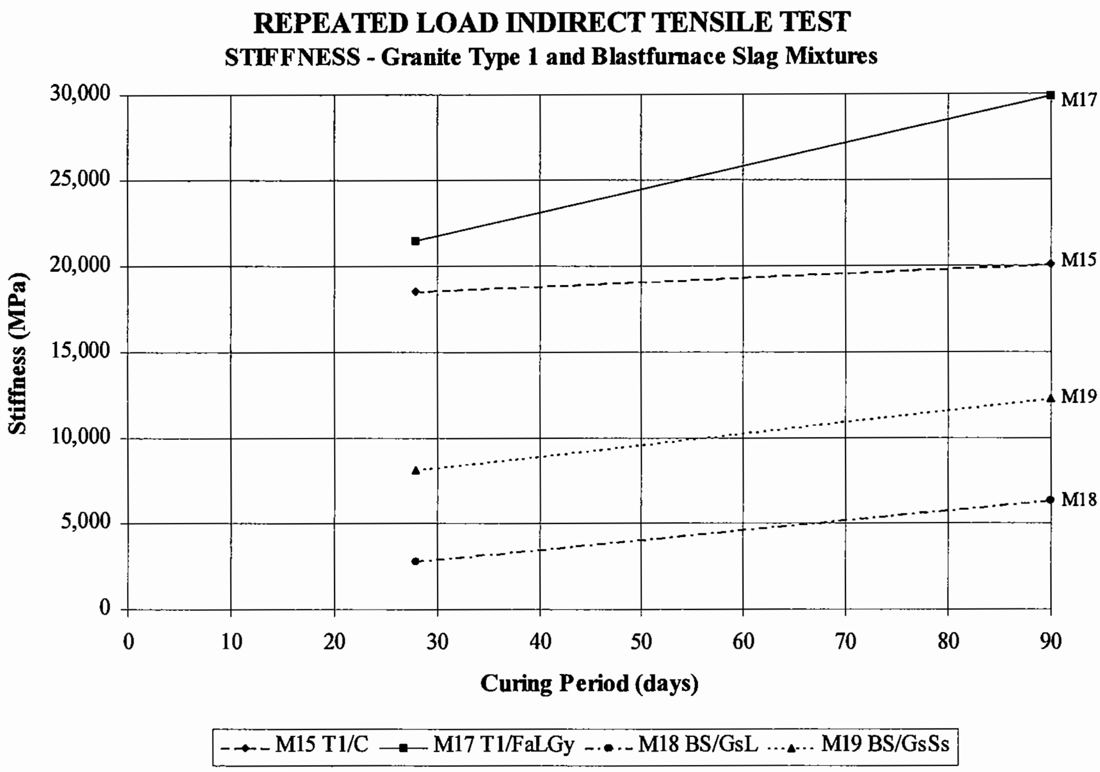


Figure 8.17 Stiffness of the mixtures with Type 1 aggregate or blastfurnace slag as a major component.

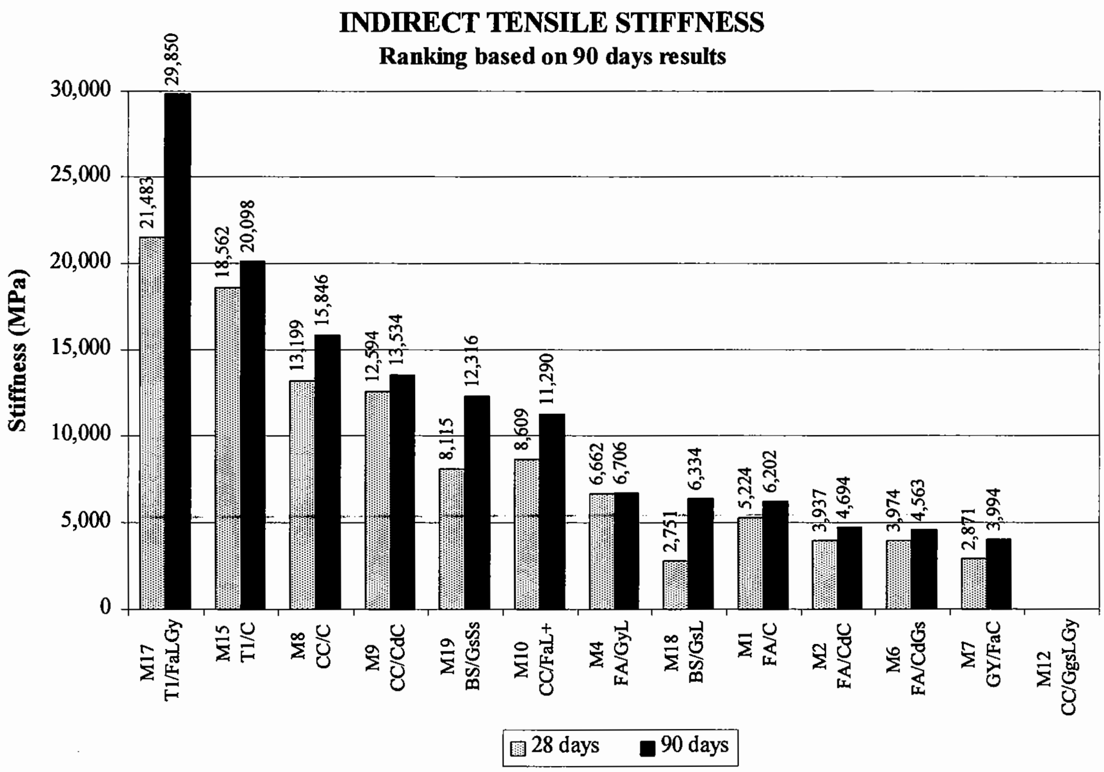


Figure 8.18 Ranking of materials based on the stiffness of the mixtures at 90 days.

Table 8.7 Results of the durability assessment.

Material Code		Major Comp	Binder									Ratio immersed / control (%)			
			C	L	Fa	Gs	Ggs	Cd	Gy	Ss	+	Strength @		Stiffness @	
		%	%									28 days	90 days	28 days	90 days
Pulverised Fuel Ash															
M1	FA/C	93	7									82	83	90	81
M2	FA/CdC	90	3					7				72	103	102	102
M4	FA/GyL	91		4					5			96	92	102	106
M6	FA/CdGs	70				10		20				88	93	109	131
FGD Gypsum															
M7	GY/FaC	82	8		10							63	63	89	75
China Clay Sand															
M8	CC/C	93	7									90	86	106	82
M9	CC/CdC	90	3					7				102	107	102	106
M10	CC/FaL+	80		4	15.						0.5	151	99	117	107
M12	CC/GgsLGy	92.5		1	5		6		0.5			---	---	---	---
Granite Type 1															
M15	T1/C	96	4									109	142	114	99
M17	T1/FaLGy	92		1.5	6				0.5			91	95	88	108
Blastfurnace Slag															
M18	BS/GsL	84		1		15						122	119	114	145
M19	BS/GsSs	70				20				10		---	87	---	89

KEY:
Aggregates
FA=pulverised fuel ash GY=gypsum CC=china clay sand T1=granite Type 1 BS=air-cooled blastfurnace slag
Binders
C=cement L=lime Fa=pulverised fuel ash Gs=granulated blastfurnace slag Ggs=ground granulated blastfurnace slag
Cd=cement kiln dust Gy=FGD gypsum Ss=steel slag + =sodium carbonate (Na2CO3)

8.6.2.4. Resistance to fatigue

The results obtained are shown in Figures 8.19 to 8.21 for each family of mixtures. In those graphs σ_0 represents the indirect tensile strength and σ_N the indirect tensile stress resulting in failure (defined as complete rupture) after N cycles of load application.

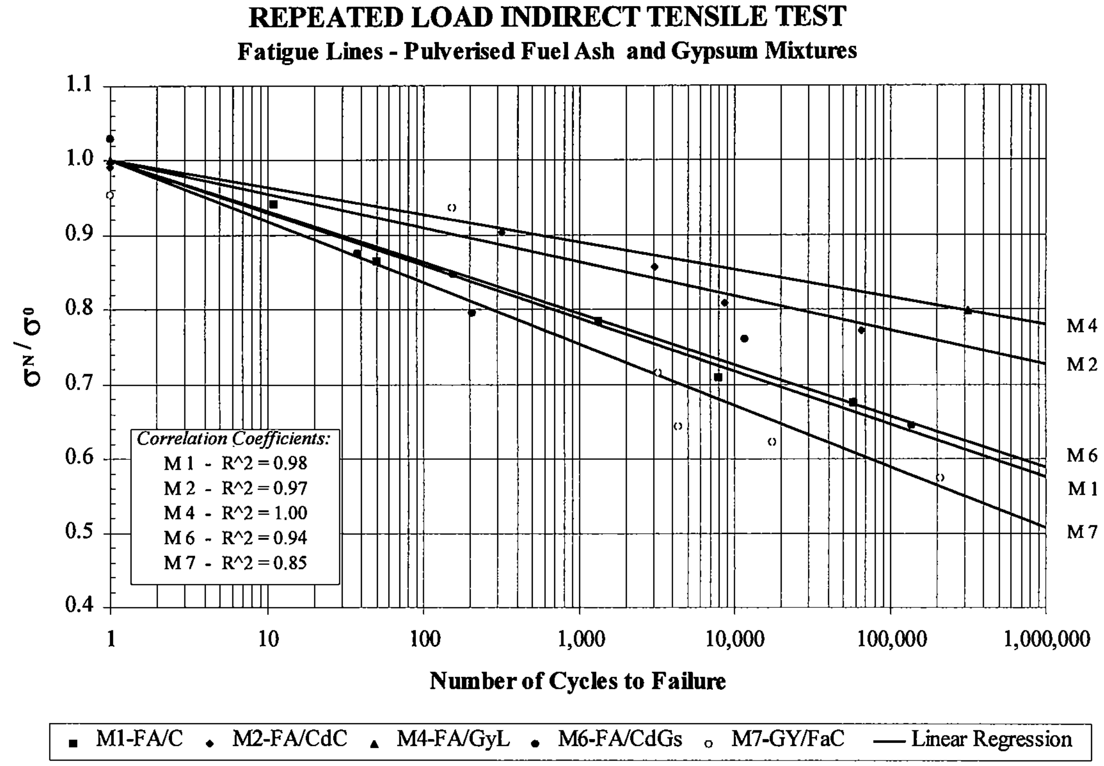


Figure 8.19 Fatigue lines of the fly ash and gypsum mixtures.

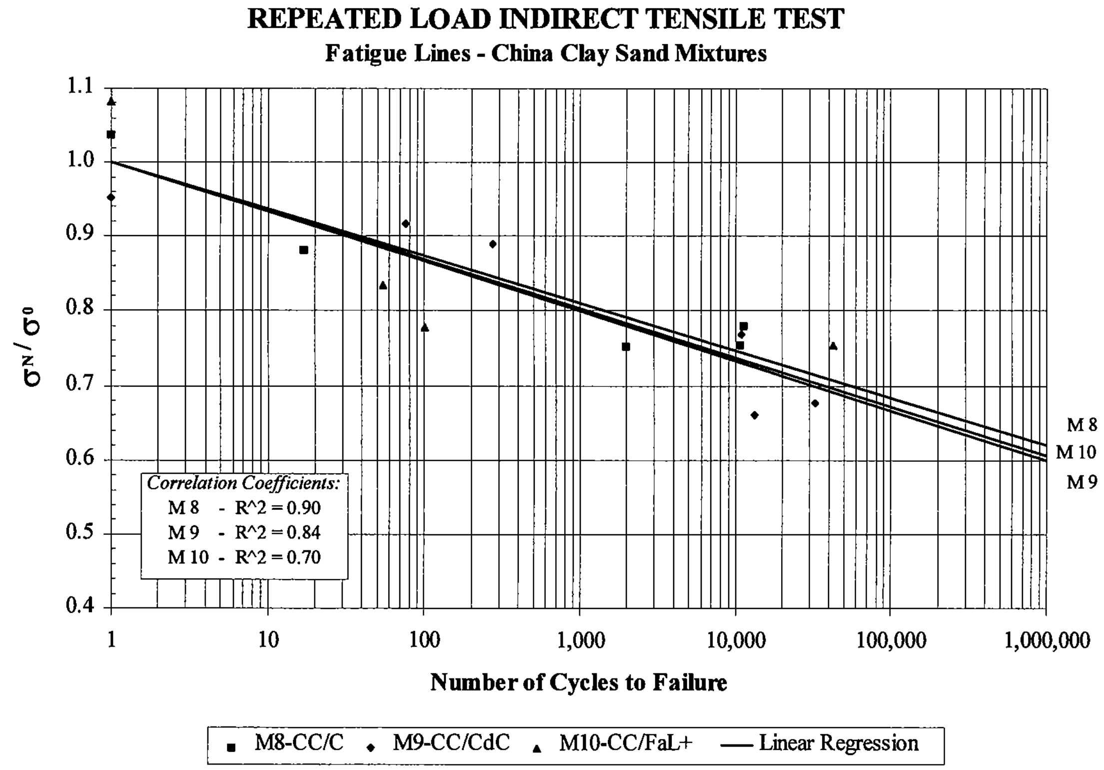


Figure 8.20 Fatigue lines of the china clay sand mixtures.

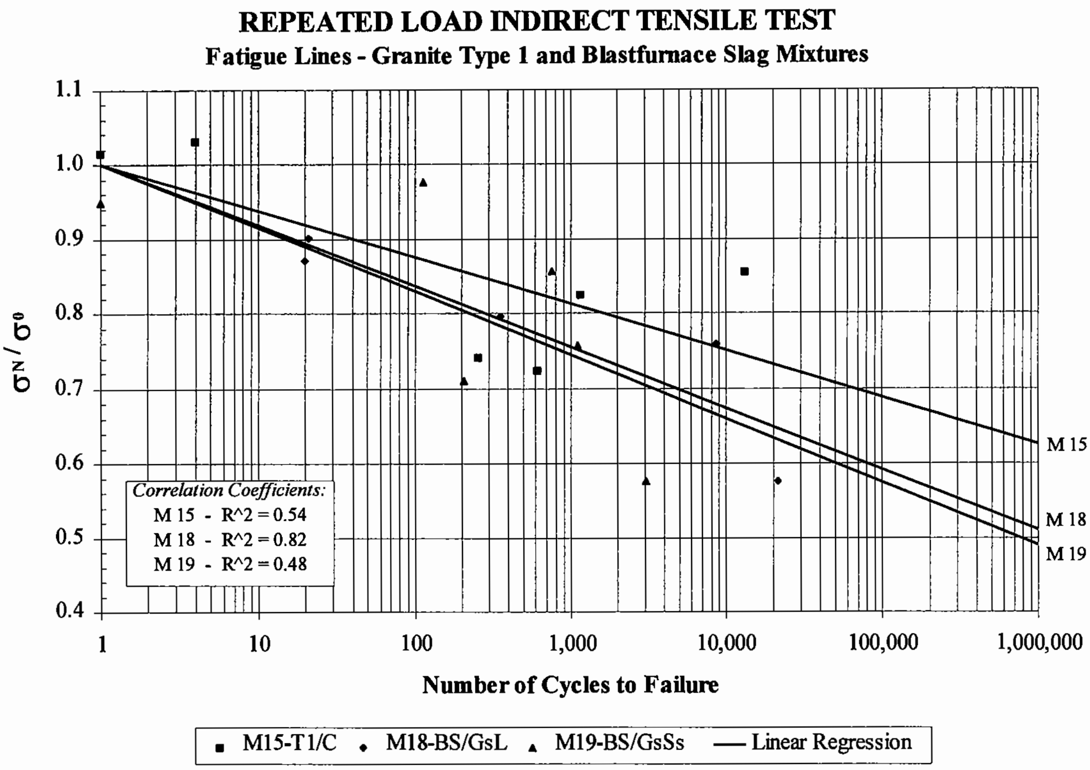


Figure 8.21 Fatigue lines of the Type 1 aggregate and blastfurnace slag mixtures.

8.6.3. Discussion of results

As for the compressive strength of cubes, there is no clear relationship between 28 and 90 day test values of mechanical properties such as indirect tensile strength and stiffness modulus. Models that may eventually be developed for one mixture will not be valid for all mixtures even when considered within the same family of materials. The ratio between properties obtained at 90 and 28 days varied considerably. The highest changes experienced occurred for the tensile strengths of mixtures M10 (Figure 8.12) and M17 (Figure 8.13) which illustrate a strong development of strength between 28 and 90 days of curing. In both cases a fly ash / lime type of binder was used, which was not included in any other mixture. With this type of binder a slow cementing process may be expected, which may justify higher improvements in relation to other faster mixtures, though the justification for the magnitude of the differences in strength is not completely clear. These results should be investigated to confirm whether this represents a real property of the materials.

Stiffness / Tensile Strength Relationship

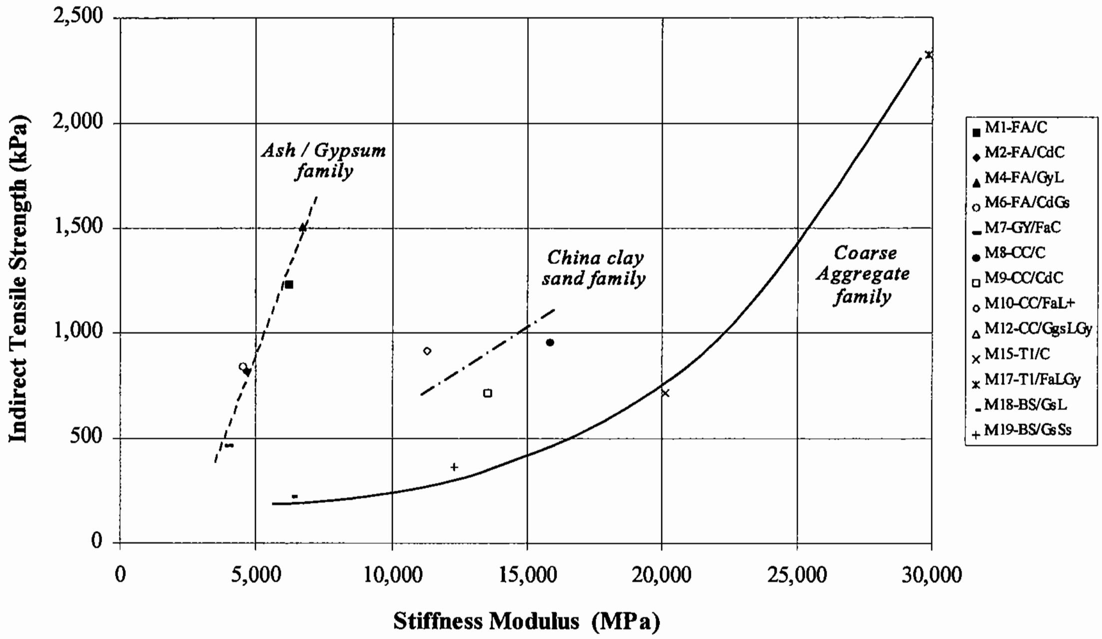


Figure 8.22 Analysis of relationship between stiffness modulus and tensile strength.

Compressive / Tensile Strength Relationship

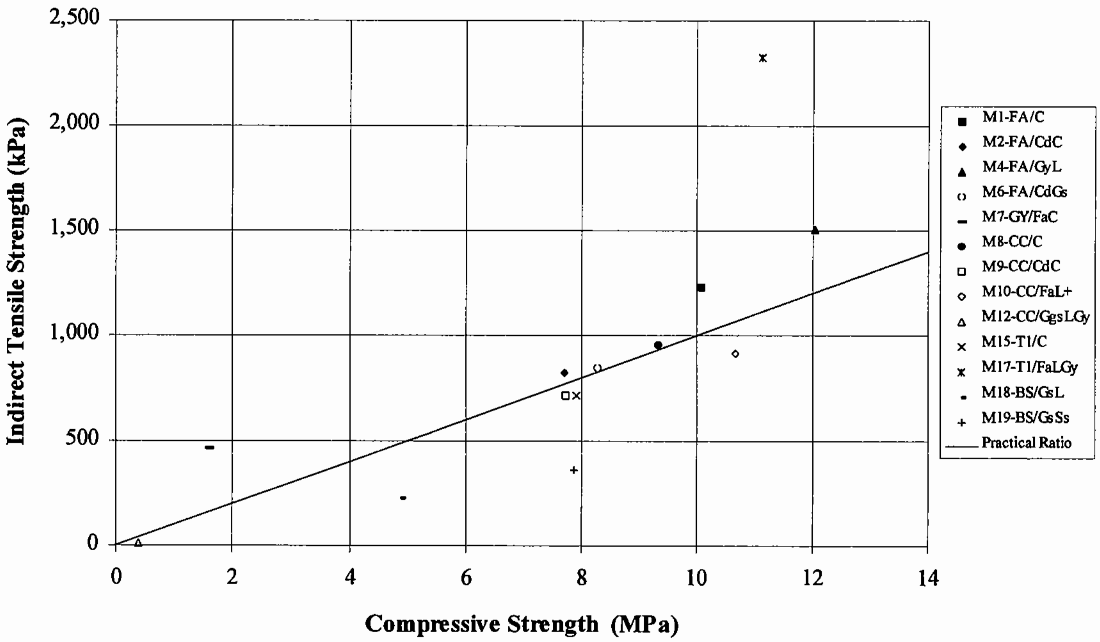


Figure 8.23 Analysis of relationship between compressive and tensile strengths at 90 days.

The cement stabilised type of materials (mixtures M1, M8 and M15) clearly develop strength at faster rates, as it would be expected, showing very flat lines from 28 to 90-day results, both in terms of tensile strength and stiffness modulus.

Similarly, no unique relationship between stiffness modulus and tensile strength (Figure 8.22) or between compressive and tensile strengths (Figure 8.23) was found in the analysis conducted. In the first case, however, the particle size of the mixtures' major component seems to have a strong influence on the results. In fact, the results may be grouped according to the family of the mixtures as shown in Figure 8.22. For the fly ash and gypsum family there appears to be a linear relationship between stiffness and strength while for the coarse aggregate family that relationship seems to be of the exponential type. The type of relationship for the china clay sand-based mixtures is not so clear because the results are grouped in the same area of the graph. Nevertheless, they are clearly located between the fine-graded and the coarse-graded mixtures emphasising the influence of aggregate size. These relationships should be further investigated in the future to assess the possibility of estimating stiffness modulus from tensile strength results.

In Figure 8.23, the relationship frequently adopted in practice between compressive and tensile strengths is also plotted. In average terms the practical ratio of 10:1 between those two properties represents a good approximation of the results. In fact, the average value obtained was 10.9:1. Nevertheless, one must bear in mind that some materials deviate considerably from the average results (especially M17) and so that relationship should be considered only for a first estimation of properties.

Any classification of the mixtures studied should be done by taking into account all important properties that will affect the performance of the material in the pavement. In this case, the mechanical properties that should be accounted for are tensile strength, stiffness modulus and fatigue characteristics, all of which were studied in this project. Compressive strength should be excluded from such a classification as it is not a relevant performance-related measure for road pavements.

It is possible to classify these materials for pavement design purposes by using the classification that has been proposed for fly ash bound mixtures (FABM) in the draft for European Standards [CEN, 1995 i]. The classification is based on the results at 360 days of the tensile strength of the mixture in direct tension and the stiffness modulus either in direct tension or in compression. The draft considers five grades ranging from materials with the lowest quality, grade G0, to the highest quality, grade G4. The resulting classification is listed in Table 8.8.

Table 8.8 Simplified classification of the materials based on stiffness modulus and tensile strength.

Material Code		Major Component	Binder									Simplified Classification
			C	L	Fa	Gs	Ggs	Cd	Gy	Ss	+	
		%	%									FABM
Pulverised Fuel Ash												
M1	FA/C	93	7									G 4
M2	FA/CdC	90	3					7				G 3
M4	FA/GyL	91		4					5			G 4
M6	FA/CdGs	70				10		20				G 3
FGD Gypsum												
M7	GY/FaC	82	8		10							G 2
China Clay Sand												
M8	CC/C	93	7									G 3
M9	CC/CdC	90	3					7				G 2
M10	CC/FaL+	80		4	15.5						0.5	G 3
M12	CC/GgsLGy	92.5		1			6		0.5			G 0
Granite Type 1												
M15	T1/C	96	4									G 2
M17	T1/FaLGy	92		1.5	6				0.5			G 4
Blastfurnace Slag												
M18	BS/GsL	84		1		15						G 0
M19	BS/GsSs	70				20				10		G 1

KEY:
Aggregates
FA=pulverised fuel ash GY=gypsum CC=china clay sand T1=granite Type 1 BS=air-cooled blastfurnace slag
Binders
C=cement L=lime Fa=pulverised fuel ash Gs=granulated blastfurnace slag Ggs=ground granulated blastfurnace slag
Cd=cement kiln dust Gy=FGD gypsum Ss=steel slag + =sodium carbonate (Na_2CO_3)

It was necessary to estimate 360 day results from the 90 day results using the following approximate relationships [CEN, 1995 i]:

$$\sigma_{td} = 0.8 \times \sigma_{ti} \qquad 8.1$$

$$\sigma_{td\ (360\ days)} = 1.5 \times \sigma_{td\ (90\ days)} \qquad 8.2$$

$$S_{m\ (360\ days)} = 1.3 \times S_{m\ (90\ days)} \qquad 8.3$$

where: σ_{td} tensile strength in direct tension;

σ_{ti} tensile strength in indirect tension or Brazilian splitting test;

S_m stiffness modulus.

The relationship between direct and indirect tensile strengths was already discussed in Chapter 7 (Section 7.2). The utilisation of the other relationships, between 90 and 360 days testing, for the materials for which the classification was developed (fly ash bound mixtures) has to be made with caution because of the existence of different relationships for different mixtures, as demonstrated by the results presented. Their application for materials other than fly ash bound mixtures is even less reliable, for example, their use for cement stabilised type of mixtures, which develop strength at a faster rate, would clearly overestimate 360-day properties.

A summary of the performances based on tensile strength, stiffness modulus and material grade in accordance with the CEN draft is given below [2].

---

2 **KEY:**

Aggregates

FA=pulverised fuel ash **GY**=gypsum **CC**=china clay sand **T1**=granite Type 1 **BS**=air-cooled blastfurnace slag

Binders

C=cement **L**=lime **Fa**=pulverised fuel ash **Gs**=granulated blastfurnace slag **Ggs**=ground granulated blastfurnace slag **Cd**=cement kiln dust **Gy**=FGD gypsum **Ss**=steel slag **+** =sodium carbonate (Na2CO3)

Mixtures

- best performance: M4-FA/GyL, M17-T1/FaLGy, M1-FA/C
- worst performance: M18-BS/GsL, M12-CC/GgsLGy

Aggregates (major component)

- best performance: FA, T1
- worst performance: BS

Binders

- best performance: FaLGy, FaC, FaCdGs, FaCdC
- worst performance: GsL, GgsLGy

Early-age strength / stiffness development

- Fast rate: testing at 7 days is more adequate for this assessment
- Slow rate: M7/GY/FaC, M17-T1/FaLGy and M10-CC/FaL+ for strength development; and M19-BS/GsSs, M18-BS/GsL for stiffness.

With regard to the durability assessment (Table 8.7) there is only one material raising some doubts in terms of long term durability, which is mixture M7 with gypsum as a major component. Mainly in terms of tensile strength the effect of immersion in water causes that property to decrease substantially (by 37 %).

In several cases, the immersion in water resulted in higher tensile strengths and stiffnesses. This results from the hydration of binder that did not react with the water used during compaction. This points out the fact that the optimum moisture content may not be always the most adequate and that some mixtures including secondary materials and binders may benefit from a compaction using slightly increased values. This would introduce another variable in the stabilisation process, that is, the moisture content optimisation to achieve the best end-product. In practice, it means an increase in the number of mixtures to be considered in any laboratory study owing to the consideration of mixtures with different binder contents as well as different moisture contents.

Regarding the results of the fatigue tests, their behaviour is not of much concern. The tensile stress causing failure after 10^6 cycles of load application is estimated to be 50 % of the tensile strength or higher for all materials studied. It is necessary to mention, however, that for mixture M4 the fatigue line presented (Figure 8.19) is only a crude estimation of the fatigue performance. As a matter of fact, only two specimens were tested due to the high tensile strength of this material and the limitations of the equipment in terms of maximum load capacity. For the same reason, it was not possible to determine the fatigue resistance of mixture M17.

The fly ash mixtures tend to be as strong against fatigue effects as the china clay sand mixtures or better, and the regression lines obtained with this material have the highest correlation coefficients. Typically, the tensile stress causing failure after 10^6 cycles ranges from 60 to 70 % of the tensile strength. However, the gypsum mixture considered in the same family is rather more susceptible to fatigue, that percentage being of approximately 50 % in this case.

With the exclusion of mixture M12, not considered for fatigue testing, the china clay sand mixtures tend to present a very uniform behaviour despite the type of binder used. In fact, the lines obtained are practically identical and the tensile stress after 10^6 cycles is approximately 60 % of the tensile strength.

The third family with granite Type 1 or blastfurnace slag as the major component gave the poorest results both in terms of fatigue susceptibility and quality of the regression to determine the fatigue line. The tensile stress after 10^6 cycles ranges from 50 to 60 % approximately of the tensile strength. A higher number of specimens would be essential for a more reliable characterisation of the fatigue properties of mixtures M15, M19 and also for M10 of the china clay family.

A comment should be made here on the grading of the granite Type 1 and blastfurnace slag used in this study. The maximum particle size was limited to 28 mm which, considering the diameter of the indirect tensile specimens, 150 mm, seems to be adequate enabling a relationship between the mould diameter and the maximum particle size higher than 5. Yet, the thickness of the specimen is only 70 mm and if this does not cause any problems in relation to the fly ash and china clay sand mixtures it may be

responsible for the lower correlation coefficients obtained in the fatigue lines of these mixtures. Indeed, the presence of large particles near the surface to which the load is applied and other size effects may influence the behaviour. Thus, it is recommended that this issue should be further investigated by comparing the results from indirect tensile specimens compacted according to the procedures described here, with the results obtained with cores (obtained from slabs either compacted in the laboratory or cut from a pavement).

8.6.4. Classification system proposed

The classification system proposed in the CEN draft [CEN, 1995 i] has the merit of incorporating in the same chart two important properties of hydraulically bound materials, stiffness modulus and tensile strength, which 'work' together to produce the final performance of the material in the pavement structure. This system is believed to derive from work conducted in France since a similar type of classification is included in the French specifications [SETRA & LCPC, 1984]. On the other hand, it has the inconvenience of using the properties of the materials determined at 360 days which is considered less practical or more dependent on empirical relationships between results at 360 days and other curing periods. The reliability of such relationships is limited to the material used for determining it and may differ significantly from material to material as demonstrated in previous sections. Another shortcoming of that classification procedure is that it does not consider another essential property of this type of material, that is, the fatigue resistance.

A new performance classification system was developed based on the existing CEN proposal but incorporating the issues mentioned above. The chart is presented in Figure 8.24 for which the properties are determined in the repeated load indirect tensile apparatus at 90 days rather than 360 days. For this purpose, the classes suggested by CEN were down rated using Equations *8.1* to *8.3*. Five classes of materials were considered, and were ordered from best (G1) to worst (G5) following the same criteria adopted in the classification chart proposed in Chapter 6, which was based on the results of the repeated load triaxial tests. The classes were slightly adjusted to produce more uniform middle classes (G2 to G4).

To consider the fatigue resistance of the materials a fatigue ratio (R_{fat}) was introduced in the Y axis which is multiplied by the indirect tensile strength. That ratio is defined as follows:

$$R_{fat} = \frac{\dfrac{\sigma_{N=10^6}}{\sigma_{ti}}}{\left(\dfrac{\sigma_{N=10^6}}{\sigma_{ti}}\right)_{Ref}} = \frac{\sigma_{N=10^6}}{0.6 \quad \sigma_{ti}} \qquad 8.4$$

where: $\sigma_{N=10^6}$ — indirect tensile stress resulting in failure after 10^6 cycles;

σ_{ti} — indirect tensile strength;

$(\sigma_{N=10^6} / \sigma_{ti})_{ref}$ — tensile strength ratio for a reference fatigue line.

The reference fatigue line was considered to correspond to the average fatigue characteristics obtained in this project for all mixtures tested. For this line the strength ratio $\sigma_{N=10^6} / \sigma_{ti}$ equals 0.6. This parameter may be modified in the future after further research is carried out. The objective is to represent an average hydraulically bound material, which is not necessarily equal to the average of the results obtained in this work.

In practical terms, for a material with average fatigue performance the fatigue ratio is equal to 1 and so the Y axis becomes tensile strength. When the materials are less susceptible to fatigue that ratio is higher than 1 and consequently it is plotted at higher levels in the Y axis, towards higher classes.

The results obtained in this research were plotted in the classification system and as Figure 8.24 shows, it enables a good discrimination of the mixtures studied. Materials such as M8, M10 and M6 are expected to have similar performance in the field, where a small increase in stiffness (M8) is also compensated by a small improvement of the tensile strength. Comparatively, these materials are expected to perform slightly better than mixtures M9 and M15, since having similar stiffnesses they will be subjected to the same level of stresses in the pavement (for the same traffic conditions) and their tensile strength is higher, which makes them less susceptible to fatigue cracking. Overall, class G5 includes materials with low combined stiffness and tensile strength values and for this reason it may be defined as containing improved granular materials. On the other hand, classes G1 to G4 include stabilised type of materials.

To further check the classification system the results obtained with pulverised fuel ash, stabilised with 2, 5 and 9 % of cement and presented in Chapter 7, were included in the same chart. They confirm a significant improvement of material performance by increasing the binder content from 2 to 5 %. From 5 to 9 % the mixture continues to improve but at much lower rates. It is also worth noticing that the performance of fly ash with 9 % cement resulted in a slightly lower position than mixture M1 (fly ash stabilised with 7 % cement). There are a number of factors that may be responsible for this result:

- the difference in the definition of the percentages of binder. In the study reported in this chapter the percentages of binder were by weight of the total dry material, while in previous chapters those percentages were considered in relation to the weight of the major component (binder / dry aggregate ratio). This reduces the difference in binder content;
- the use of different compaction methods.

Finally, these results seem to further confirm the existence of a linear relationship between stiffness modulus and tensile strength previously discussed.

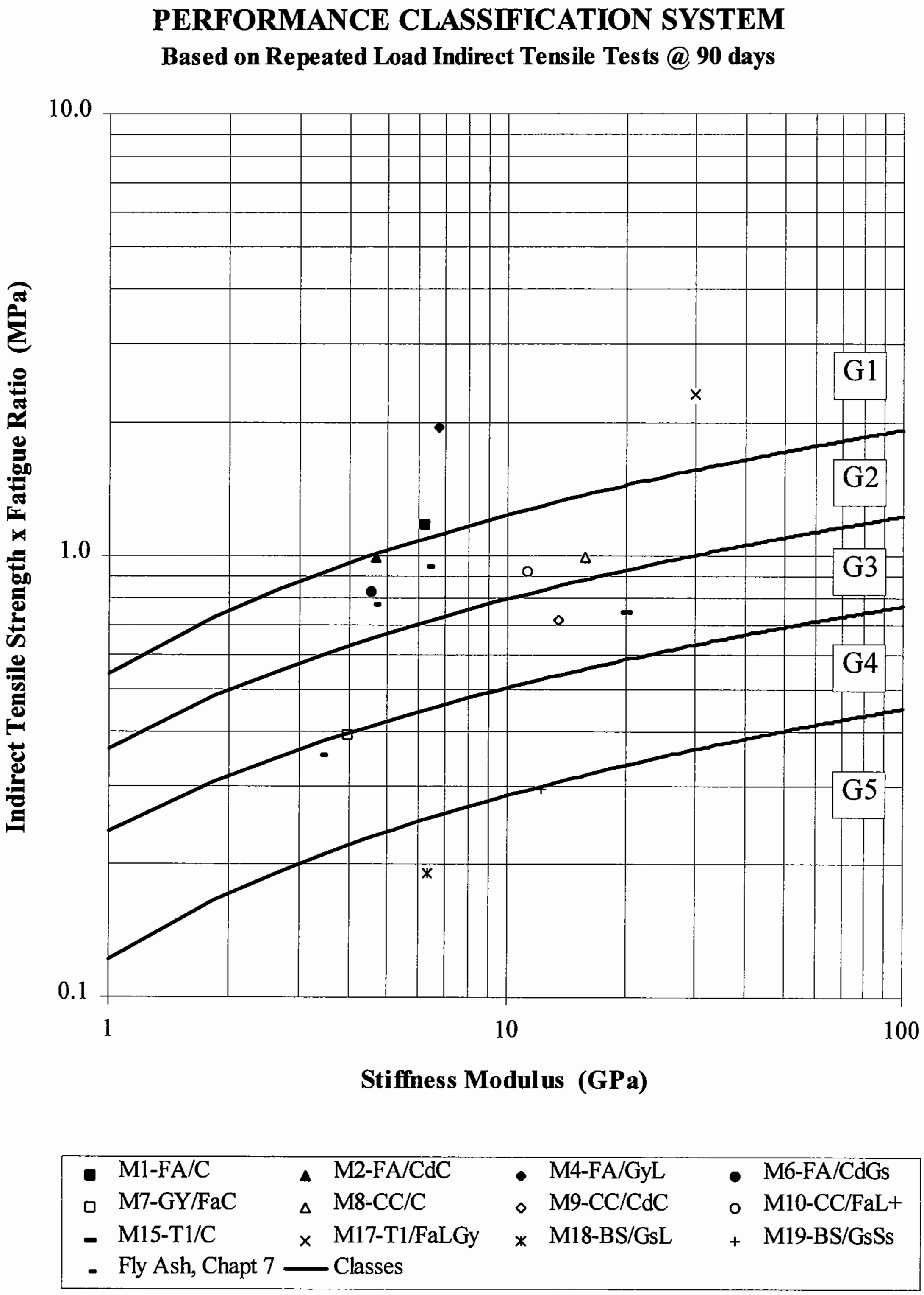


Figure 8.24 Classification system proposed.

8.7. Selection of mixtures for full scale trials

Based on the programme of testing described it was possible to select six materials which may be assessed in the TRL Pavement Test Facility after an optimisation of the mixtures presently being conducted. An overall appraisal of the materials' performance is carried out to help in the selection process.

Performance of materials

Only mixture M12 (CC/GgsLGy) was entirely unsuccessful. The results were obtained from batches made on different occasions so they appear to be valid for the constituents used. A modified mixture M12 tested in the repeated load indirect tensile apparatus has demonstrated good performance [Cheema, 1996]. This mixture comprised the following components (in brackets the original values of M12):

- china clay sand . . . . 88.0 % (92.5)
- ground granulated blastfurnace slag . . . . 10.0 % (6.0)
- lime . . . . 2.5 % (1.0)
- gypsum . . . . 0.5 % (0.5)

The reasons for the poor cementation of the original M12 are not clear but low binder and activator content are a possibility. This seems to be the case when compared with the French "*grave-laitier*", referred in Chapter 2, in which the binder is composed of

- granulated blastfurnace slag . . . . 8 to 20 %
- partially ground granulated blastfurnace slag . . . . . 8 to 15 %
- ground granulated blastfurnace slag. . . . . 3.5 to 5 %
- lime or gypsum with soda . . . . 1.0 %

Mixture M7 (GY/FaC) performed well when kept dry (except with regards to compressive strength) but gives rise to concern over its durability.

Of the remaining eleven mixtures all perform well enough to make them conceivable road construction materials. All mixtures (except M12-CC/GgsLGy) have stiffnesses significantly higher than conventional sub-base aggregates, and the stiffest material approaches the behaviour of lean concrete. On the other hand, for the lowest strength mixtures (M18-BS/GsL and M19-BS/GsSs) rutting is not likely to be of concern owing to the very low permanent deformations measured in the repeated load triaxial tests performed on lightly-treated materials (see Chapter 6).

The best performing mixture is M17 (T1/FaLGy) which has a high stiffness modulus and high tensile strength. In a pavement this could lead to fatigue cracking problems as the stress would be concentrated in a layer of this material. Such cracks would tend to be widely spaced because of the high strength and this could result in reflective cracking in bituminous overlays. However, construction techniques may be adopted to reduce this possibility.

Mixture M18 (BS/GsL) has a reasonably high stiffness but low strength (compressive and tensile). In practice this is likely to lead to frequent small cracking and a consequent reduction in overall stiffness when placed as a pavement layer. As a result, it may be advisable to consider M18 as an improved granular material.

Considering the results more generally it is evident that a fly ash / lime / gypsum binder functions well. Other binders incorporating fly ash were also often found in the more successful mixtures. These could be considered for a more general use. Conversely, the binders containing slag were generally found in the moderate or poorly performing mixtures. This suggests that slag mixtures may require more careful attention in order to get the best results. Cement kiln dust appears to function as an adequate mechanical cement replacement (given the extended curing times), albeit requiring higher treatment levels.

Considering the material forming the major component of the mixtures, fly ash and conventional aggregate seem to give rise to beneficial behaviour whilst the gypsum and slag-based mixtures performed less well.

Selected mixtures

For the selection of the six mixtures the following acceptance criteria were considered:

- available and relatively economic to supply;
- environmentally acceptable;
- compressive strength above 1 MPa at 7 days;
- stiffness modulus above 1 GPa at 28 days;
- less than 30 % drop in strength due to immersion;
- less than 50 % drop in strength after 10^6 cycles.

Bearing in mind these acceptance criteria, two mixtures were selected from each family and materials that demonstrated top and bottom performances within the same family. The following mixtures were selected:

- *Ash or gypsum-based mixtures* M4 and M6;
- *China clay sand-based mixtures* M9 and M10;
- *Coarse aggregate-based mixtures* M17 and M19.

After the chosen materials are evaluated in the pavement trials, it is anticipated that the performance of the ones with intermediate properties may be estimated.

8.8. Summary

- A comprehensive laboratory assessment of thirteen hydraulically bound mixtures including primary and secondary aggregates and binders was conducted. The testing programme included compactibility tests, determination of the compressive strength of cubic specimens and the determination of strength, stiffness and fatigue properties using repeated load indirect tensile tests. In addition, the durability of the mixes was assessed by quantifying the effect of immersion in water on the strength and stiffness determined in indirect tensile mode.

- Compactibility tests were carried out with some variation in relation to the British Standard methods. In particular, the amount of material used in each portion was roughly determined in advance, in order to produce specimens with approximately the same height after compaction. This is an important modification for secondary materials for which the range of particle densities is wide and as a result following the British Standard the compaction energy may be applied to very different volumes of material.

- A new compaction technique was developed to manufacture indirect tensile specimens. The standard rig to perform the compactibility tests was used and a three-piece metallic mould designed to fit in that rig. This procedure is more expeditious and the specimens are compacted in one layer, in their final form, using a vibrating hammer in the same conditions as for the compactibility test. The use of a polymeric specimen case protects the specimen during curing and overcomes corrosion problems experienced with metallic moulds.

- No unique relationships were found between the mechanical properties of the materials and the curing time, between stiffness modulus and tensile strength or between compressive and tensile strength. However, when the analysis was conducted within the same family of materials a strong influence of aggregate size on both stiffness modulus and tensile strength was observed. Relationships of the linear and exponential types were found for the fly ash / gypsum family and for the coarse aggregate family respectively. Existing empirical equations have a restricted applicability to the materials for which they were determined.

- Immersion results seem to indicate the need to optimise the compaction moisture content on the basis of binder hydration and density to achieve the best final product.

- Fatigue results presented higher scatters for the coarser mixtures stressing the need to test a higher number of specimens and to study the effect of compaction technique and specimen dimensions on fatigue results.

- A new classification system based on the results of repeated load indirect tensile tests was developed. This system evolved from the existing CEN proposal and answers to the shortcomings here identified. In particular, properties are determined at 90 instead of 360 days, and the resistance to fatigue of the materials is considered through the inclusion of a fatigue ratio.

- Apart from mixtures M12 which was totally unsuccessful and M7, essentially due to durability doubts, all the other mixtures demonstrated moderate to high potential for pavement construction. The best performances resulted from mixtures with fly ash and conventional aggregate as major components and fly ash or combinations of fly ash / lime / gypsum as binders.

- Six of these mixtures were selected for assessment in full scale trials to be conducted in the Pavement Test Facility of the Transport Research Laboratory.

CHAPTER 9

OVERALL DISCUSSION AND DESIGN CONSIDERATIONS

9.1. Introduction

The experimental results obtained with unbound and stabilised materials presented in previous chapters are now considered in overall terms, with the objective of comparing the type of information that results from conventional tests with that obtained from the more fundamental tests used during this research. This comparison is carried out in terms of material discrimination and ranking since the properties measured differ in each case. The mechanical characteristics of the materials determined experimentally are used in the structural analysis of typical pavement sections, in order to illustrate their application to the design procedure and to emphasise the potential of alternative materials in layers of the pavement as high as the roadbase.

9.2. Unbound materials

Particle strength tests and the California Bearing Ratio test are currently used for the characterisation of unbound materials and requirements relating these properties are specified by the Department of Transport. In Figure 9.1 results obtained with several materials are presented. The information resulting from Aggregate Crushing Value (ACV), Aggregate Impact Value (AIV) and dry Ten per cent Fines Value (TFV) tests is similar. Note that higher ACV and AIV values correspond to higher percentages of fines generated during the test, that is, weaker aggregate particles and lower TFV. However, better discrimination of materials was obtained using the dry TFV test.

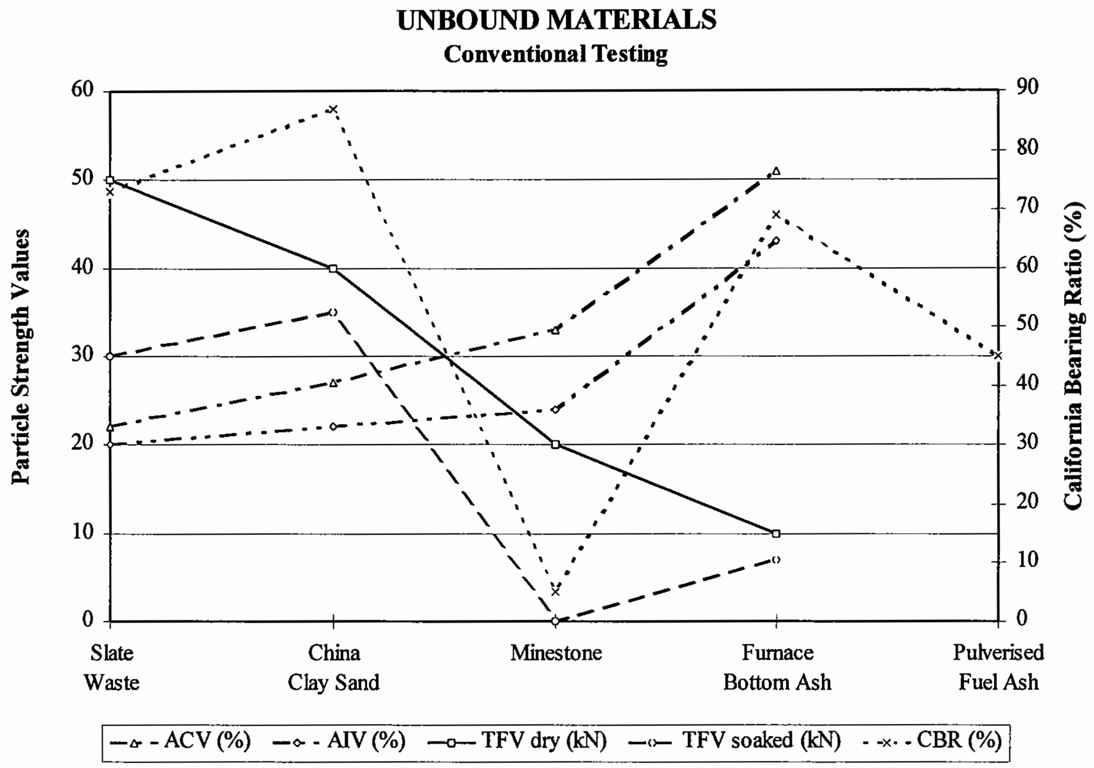


Figure 9.1 Particle strength and CBR results.

For the materials tested, the CBR test results present a pattern similar to the soaked TFV results. Of the materials shown in Figure 9.1, only slate waste and china clay sand were tested in an unbound form in the repeated load triaxial apparatus. Both resilient modulus and triaxial strength obtained in that equipment show a different tendency, with slate waste demonstrating slightly better properties (Chapter 6), contrary to the information from the soaked TFV and CBR tests. In the case of the TFV test, the level of stress to which the particles are submitted during the test is excessively high when compared with the pavement conditions. In addition, the fractions used for testing are not representative of the grading used in the field which contributes to the inability of this test to predict the performance of the material in pavement layers.

On the other hand, and despite being widely used to test soils and granular materials, the CBR test has also important shortcomings:

- the load is applied monotonically instead of repeatedly for better simulation of traffic loading;
- the stress state induced in the specimen is difficult to determine and highly variable, with very high stresses in the proximity of the load plunger contrasting with modest stresses caused in other parts of the specimen. That stress state is not taken into account when determining the CBR value;
- the test does not simulate any situation in the field;
- the plunger size is small relative to coarse aggregates when compared with the relationship between vehicle tyre and aggregate size.

Brown [1996] pointed out that CBR relates to shear strength and that for this reason it is surprising that it is used to estimate resilient properties such as the resilient modulus, an issue that is reinforced by the unrepresentative results obtained.

9.3. Stabilised materials

Regarding lightly stabilised materials, a comparison of results obtained using compressive strength tests and repeated load triaxial tests is shown in Figure 9.2. For each material the strength or resilient modulus (respectively) is plotted as a percentage of the maximum value obtained in the group of four materials presented. This procedure permits a more direct comparison of results and the assessment of the discrimination capabilities of the tests. The materials were ranked taking into account the measured resilient modulus, whose rank approximates the one given by the classification system of Section 6.8. A different ranking is obtained based on compressive strength, with the greatest discrepancies observed for stabilised china clay sand and minestone.

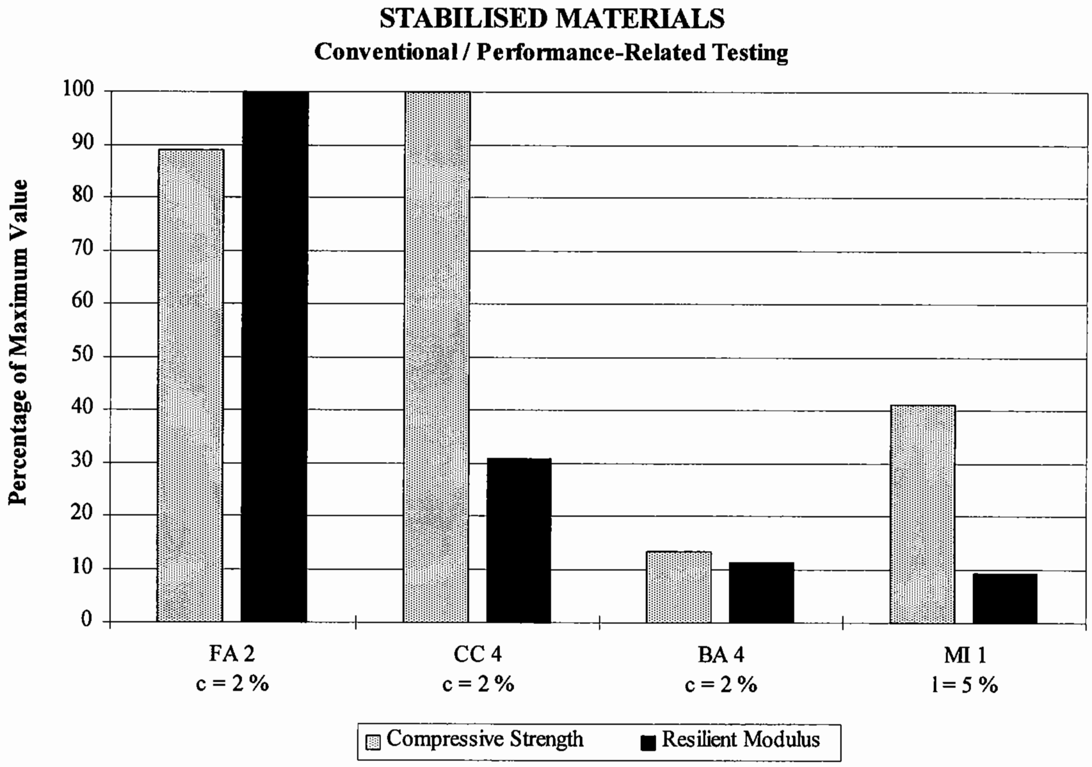


Figure 9.2 Comparison of results (light stabilisation).

For the more heavily stabilised materials, the results are presented in Figure 9.3 in terms of compressive strength, tensile strength and stiffness modulus obtained in indirect tensile mode. In addition, the ranking resulting from the application of the performance classification system proposed in Chapter 8 is also shown which represents the most satisfactory characterisation. It can be seen that compressive strength does not follow the same pattern as that of the ranking line, confirming previous results. Furthermore, none of the tests *per se* are capable of predicting performance. For example, a high stiffness needs to be combined with high tensile strength and adequate fatigue resistance to ensure a good performance.

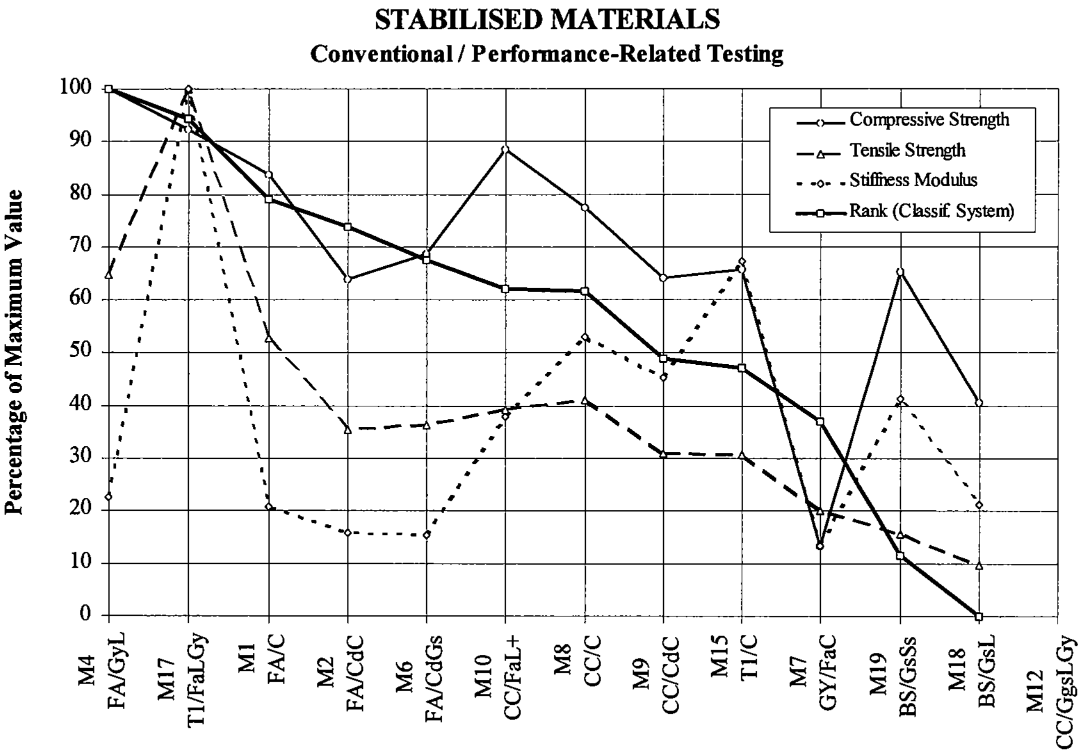


Figure 9.3 Comparison of results (heavy stabilisation).

The use of the compressive strength tests for the characterisation of stabilised materials may, once again, be questioned based on the analysis presented above. It seems clear that it produces different rankings from those obtained with performance related tests and thus it is not a totally reliable indicator of material performance. In addition, the failure mechanisms in pavement structures do not involve failure in compression of stabilised layers and so it is not a relevant property on which to base material characterisation.

9.4. Pavement design considerations

9.4.1. Behaviour of flexible composite pavements

The design of pavements incorporating layers with hydraulically bound materials requires a different approach to that for flexible pavements due to the specific aspects of the behaviour of these materials under traffic loading. In particular, these materials tend to crack and this aspect has to be taken into account because of the consequent modifications introduced to the stiffness and stress state. Eventually these cracks may progress to the surface, a phenomenon known as reflective cracking, and ultimately be responsible for the failure of the pavement.

There are two types of cracking occurring in hydraulically bound materials, primary and secondary cracking. For the type of pavement considered here, layers with these materials are laid without joints, except construction joints. These layers tend to contract due to:

- hydration of binder;
- variations in moisture content within the layer;
- daily or seasonal temperature reductions.

As this contraction is restrained by the friction with confining layers, high tensile stresses are then generated in the longitudinal direction, and consequently contraction cracks may be generated in the transverse direction. This process is known as *primary cracking*. Based on information gathered regarding the French road network, the main factors that seem to influence the process are [LCPC, 1988]:

- type of aggregate - its influence is mainly the result of the variation in the coefficient of thermal expansion among aggregates;
- grading - in general stabilised sands are less susceptible to crack, although this advantage is counterbalanced by the weaker residual particle interlock across the crack surfaces which tends to disappear over the time;
- type and quantity of binder - roads with sand-fly ash or sand-cement layers

present less cracks than others incorporating lean concrete, or gravel-pozzolana-lime and gravel-cement mixtures;

- construction period - this is related to the average temperature at the time of construction, with layers built during colder months showing reduced cracking;
- annual climatic conditions - exceptionally severe winters may cause the sudden appearance of widespread cracking;
- traffic - it is not a determinant factor in this type of cracking but it has an amplifying effect;
- thickness and type of bituminous surfacing - in some cases it affects crack spacing, although in some cases no effect is observed.

It is apparent from these observations that thermal contraction is the principal factor responsible for primary cracking, whose spacing may vary from 5 to 20 m or more [LCPC, 1988]. Thermal stresses decrease substantially with the depth of the layer in the pavement and as a result their effect is more important before the placement and compaction of overlying materials.

When transverse cracking is closely spaced and maintains a good aggregate interlock and friction, the performance of a pavement is generally good despite the discontinuities. In fact, one of the techniques to control this type of cracking is to induce them during construction through cuts performed along the complete width of the pavement in the top part of the hydraulically bound layer. Usually, spacings of 3 to 5 m are used in an attempt to reduce the crack widths and to assure good load transfer capabilities. The current Design Manual for Roads and Bridges [DoT, 1994] also confirms that primary cracks do not have a significant effect on the structural performance of the pavement, except when the subgrade soils are moisture susceptible.

When primary cracking does occur, the stress regime at the crack edges is altered under traffic loading, in particular the tensile stresses in the base of the hydraulically bound layers, parallel to the crack, increase. The combined effect of traffic and temperature may cause further cracking, longitudinal in the first instance and known as *secondary cracking*, which results from the accumulated damage caused by the repeated action of wheel loads. These cracks are normally initiated at the primary cracks' location and are facilitated by other factors such as:

- the deterioration under traffic of the surfaces of transverse cracks due to increased displacements in this area;
- when the crack propagates to the surface, the infiltration of water may cause more damage to the structure.

Contrary to what happens with transverse cracking which tends to close under the action of traffic-induced longitudinal forces, this does not happen with secondary cracking. These aspects and the drastic reduction in stiffness modulus further accelerate pavement damage and ultimately result in failure.

9.4.2. Design criteria

Roads in the UK are designed to have a 40-year life [DoT, 1994]. However, assessment of the whole life cost shows that it is economically preferable for flexible and flexible composite pavements to be designed to 'critical' condition for a 20-year period, after which major maintenance is carried out in the form of overlying and / or partial reconstruction. Other types of pavements, namely rigid and rigid composite are designed to failure condition for the complete 40-year period. 'Critical' condition is defined as corresponding to a 10 mm structural rut or to the first appearance of longitudinal cracks in the wheel paths. On the other hand, failure condition corresponds to 20 mm structural rut or extensive cracking.

The design criteria used for pavement structures incorporating layers with materials stabilised with hydraulic binders are:

- horizontal tensile stress (or strain) in the base of bound layers. This criterion aims to limit fatigue cracking;
- vertical compressive stress (or strain) in the top of particular unbound layers, generally the subgrade. This criterion aims at the limitation of permanent deformation in the structure.

For bituminous layers the analysis is generally conducted in terms of strains (ϵ_t), whereas for hydraulically bound layers the stress criterion (σ_t) is usually adopted. These are illustrated diagrammatically in Figure 9.4. In flexible and flexible composite pavements rutting is analysed by reference to the subgrade strain (ϵ_z).

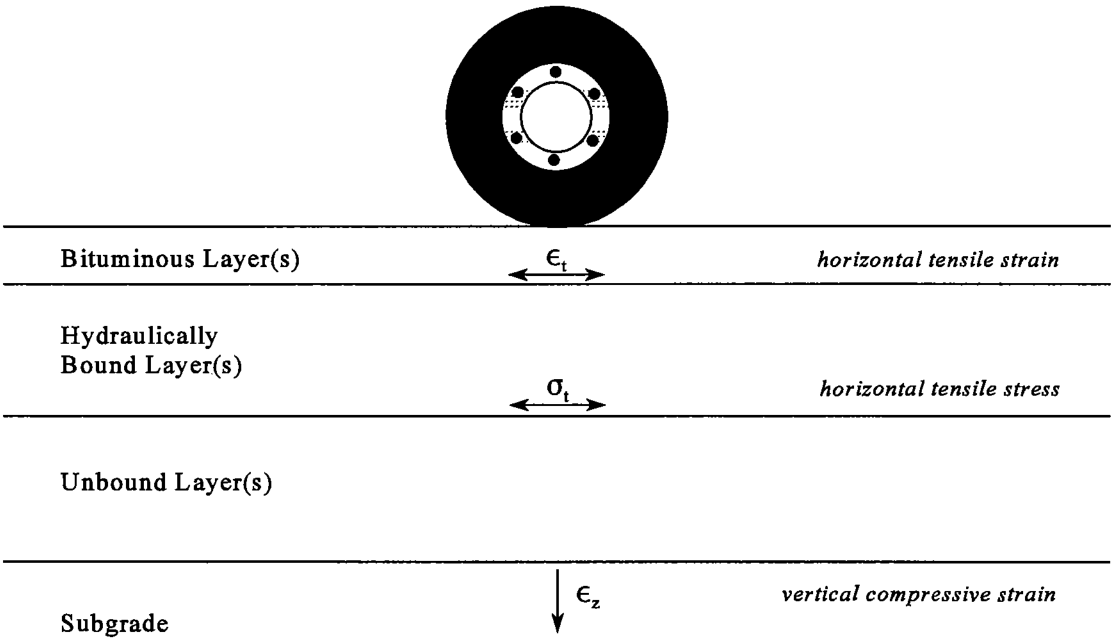


Figure 9.4 Design criteria used for pavement analysis.

9.4.3. The DoT approach

The Design Manual for Roads and Bridges [DoT, 1994] presents the mandatory design procedure of the Department of Transport to be used for trunk roads including motorways. It contains design charts for the determination of the thicknesses of:

- capping and sub-base layers based on the subgrade CBR value;
- roadbase in cement bound material (CBM3 or CBM4) and bituminous material. These thicknesses are obtained directly as a function of the design traffic.

A sample application of this Manual was carried out for the design of a typical flexible composite pavement section. The resulting thicknesses obtained for the loading, base material and subgrade condition indicated are shown in Figure 9.5.

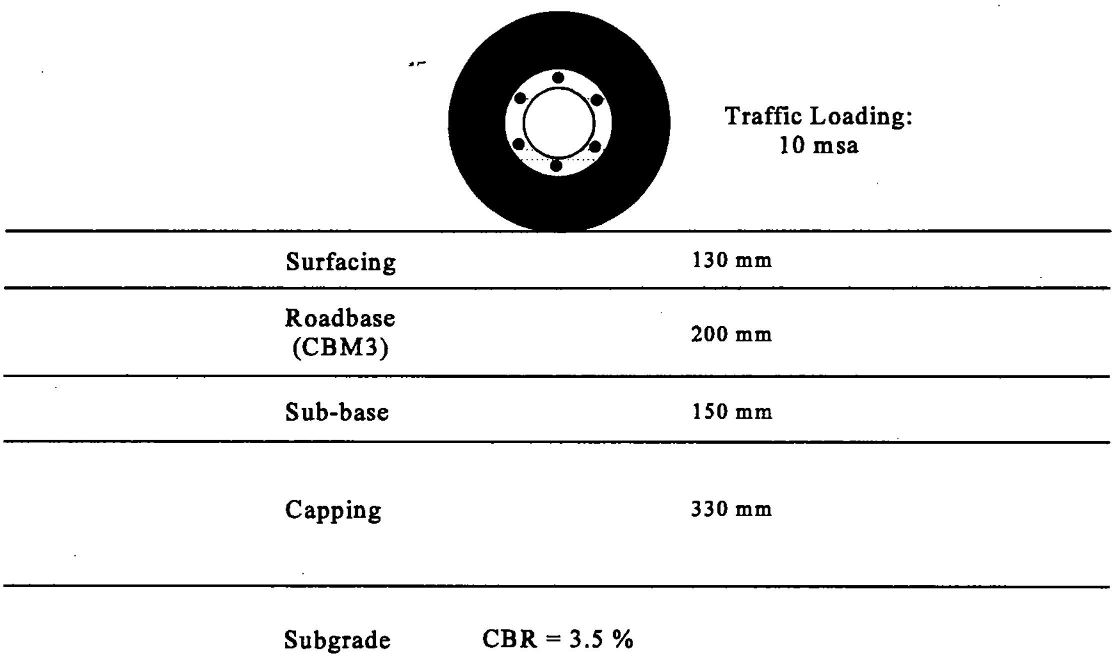


Figure 9.5 Results of the DoT approach to pavement design.

9.4.4. The analytical approach

For the analytical methods of pavement design, also known as mechanistic or rational methods, the pavement is associated with a multi-layer system and the analysis conducted making use of Burmister's theory of stresses and displacements in layered systems [Burmister, 1945]. This theory is based on the following assumptions:

- materials forming the layers are homogeneous, isotropic and linear elastic, for which Hooke's law is valid. The materials are characterised by a constant Young's modulus and Poisson's ratio for each layer of the system;
- layers are infinite in extent with limited thicknesses, with the exception of the last

layer (subgrade or bedrock) considered infinite in depth;

- shear stresses are not developed in the surface of the layered system, that is, only vertical contact pressures are produced by the wheels.

This theory is widely adopted in practice and is the basis of several computer programs used in analytical pavement design for the determination of stresses, strains and displacements in the layered systems. However, in the case of flexible composite pavements the principal limitation is the assumption of infinite layers in extent, as this does not correspond to the reality of discontinuities that occur in hydraulically bound layers at cracks and construction joints. The modelling of such cracks is complex and requires the use of finite element programs to model the discontinuities physically and geometrically. For the purpose of this chapter, however, the use of a program based on Burmister's theory is believed to be sufficiently accurate as long as the practical issues of material behaviour are also considered. These issues are discussed below.

Design traffic

The real traffic loads are highly variable due to the diversity of vehicles in circulation. For design purposes, only commercial vehicles over 15 kN unloaded weight are considered and they are converted into a number of standard axles causing equivalent damage to the pavement. The common equivalence factor (EF_w) considered is of the fourth power type:

$$EF_w = \left(\frac{W}{W_s} \right)^4 \qquad 9.1$$

where: W axle load of commercial vehicle;

W_s standard axle load (80 kN in the UK).

The power function value equal to 4 in the equation above is not valid for flexible composite pavements. For these, values ranging from 11 to 33 have been suggested by the Permanent International Association of Road Congresses [PIARC, 1987]. A comparison of results using a power function value equal to 4 and 15 is shown in Table 9.1. It is evident from those results that the damage caused by overloads is much greater in flexible composite than in flexible pavements. Hence, for the fatigue design of hydraulically bound layers it is important to have a knowledge of the spectrum of axle loads expected, from which the heavier loads will have a predominant damaging effect, whereas the effect of light vehicles is comparatively negligible.

Table 9.1 Conversion of commercial vehicles into standard axles.

Axle load of commercial vehicles	**Equivalence Factor (exponent = 4)**	**Equivalence Factor (exponent = 15)**
40	0.06	3×10^{-5}
60	0.3	0.01
80	1.0	1.0
100	2.4	28.4
115	4.3	231.3

Horizontal tensile stress criterion in layers with hydraulic binders

The existence of primary cracking in a pavement incorporating hydraulically bound layers is often unavoidable. This leads to the formation of slabs at the edges of which the stress conditions are not well predicted by direct use of the elastic layer theory in which the layers are considered as a continuum. To account for the presence of these discontinuities, the stresses calculated have to be increased, and for this purpose the following factors have been proposed in the literature:

- 1.25 [Brown, 1979];
- 1.30 [LCPC, 1988];
- 1.1 to 1.4 [SARB, 1992].

To determine the allowable stresses in these types of materials, with which the computed values must be compared, the fatigue properties have to be taken into account in order to simulate the repeated action of traffic loading. Fatigue laws of the logarithmic and exponential type have been used [PIARC, 1987]. Here the following equation is adopted:

$$\frac{\sigma_N}{\sigma_0} = 1 - 0.067 \log_{10} N \qquad 9.2$$

Hence, the allowable horizontal tensile stress becomes

$$\sigma_t = \sigma_0 \, (1 - 0.067 \log_{10} N) \qquad 9.3$$

Which gives a value of 531 kPa for the pavement of Figure 9.6.

Vertical compressive strain criterion in the subgrade

In a similar way, to estimate the vertical compressive strains in the subgrade at the crack locations a factor of 2.50 has been suggested and the allowable strains considered to be 75 % of those appropriate for flexible pavements [Brown, 1979].

The University of Nottingham design method proposes the following equations for the permanent deformation criterion [Brown & Brunton, 1988]:

- for life to critical conditions

$$\epsilon_z = \frac{1.04 \times 10^{-2}}{(N / f_r)^{0.27}} \qquad 9.4$$

- for life to failure

$$\epsilon_z = \frac{2.16 \times 10^{-2}}{(N / f_r)^{0.28}} \qquad 9.5$$

where: ϵ_z allowable vertical compressive strain;

N number of cycles;

f_r rut factor (equal to 1 for hot rolled asphalt).

The calculations conducted have shown that the use of this criterion associated with the increase of strains due to the existence of discontinuities, produce unreasonable thicknesses when compared with the ones obtained using the DoT approach, which are known from experience to perform satisfactorily.

The allowable subgrade strain given by the Shell criterion [Shell, 1978] is considerably higher than that obtained from the Nottingham method. That criterion is defined by the following equation:

$$\epsilon_z = \frac{2.8 \times 10^{-2}}{N^{0.25}} \tag{9.6}$$

Based on the experience gained with the use of the Shell pavement design manual over the years, practical guidelines for an adequate choice of safety margins have been suggested [Gerritsen & Koole, 1987]. Hence, and based on a statistical analysis of the original AASHO Road Test results the following equations were proposed, which correspond to confidence levels of 85 and 95 % respectively:

$$\epsilon_z = \frac{2.1 \times 10^{-2}}{N^{0.25}} \tag{9.7}$$

$$\epsilon_z = \frac{1.8 \times 10^{-2}}{N^{0.25}} \tag{9.8}$$

Accordingly, for the analysis conducted it was decided to use a mean value between the strains obtained through the Nottingham method and that proposed by Shell for the 95

% confidence level. In the Nottingham method a rut factor equal to 1 was used, corresponding to a hot rolled asphalt in a flexible pavement. For flexible composite pavements this value may be conservative due to the reduced permanent deformations in the hydraulically bound layers, but alternative values are not available. This approach led to a permissible ϵ_z value of 209 $\mu\epsilon$.

Structural analysis

The analysis following the analytical approach was conducted using the pavement designed in accordance with the Design Manual for Roads and Bridges, shown in Figure 9.5, but in which roadbase and sub-base were considered as a unique structural layer made of the type of materials studied in previous chapters. This layer is assumed to be constructed on top of a capping layer for which different solutions and materials may be adopted:

- unbound materials occurring either locally or imported from other sources;
- *in situ* stabilisation of the existing subgrade;
- stabilised materials from other sources.

The elastic layered system computer program ELSYM5 was used for the analysis [FHWA, 1985]. The pavement section is shown schematically in Figure 9.6 together with mechanical properties for each layer. Three types of capping layer were considered:

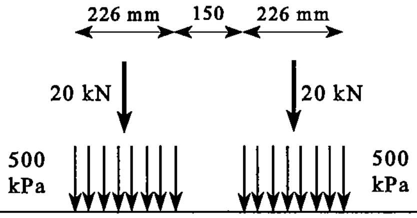
- unbound material (resilient modulus = 100 MPa);
- stabilised material (resilient modulus = 750 MPa);
- stabilised material (resilient modulus = 2000 MPa).

For the design load, a standard axle of 80 kN was considered. The geometry arrangement corresponds to the one used in the Nottingham design method. The material considered for the structural layer does not correspond exactly to any of the mixtures studied in Chapter 8, but is very close to the ones obtained with the china clay sand family. For the subgrade, the resilient modulus was estimated from the CBR value considered in the DoT approach, using the equation developed at the Transport Research Laboratory [Powell, Potter, Mayhew & Nunn, 1984]:

$$M_r = 17.6 \times CBR^{0.64}$$

where: M_r resilient modulus in MPa; and

CBR California Bearing Ratio in %.



	Thickness (mm)	Poisson's Ratio	Stiffness Modulus (MPa)	Tensile Strength (kPa)
Surfacing	130	0.35	3,000	
Structural Layer	250 300 350	0.15	15,000	1,000
Capping Layer	330	0.35	100 750 2,000	0 0 200
Subgrade		0.40	40	

Figure 9.6 Data for the analytical approach to pavement design

The design of the structural layer was carried out to resist fatigue due to traffic loading. Neither the time necessary for crack propagation, which corresponds to an underestimation of the life of the pavement, nor the influence of thermal stresses, overestimating the life of the pavement were taken into account. The overlying material (130 mm bituminous surfacing) will give considerable protection towards temperature and reduce the effect of thermal stresses.

The results in terms of stresses and strains against thicknesses are presented in Figures 9.7 and 9.8 for both criteria used in design.

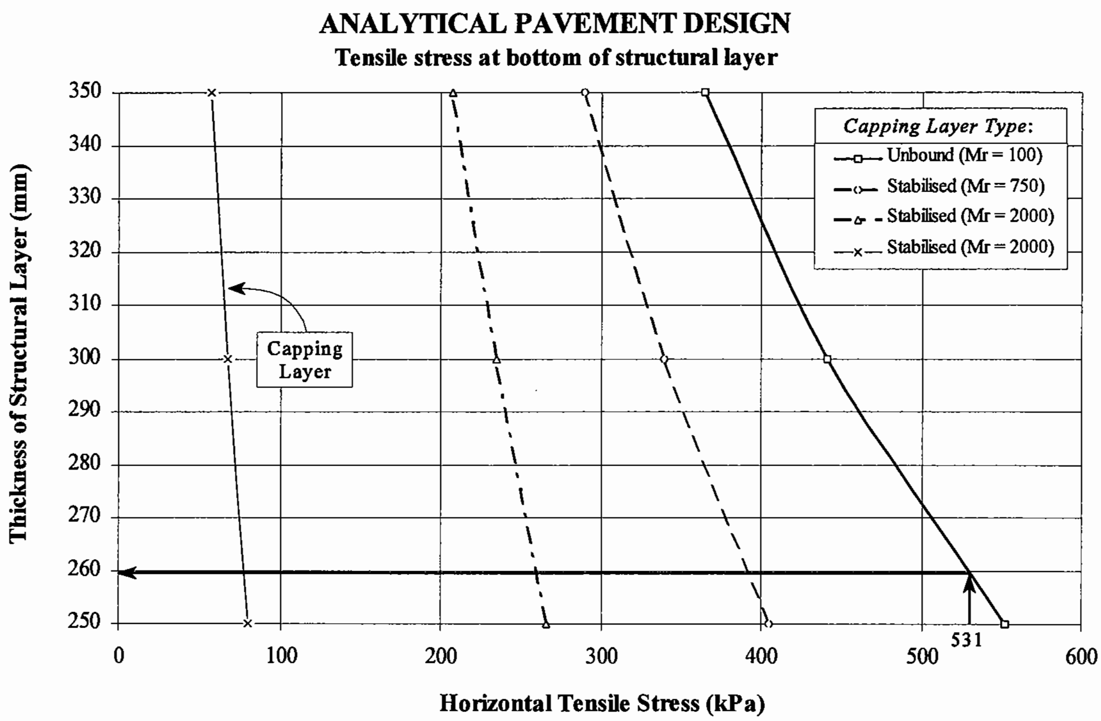


Figure 9.7 Tensile stress criterion.

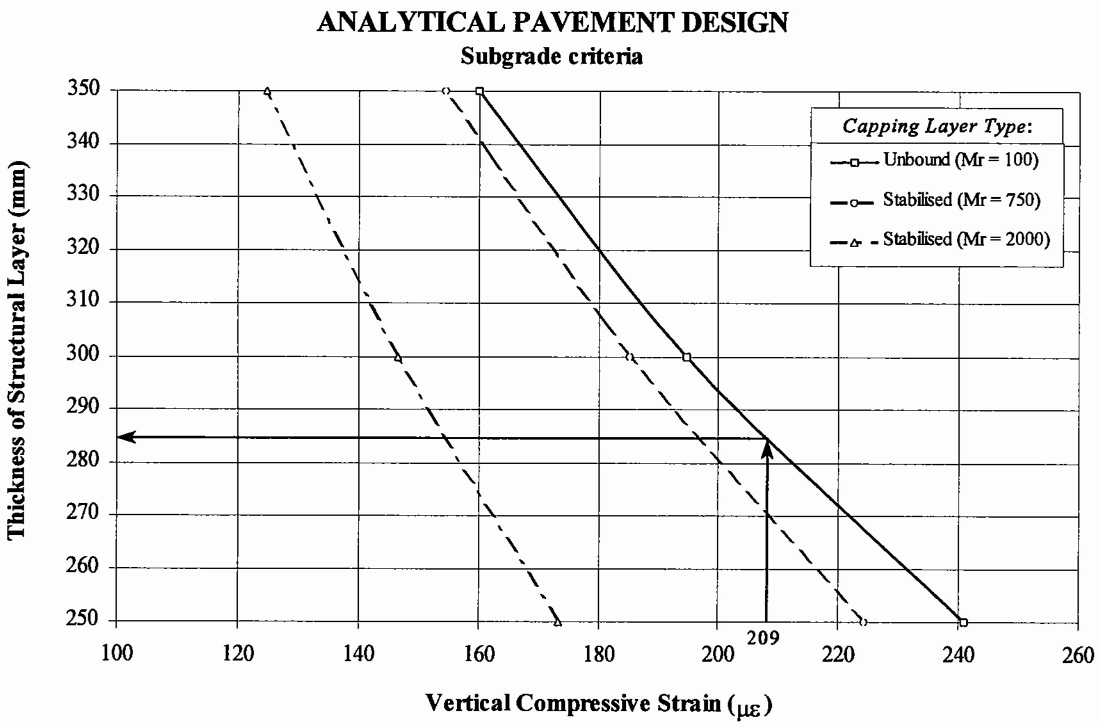


Figure 9.8 Compressive strain criterion.

With the unbound support for the structural layer the thickness required is of 285 mm and is conditioned by the compressive strain criterion in the subgrade (Figure 9.8). This value is in close agreement with the standard structures used in France for similar types of materials, as long as due consideration is given to the difference in standard axle load which is currently 130 kN and compares with the 80 kN used in the UK. It also corresponds to a reduction in the pavement thickness as designed by the DoT manual.

It is clear from the results shown that the use of a stabilised material underlying the structural layer significantly reduces both the tensile stresses in the hydraulically bound material and the vertical compressive strains in the top of the subgrade. The reduction is particularly significant at subgrade level, when the stiffness of the material is increased from 750 to 2000 MPa (Figure 9.8). In addition, the higher the stabilisation, the lower the sensitivity of the stresses and strains to a variation of thickness of the stabilised layer.

The bituminous surfacing above a structural layer with substantially higher stiffness is subjected to compression, assuming total adherence between both layers, and as a result the tensile strain criterion in this layer does not condition its thickness since the question of fatigue cracking does not arise. Nevertheless, an adequate thickness should be used to resist the propagation of reflection cracks to the surface of the pavement.

9.4.5. Discussion

With a small increase in the thickness of the structural layer and using a stronger material, for example similar to mixture M17 in Chapter 8, a reduction of the necessary thickness of capping may be achieved. The resulting thickness would be small enough to permit the use of *in situ* stabilisation to produce the capping, further reducing the strains at subgrade level. Stabilisation *in situ* enables the re-use of materials available on site, avoiding the use of quarried aggregates or the need for borrow pits. As a result, it presents similar benefits to the ones deriving from the use of secondary materials such as protection of the environment, preservation of natural resources and energy savings. Furthermore, stabilised materials present technical advantages in terms of performance, which lead to further savings through the thickness reduction of overlying materials.

The analysis presented in the previous section was carried out using heavily stabilised secondary materials and mechanical properties representative of those introduced in Chapter 8. The design of layers of lightly treated materials does not represent any extra challenge in relation to the design of unbound aggregate layers. In fact, under traffic loading these materials will produce fine, closely spaced cracking because of their low tensile strength. A good aggregate interlock should be maintained and in this manner they would perform as a good quality unbound material.

The concept of a structural layer associating roadbase with sub-base seems to produce reasonable layer thicknesses, benefiting from a reduction in relation to those resulting from the DoT approach based on design charts. The analytical approach considered would enable the use of alternative materials with lower properties when compared with the CBM3 or CBM4 required for cement bound roadbases, and this fact is compensated by a thicker layer gained by incorporating the sub-base in the structural layer. This type of approach has been used in France for several years, where a catalogue of structures has been developed and the thicknesses of the bituminous surfacing and of the underlying hydraulically bound materials is given as a function of traffic and type of foundation [SETRA & LCPC, 1984]. Moreover, the use of these materials in the structural layer would correspond to a high value / high volume application extremely important in making alternative materials competitive with conventional ones and in increasing the current volumes of utilisation.

9.5. Summary

- For both unbound and stabilised materials, the results obtained from index type of tests such as particle strength, CBR and compressive strength were shown not to produce totally reliable information on material performance, giving rise to a different ranking of materials than that obtained with more fundamental tests. It emphasises the importance of using adequate performance related tests for the characterisation of pavement materials.

- The behaviour in flexible composite pavements of the materials stabilised with hydraulic binders was outlined, with special emphasis on the types of cracking that may occur under traffic.

- Using the Design Manual for Roads and Bridges a typical flexible composite pavement was designed. This structure was further analysed using mechanistic methods based on elastic layer theory, but modified in order to account for the existence of discontinuities. Additionally, the pavement structure was modified through the consideration of a structural layer (replacing both roadbase and sub-base) and its thickness designed for the same traffic and foundation conditions. Three stiffnesses were considered for the underlying material, with the stabilised support demonstrating an important effect in reducing tensile stresses in the structural layer and reducing compressive strains in the subgrade.

- The utilisation of analytical methods of pavement design is very important for the design of roads incorporating alternative materials. For these, a single structural layer (combined roadbase and sub-base) seems to be beneficial, reducing the pavement thickness and increasing the range of materials that may be applied under the surfacing layer.

- The results from the proposed assessment strategy and equipment have been shown to enable an appropriate mechanistic design to take place, with all the consequent advantages.

CHAPTER 10

PROPOSED METHODOLOGY

10.1. Introduction

The selection of materials to be used in a particular project depends on several factors such as availability, costs and technical requirements of the project, particularly the level of the layer in the pavement structure, type of structure and of traffic. Economic considerations tend to dictate the type of material used and the most economic and locally available materials are usually preferred. Nevertheless, their technical suitability has to be demonstrated and the methodology used for their evaluation should be able to determine the adequacy, or otherwise, of any available material (both conventional and alternative).

The methodology proposed for assessing the materials is based on the performance-related tests used in this research and on the developments carried out in terms of testing equipment and techniques. The description of these was already the subject of previous chapters, in particular Chapters 6 and 7. Hence, the objective of this chapter is to outline the overall methodology for the evaluation of secondary materials that may be used on a routine basis hereafter. It may also be used for the characterisation of conventional materials, both unbound or stabilised.

10.2. Proposed performance-based methodology

The methodology proposed is presented schematically in Figure 10.1 and it may be subdivided into three main stages:

- Stage 1 - characterisation of the unbound material;
- Stage 2 - stabilisation / mix design;
- Stage 3 - assessment of mechanical properties.

Stage 1 consists of a general assessment in the first instance, using conventional characterisation and classification tests such as particle size distribution, plasticity of the fraction finer than 425 μm, water absorption, particle density, magnesium sulphate soundness and compactibility. The purposes of these tests are to determine properties that are needed for subsequent stages such as optimum moisture content and maximum dry density needed for the compaction of specimens. In addition, other tests may be conducted, as the ones referred above, which provide useful information about the material, their potential for unbound use or the need for mechanical or chemical stabilisation, the most adequate type of binder and durability issues. Note that it is not possible with this methodology to determine the binder type, which is assumed to have been chosen in advance. Although a first estimate between cement and lime may be based on the guidelines presented in Chapter 4 (Section 4.2.1), for the use of secondary binders or combination of binders further research is necessary to develop a methodology taking into account the chemical reactions produced during stabilisation.

After the general assessment, the material is tested in an unbound form using repeated load triaxial tests (RLTT) to determine both resilient and permanent deformation characteristics, that is, resilient modulus (Mr), Poisson's ratio and permanent deformations. Results obtained from this test are analysed in order to assess whether the material has an acceptable behaviour to be used in unbound layers of the pavement. Values that could be expected from a conventional unbound crushed rock granular material may be taken as a reference to determine the adequacy of the material when unbound. It may also be classified using the classification system proposed in Chapter 6 and the target class chosen according to the requirements of the project.

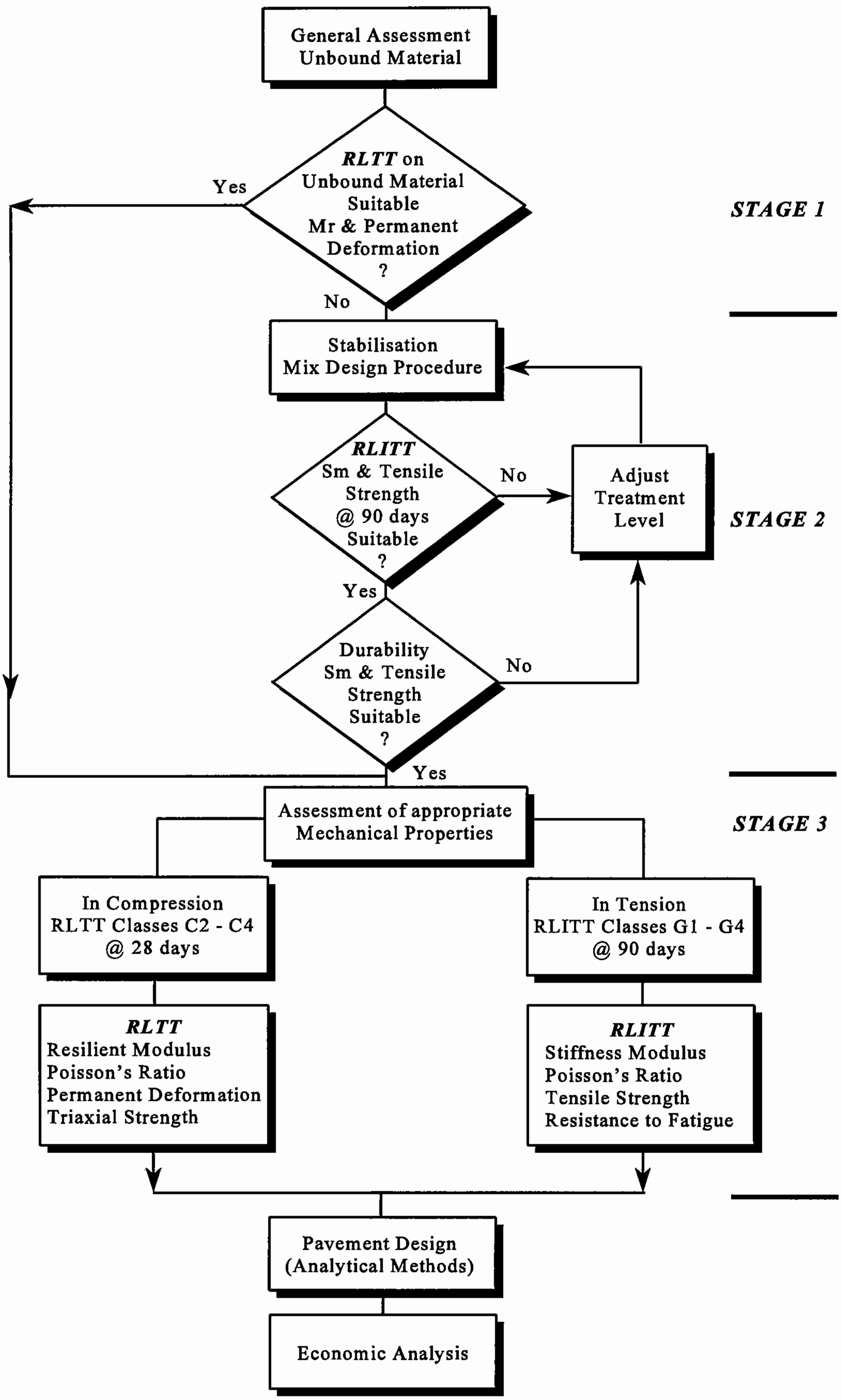


Figure 10.1 Methodology proposed for the study of unbound and treated secondary materials.

The characterisation using triaxial tests may be omitted according to the results obtained in the general assessment, when previous experience indicates that the material is not suitable for unbound applications or when the requirements imposed by the application inhibit its use.

When the unbound material does not perform satisfactorily it is stabilised either with traditional or secondary binders. This represents *Stage 2* of the methodology which makes use of the repeated load indirect tensile test (RLITT) for the mix design procedure. This is based on the determination of the following properties: stiffness modulus (Sm), indirect tensile strength and durability. The tests are performed after 90 days curing in accordance with the procedures followed in this research. It is believed that a 90-day characterisation will be able to reveal the potential of mixtures cementing at slower rates and will permit a more satisfactory comparison of results for different materials.

The level of treatment is adjusted, if required, in order to satisfy the following criteria:

- adequate mechanical properties (stiffness modulus and tensile strength) in the repeated load indirect tensile tests after 90 days of curing;
- appropriate durability of the mixtures, evaluated through the determination of the same properties on immersed and control specimens.

What is adequate will depend again on the specific requirement of the project. Classes of materials and the respective properties are indicated in the classification system proposed in Chapter 8 based on the results of the repeated load indirect tensile test. At this stage, the fatigue characteristics are not determined and so when using Figure 8.24 a fatigue ratio equal to 1 may be considered.

In practice, a number of different mixtures is considered for study, with different levels of treatment and their properties determined. The results of their characterisation permit the drawing of charts in which the properties are plotted as a function of binder content, and then the final percentage of binder chosen to achieve the target properties. The selected material is then considered for *Stage 3* of this methodology.

In order to determine the mechanical properties of the optimised mixture the same main tests / equipments are used, one for assessing the material's behaviour in compression, and the other in tension:

In Compression

- repeated load triaxial test after 28 days curing, to determine resilient and permanent deformation properties. Monotonic triaxial test to evaluate the ultimate triaxial strength is also needed in order to define the stress path used in the repeated load test;

In Tension

- repeated load indirect tensile test after 90 days curing, to assess resilient and fatigue properties, of major importance for increasing levels of treatment, as well as tensile strength using monotonic loading.

An indication of the most appropriate type of characterisation based on the class of the material is presented below:

- Classes C2 to C4 based on triaxial test results - characterisation in compression of unbound type of materials;
- Classes G1 to G4 based on indirect tensile test results - characterisation in tension of stabilised materials.

Class C1 of the first group and Class G5 of the second correspond to intermediate situations and the choice is not pre-defined. In principle, they may be characterised using indirect tensile tests. However, for materials with very low tensile strength or low combined results of stiffness / tensile strength, the determination of fatigue properties is not relevant because in the pavement they will crack frequently at close spacing. This type of material will be better characterised in compression using triaxial testing. This decision may be based on the results obtained during the mix design procedure either with the same mix if this was studied at that stage, or by estimation of the properties through interpolation between mixtures studied.

The characterisation in compression is proposed to be done at 28 days. In principle, the materials characterised in this way are lightly treated corresponding to improved unbound granular materials. The effect over the time is not as significant as for more heavily stabilised materials and for this reason a 28-day period of curing seems to be adequate.

Using the mechanical properties obtained in Stage 3 it is possible to use analytical methods of pavement design, after which an economic analysis of the solution adopted should follow. This is not covered here as it is outside the scope of this research.

10.3. Methodology discussion and advantages

The methodology proposed for the study of secondary materials represents a step forward in relation to the present situation, in which secondary materials are discriminated in relation to primary materials, according to specifications developed essentially with and for conventional materials. The Specification for Highway Works [DoT, 1993] does not go far enough in this subject and despite some 'end-product' specifications introduced, the limits required remain restrictive for the application of this type of material.

More fundamental tests such as the repeated load triaxial test and the repeated load indirect tensile test represent a good compromise in complexity, simulation of field performance and reliability of results. The following advantages may be identified from the implementation of the methodology proposed:

- mechanical characterisation of materials and mixtures using fundamental performance-based tests. Of particular importance is the replacement of the compressive strength test presently used by the repeated load indirect tensile test for the mix design procedure;
- common approach for both conventional and secondary materials, thereby facilitating a fair engineering comparison;
- enables a complete study and characterisation of materials, with the determination of the mechanical properties necessary to conduct structural analysis of pavements;
- as these tests are performed on the end-product and the properties determined used in pavement design, the problem of excessive safety margins in specification requirements is overcome;
- easy discrimination of materials, important when there is a wide range of possible materials to be used, which is of special interest for secondary materials.

It is believed that this procedure is feasible on a routine basis and thus the drafting of specifications based on these laboratory tests would represent an important contribution towards a more widespread use of alternative materials.

CHAPTER 11

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

11.1. Introduction

In this chapter the main conclusions from the research work conducted are outlined. These have been grouped into the following sections: materials, equipment, repeated load triaxial testing, repeated load indirect tensile testing and application. Further research is recommended in several areas covered or related to this investigation and these are also presented.

11.2. Materials

A wide range of alternative materials for road construction have been studied both in an unbound form (6 materials including a conventional crushed granite) and stabilised with various binders (21 mixtures in total). The initial study was conducted following the traditional approach which is based on the Specification for Highway Works. With the exception of unbound slate waste, a material that is processed at source to meet all requirements for sub-base Type 1 aggregate, all the others failed to pass the requirements imposed by the DoT Specification. Minestone and pulverised fuel ash are believed not to be suitable for unbound applications in pavement foundations but all the others demonstrate some potential that is not revealed by the results obtained using the existing specifications. In particular, the strength requirements of both the unbound particles and cement bound materials are too harsh for stabilised secondary materials, aggravated by the characterisation at 7 days that does not take into consideration slower rates of strength gain. Consequently, performance-based tests were considered for their evaluation and the development of testing equipment and techniques was pursued.

The tests chosen for the characterisation of the material were repeated load triaxial and indirect tensile tests. Based on these tests, a methodology for the evaluation of secondary materials was proposed, which includes the following three stages: characterisation of the unbound material, stabilisation / mix design and assessment of relevant mechanical properties in compression or in tension. This methodology presents several advantages in relation to the current practice, namely:

- characterisation based on performance-based tests;
- determination of mechanical properties necessary for using analytical methods of pavement design.

Recommendations for future research

- Further work is recommended considering the same materials but supplied from alternative sources, to study the effect of the variability of these materials on the mechanical properties of the mixtures. This issue is important for the quality assurance and certification that has to be provided by producers to generate more confidence in secondary materials.
- The programme of testing should be extended, to other alternative materials, or combinations of materials, not considered in this study. This is important to establish a comprehensive knowledge on alternative materials that will contribute to a more widespread use in road construction when available to producers, engineers and the industry in general.
- Further research is also recommended into the extension of the methodology developed to incorporate the selection of the most appropriate binder type, according to the type of soil or aggregate to be stabilised.

- Performance-based specifications in the style of the latest CEN standards should be drafted, incorporating laboratory characterisation of the mixtures using stiffness modulus, tensile strength and fatigue characteristics as proposed in Chapter 7.

11.3. Equipment

Improvements were made in the repeated load triaxial apparatus, namely the modification of the location studs and the development of a new load cell, which is less susceptible to electrical noise and has the capacity to perform small rotations that result in the application of more uniform axial stresses.

A repeated load indirect tensile apparatus was developed, using a servo-hydraulic loading system shared with the triaxial rig. A testing sub-frame was specially designed to incorporate 150 mm specimens which may be used in other testing rigs. It contains vertical guide rods and an 'internal' load cell for more adequate application and measurement of the load transmitted to the specimens. The LVDT frame traditionally used for measuring the horizontal deformations was found to be adequate for hydraulically bound materials and an alternative frame, supported at single 'points' at the centre of the specimen, produced similar results. The results obtained with this apparatus seem to give results with adequate repeatability.

Compaction techniques and the necessary equipment were developed for the compaction of both triaxial and indirect tensile specimens. The former combines the action of a vertically-guided vibrating hammer with the one of a vibrating table, while the latter uses the standard compactibility test rig and a specially designed mould to compact the specimens in one layer, in their final form.

Recommendations for future research

- Development of compaction equipment / techniques representing more closely the compaction carried out in the field is recommended. Methods based on vibro-compression techniques have been developed and used in France which are reported to produce specimens with highly homogeneous densities [Chi, 1976; NF P 98-230, 1992]. Although they do not closely simulate compaction under vibrating rollers *in situ*, they represent an improvement in relation to current practices and should be taken into account for the development of new procedures.

- Repeatability and reproducibility studies with the repeated load indirect tensile apparatus need to be extended and conducted respectively, to permit the incorporation in the national standards of the methods of testing developed during this research.

11.4. Repeated load triaxial testing

The test techniques for preparation of specimens, conditioning, resilient and permanent deformation testing were presented which required some modifications in relation to the normally recommended procedures for this type of testing. It was found that the stress path used for testing has to be chosen with reference to the failure line of the material, limited to 70 % of failure values.

In terms of mechanical properties two groups of materials were identified. The first comprised unbound materials and other materials that, despite being stabilised, give the same type of results. Hence, the first group of materials was characterised by its 'unbound type of behaviour'. On the other hand, the second group comprised materials with higher resilient moduli and triaxial strengths, lower Poisson's ratio and reduced non-linearity, hysteresis and stress-dependency. This group was characterised by the materials' 'treated type of behaviour'. The definition of these groups has implications in the modelling of materials and in the identification of resilient models able to produce the best results in each group.

Stabilisation was found to improve triaxial strength, resilient modulus and reduce the accumulation of permanent deformations. The degree in which those properties improved varied from material to material, the type of binder used having an important role.

Resilient models available were thoroughly reviewed and discussed. Modelling using Mr-ν models, such as the K-θ and Uzan models, was generally found to be poor. The hypotheses considered for study were Pappin's model, Mayhew's model and modified versions of Mayhew's model incorporating the parameter p^* and stress path length dependency. These were found to model adequately materials with 'unbound type of behaviour' but their performance declined for the 'treated type of behaviour'. A new mathematical model was developed which produced consistently better performance for that group of materials. Boundaries for those two groups and of validity for the corresponding resilient models were proposed.

The behaviour in terms of permanent deformation was studied using the French model. With regards to the equation representing the influence of the number of cycles, it models well the results obtained with unbound materials. On the other hand, the effect of the stress level is not of the same quality and the simplified approach using data from failure lines is not recommended. For treated materials the performance of that model decreased due to non-asymptotic accumulation of permanent deformations which is not well reproduced by that model. Nonetheless, it is believed that the development of a specific model for this type of material is not necessary because the permanent deformations obtained experimentally were very small and so internal material deformation will not be a significant contributor to overall permanent deformation of a pavement structure.

A new classification system was proposed, which is based on characteristic resilient modulus and corrected permanent axial strain after 80000 cycles. The chart represents an important tool for understanding the significance of the experimental results.

Recommendations for future research

- It is recommended that the model developed for the resilient behaviour of lightly treated secondary materials be used to analyse experimental results obtained with other stabilised conventional and secondary materials. In the study carried out, the fitting capability of the shear model was good for all materials, including the unbound ones. Further studies may be conducted with the volumetric model to extend its applicability to unbound and unbound type of materials.

- The current state-of-the-art with regards to permanent deformation behaviour of unbound materials under repeated loading remains essentially at an empirical level. Research is recommended in this domain for the development of mathematical models capable of describing more adequately the laboratory response and applying it to *in situ* behaviour.

11.5. Repeated load indirect tensile testing

At present, this form of testing is not covered by standards with the exception of the determination of the indirect tensile strength. This represents a lacuna in the specifications that the work presented here may help to overcome since this form of testing has considerable potential for the study of stabilised secondary materials. The techniques for specimen preparation, testing and analysis of results were described in detail. These are proposed for the assessment of all relevant properties of hydraulically bound materials: tensile strength, stiffness modulus, Poisson's ratio, resistance to fatigue, durability and the development of properties with time.

For the determination of Poisson's ratio the use of on-sample instrumentation positioned in the central part of the specimen is very important enabling a more accurate measurement of vertical and horizontal deformations in an area where the stresses are approximately constant. On the other hand, the traditional LVDTs frame currently used for bituminous materials was found to be an acceptable alternative for stiffness quantification purposes, considerably reducing testing times.

Durability assessment based on tensile strength and stiffness determinations was found to give adequate discrimination of materials and the consideration of two periods of immersion to increase qualitatively the information about the materials was proposed.

The effect of frequency on the fatigue properties was found to be more important for large variations in the frequency of loading. For small changes the effect is negligible. In addition, it has been shown that higher binder contents produce materials less susceptible to fatigue for the same stress levels.

Good agreement was found between stiffness results obtained using triaxial and indirect tensile testing under repeated loading, thus contributing to the validation of equipment and techniques developed.

An application of the testing equipment and techniques developed was carried out on thirteen mixes from which six were selected for full-scale trials. In general terms, the best performing 'aggregates' were found to be conventional aggregate and pulverised fuel ash. China clay sand also performed well, in a similar way to a cement stabilised granite. The most successful binders were fly ash with cement or lime and the combination fly ash / lime / gypsum.

Empirical relationships between the mechanical properties of the materials and the curing time, or between compressive and tensile strength were found not to be totally reliable. It is important to actually perform the necessary laboratory tests and assess a particular material being studied.

Based on the existing CEN proposal, a new classification system was developed. It uses the following results obtained through repeated load indirect tensile tests: stiffness modulus, tensile strength and fatigue properties all obtained at 90 days curing. By incorporating fatigue and characterisation at 90 days, instead of 360 days, the chart is believed to be a more complete and practical classification system in comparison with existing proposals.

Recommendations for future research

- The effect of the moisture content should be more closely studied in order to optimise the mixtures with the partially conflicting requirements for optimum density and for full hydration of the binder. This may be investigated by studying the mixtures compacted using a range of moisture contents, for example using the optimum moisture content obtained in compactibility tests (OMC) and OMC ± 2 %.

- The effect of the maximum particle size of the aggregate on the compaction of indirect tensile specimens should be studied. This may be conducted by compacting slabs using the same mixtures and coring from them the cylindrical specimens used for testing. The results should be compared with the ones obtained following the compaction method described in Chapter 8, which should provide the necessary information to define the interaction effect between maximum aggregate and specimen sizes.
- The study of fatigue characteristics in the repeated load indirect tensile apparatus should be extended using lower levels of stress, in percentage terms, to cause failure after higher number of load applications (10^6 cycles and above) with the aim of studying the existence of endurance limits that would have to be taken into account in pavement design.
- The results obtained in indirect tensile mode should be validated / calibrated with results obtained in pavements. Initially, this is to be carried out in full-scale trials in the Pavement Test Facility of the Transport Research Laboratory. After this, the instrumentation and testing of site trials should provide the complementary and definite information necessary for the validation and implementation of the results.

11.6. Application

This research was essentially concentrated on laboratory studies of alternative materials. Their actual application in roads and the assessment of both equipment and methods with field results was not carried out. Nevertheless, their use was evaluated using analytical methods of pavement design in which the pavement structure is considered as an elastic layered system. This study has shown the following benefits resulting from the inclusion of layers with stabilised secondary materials:

- the possibility of utilisation of these materials at levels in the pavement structure as high as the roadbase;
- their application in a structural layer, replacing the current roadbase and sub-base, was illustrated and seems to be beneficial thus permitting the reduction of the overall thickness of the pavement and enabling the application of a wider range of materials at this level when compared with the present situation.

Remaining problems are likely to be:

- durability of the mixtures, namely the gypsum-based mixture, which will require a careful application in controlled situations;
- environmental cracking, due to temperature changes. Strong materials will tend to produce a more widely-spaced and wider type of cracking, an aspect that needs careful consideration to control the process effectively;
- reflection cracking is a problem with flexible composite type of pavements. There are a number of construction practices that may be used to stop or delay the propagation of cracks to the surface of the pavement
  - induction of cracks in a controlled manner and with appropriate spacing between cracks to limit their width and maintain adequate levels of aggregate interlock and load transfer capabilities;
  - use of geotextiles or geogrids between hydraulically-bound and the overlying bituminous materials;
  - construction of a layer with selected materials separating those two layers.

11.7. Final remarks

The present research work aimed to technically contribute to the use of alternative materials in road construction. It is believed that the principal objectives were accomplished through the development of methodologies for their evaluation, testing equipment and the drafting of testing techniques specific for this type of materials. This was complemented by the development of a new mathematical model and of classification systems that should provide engineers with the necessary tools to deal with most applications of alternative materials in pavements. Some evidence that seems to confirm the achievement of those objectives are:

- the utilisation of the same repeated load indirect tensile apparatus and testing procedures in subsequent research works [Cheema, 1996], including the ongoing 'LINK II' research project carried out by the Transport Research Laboratory with the University of Nottingham;
- the interest manifested by the Department of Transport, secondary materials' producers and consulting engineers in the equipment and testing methods here developed. These are also a positive indication of their possible incorporation in national and international standards.

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Abbreviations

ARRB	Australian Road Research Board
ASTM	American Society for Testing and Materials
BLA	British Lime Association
BRE	Building Research Establishment
BS	British Standard
CEN	Comité Européen de Normalisation (European Committee for Standardisation)
CIRIA	Construction Industry Research and Information Association
CRR	Centre de Recherches Routières (Centre for Road Research)
DoE	Department of the Environment (UK)
DoT	Department of Transport (UK)
FHWA	Federal Highway Administration
FT	Financial Times
HMSO	Her Majesty's Stationery Office
LCPC	Laboratoire Central des Ponts et Chaussées (Central Laboratory for Bridges and Roads)
LNEC	Laboratório Nacional de Engenharia Civil (National Laboratory of Civil Engineering)
NF	Norme Française (French Standard)
No.	Number
NP	Norma Portuguesa (Portuguese Standard)
OECD	Organisation for Economic Co-operation and Development
PIARC	Permanent International Association of Road Congresses
RRL	Road Research Laboratory
SARB	South African Roads Board
SETRA	Service d'Études Techniques des Routes et Autoroutes (Service of Technical Studies for Roads and Highways)
SHRP	Strategic Highway Research Program
SPRINT	European Community Programme for Innovation and Technology Transfer
SWK PE	SWK Pavement Engineering Limited
TRL	Transport Research Laboratory
TRRL	Transport and Road Research Laboratory
TRB	Transportation Research Board
Vol.	Volume
pp.	Pages

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