UNIVERSITY OF NOTTINGHAM
DEPARTMENT OF CIVIL ENGINEERING

REINFORCED ASPHALT OVERLAYS FOR PAVEMENTS

By

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To my family.

Thank you for the sacrifices you have made to give me the freedom to undertake this work.

ABSTRACT

The maintenance of road pavements in England has become a costly necessity, due largely to the large volume of commercial vehicles using the roads which cause pavements to deteriorate quickly, and makes their repair more difficult to carry out. These roadworks incur not only direct works costs, but also indirect costs from factors such as congestion, motor accidents and pollution. There is obviously a need for cost-effective maintenance that minimises the occurrence and duration of these disruptions.

To strengthen pavements bituminous overlays are often used, but may crack prematurely when placed over a layer with discontinuities such as cracks or joints, or deform excessively under wheel loading. The problem of 'reflective cracking' is widespread and reduces the life of maintenance treatments considerably. To increase the time before cracking appears on the surface of a pavement, a (more expensive) thicker overlay may be used, but this can lead to problems with property thresholds and bridge clearance. One possible option of reducing the thickness of overlays by making them more resistant against cracking and deformation, is to place a layer of reinforcement within or at the bottom of the overlay. Although this approach has been used occasionally to reinforce overlays, over 40 years or so, it is not favoured with many road authorities, as the results of these treatments are difficult to anticipate, and may not be cost effective.

This thesis describes an investigation into the effect of reinforcing thin bituminous overlays to identify key factors that significantly influence their performance. By identifying these factors, optimum use of reinforced asphalt should be possible, and thus maintenance of the road network made more cost effective.

The investigation was principally carried out in the laboratory using beam tests, shear box tests, tensile tests on reinforcement and large-scale wheel tracking tests. 2-D Finite Element Analysis was used in the analysis of test results.

Results show that properly constructed reinforced overlays can be between two or three times more resistant to cracking, and have less than half the permanent deformation of unreinforced materials.

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DECLARATION

The laboratory work described in this thesis was conducted at the University of Nottingham Department of Civil Engineering between December 1995 and April 2000. Subsequent to the laboratory work, much of the analysis has been carried out at my home in Berkshire.

I declare that this work is my own and has not been submitted for a degree at another university.

Paul Sanders P Sanders November 2001

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CHAPTER 1

INTRODUCTION

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1 INTRODUCTION

1.1 General

For an economy to be successful and efficient, a freely flowing transportation network is important. In Britain, over the past half century or so, industry has increasingly used the road network to fulfil this function, with rail and waterways becoming less well-used. However, as traffic levels continue to rise, the road network is becoming less able to fulfil the needs of the economy, which in turn leads to (*inter alia*) more expensive goods and services and environmental damage.

To relieve the general problem of traffic congestion, the historical approach has been to build new roads. However, this is now becoming less desirable, less economic and less environmentally acceptable in Britain and most other developed countries. Unless traffic levels are reduced, therefore, it follows that road congestion will not be relieved and will probably increase as the road network increased travel times, traffic delays, increased pollution and enforced lower speed limits (for safety reasons).

From the above, it is appears that maintenance measures to arrest or delay road deterioration are required and should be quick to implement and long-lasting. These help to reduce traffic congestion by both keeping the works period as short as possible and by increasing the period between maintenance treatments. An added incentive for more effective (long-term) maintenance is the lengthening back-log of road maintenance as a result of a reduction in funding in recent years [1.1, 1.2, 1.3, 1.4, 1.5, 1.6]. This reduction of maintenance budgets has led to some lengths of pavement requiring a structural treatment only receiving a superficial treatment, to ensure safety is not compromised. The effect of postponing structural maintenance in the short term is often an increase in the cost required to bring and maintain the road network to an acceptable standard in the longer term. This problem has been recognised by the UK Highways Agency (HA) who are now committed to evaluation of highway construction and maintenance in terms of Whole Life Costs [1.7], an approach that facilitates comparison of different construction and maintenance options. The need for quick and effective maintenance treatments has also been emphasised with the advent of long-term Design, Build, Finance and Operate (DBFO) contracts, where efficient maintenance strategies can make the difference between success and failure.

At present the most commonly used maintenance treatments include

a) surface dressings
b) thin wearing courses
c) inlays,
d) resurfacing,
e) overlays
f) partial or full reconstruction.

In terms of their classification, a) and b) are not considered structural maintenance treatments, c) and d) may be considered as enhancing the pavement structure, but e) and f) increase the structural capacity of a pavement.

Where a pavement structure requires strengthening, options to reduce overlay thicknesses are desirable for protection of the environment, and economy, i.e. to reduce the quarrying of aggregates, and to provide a pavement with adequate performance at reduced cost. One approach to achieve this, that has been used to a limited degree over the past 50 years or so, is asphalt 'reinforcement[1]' i.e. the inclusion of interlayer materials placed typically between an existing pavement and the overlay. As described in Chapters 2 and 3 (the Desk Study), a range of materials are commercially available that reputedly reduce rutting and/or cracking. These include grids, fabrics and composites (having elements of both grids and fabrics), which may comprise plastic, glass or steel. As will be later seen, the option of asphalt reinforcement is not a straightforward option, and the significantly different nature of some of these materials (produced to combat the 'same' defect or defects) is indicative of problems in (i) characterising the nature and causes of cracking, and (ii) providing a solution to the problems.

Historically, limited use of grids and fabrics has been made if compared to other approaches such as partial reconstruction or thicker overlays. This is so for a number of reasons that have contributed to the general lack of confidence in their abilities. It follows then that highway authorities are understandably reluctant to give approval to maintenance treatments that have a relatively short (if any) track record in the UK. This in turn makes it more difficult for performance data to be accumulated. Accordingly, maintenance treatments using grids or fabrics are more often found on county roads than on trunk roads, whereas other more conventional solutions such as thicker overlays and bituminous mixtures incorporating modified binders, for instance, are usually adopted in preference on the trunk road and motorway network. Another reason for the lack in confidence in using reinforced asphalt stems from reluctance to use it on the part of contractors, who, during construction, may encounter difficulties if they are not experienced in laying grids and fabrics.

A brief investigation of the market relevance of the project shows that the current UK use of asphalt pavement interlayers (grids and fabrics) has an annual value of around 2.5 million pounds. To be more meaningful, however, this figure needs to be considered with savings made due to reduced overlay thicknesses, or treatments resulting in fewer interventions in the future. When considered in the light of the budget for structural maintenance of the Motorway and Trunk Road network of approximately 250 million pounds, it is understandable that it is still seen as a small market niche by manufacturers. However, the market is likely to grow substantially as the need for more cost-effective maintenance and alternatives to pavement reconstruction increases.

[1] Note that for the purposes of this document, the term 'reinforced asphalt' refers to asphalt layers that include grids and fabrics, and not fibre-reinforced materials.

1.2 Pavement Failure Mechanisms

1.2.1 General

It follows that before effective pavement treatments can be devised and evaluated, an understanding of the way in which pavements fail is required. For brevity, the following discussion is restricted to the main modes of failure of bituminous surfaced roads, which include fully flexible pavements, flexible composite pavements and overlaid rigid pavements. These structures are shown in Figure 1.1.

Typically, pavements 'fail' in serviceability by developing poor riding quality (manifested by driver discomfort and measured by longitudinal or transverse unevenness), or becoming unsafe, particularly through reduced skidding resistance. The deterioration of riding quality of a pavement is measured as unevenness of the surface which may be due to permanent deformation of bound materials or differential settlement of supporting layers. An unsafe pavement on the other hand is normally one with poor skidding resistance. This occurs when the pavement surface is made smooth by the passage of traffic, an excess of bitumen in the surface (bleeding), or standing water. Apart from an obvious design or construction fault with surface levels, ponding of surface water occurs in ruts, or is due to the settlement of the pavement support.

Apart from loss of skid resistance the two most common symptoms of 'failure' of bituminous-surfaced roads are rutting and cracking, and these are discussed in more detail below. As a general comment it is noted that 'failure' of pavements relates almost always to that of serviceability and not of 'destruction' as might be the case with other engineering structures.

Cracking affects pavements detrimentally in various ways. Initially, layer strength is lost which leads to overstressing of lower layers, consolidation and as a consequence, permanent deformation. In addition to the reduction in strength (due to less intact material), cracks provide access for water which softens unbound materials and reduces shear strength.

Cracks and permanent deformation are normally attributed to traffic and/or environmental influences. However, aspects of construction may also help induce problems such as when carriageways are widened. For instance, cracking may occur at the junction of old and new constructions as a result of differential deflections across the vertical interface, which are due to differences in support of the old and new constructions. The principal crack types associated with pavements are now described.

1.2.2 Fatigue Cracking

Fatigue cracking occurs due to repeated applications of tensile strain which eventually overcome the resistance of the material. This phenomenon may be considered as having two phases: (i) initiation and (ii) propagation. Crack initiation can be considered as where the repeated application of tensile strains cause micro-cracks to join and form a macro-crack. The continued application of tensile strains then causes growth and progression of this macro-crack through the material, which is known as the propagation phase.

then causes growth and progression of this macro-crack through the material, which is known as the propagation phase.

Crack initiation has been relatively well investigated and defined through functions that relate tensile strain to number of load repetitions (see References 1.8 and 1.9 for example). Crack propagation on the other hand is less understood and defined and is influenced by factors such as

- the type and amount of bitumen in the mixture,
- the amount and type of aggregate present,
- adhesion of the bituminous binder to the aggregate [1.10]
- the nature of applied load (traffic and environmental).

Some of these factors are difficult to quantify and so present problems in predicting performance.

The three main modes of crack propagation which may be considered are shown in Figure 1.2, and are termed:

Mode I (opening mode)
Mode II (shearing mode) and
Mode III (a 'tearing' mode),

In practice Mode I cracking could be expected at the bottom interface of a bound pavement layer when loaded, and Mode II cracking might be expected in material bridging a crack or joint subject to differential movement. Mode III cracking is perhaps more difficult to visualise but could possibly occur adjacent to wheel loads (longitudinally) in a pavement.

In the classical pavement bending mode, crack initiation is normally expected to occur at the lower interface of a layer, although in thick bituminous layers (typically in excess of 250mm [1.11] as are found on many of Britain's trunk roads and motorways), cracking has often been found as a top-down phenomenon. Also, where pavements have rutted, longitudinal cracks may be found adjacent to the 'shoulders'. These modes of pavement behaviour are illustrated in Figure 1.3.

1.2.3 Reflection Cracking

Mechanisms of reflection cracking are complex and can be due to a combination of the movement of joints or discontinuities in the pavement beneath an overlay, and environmental influences at the pavement surface. In general, the main contributor to vertical movement of cracks or joints is taken to be traffic loading. Horizontal movements are assumed to be caused by differential thermal expansion and contraction of the pavement layers. The magnitude of vertical movements depends on a range of factors including support to the layer being cracked and roughness of the crack faces (interlock). For thermal loading on the other hand, the severity of temperature gradients through the layers, plus the coefficient of thermal expansion/contraction of the different materials are of prime importance. More detailed descriptions of reflective cracking are to be found in proceedings of the 4 RILEM conferences [1.12,1.13,1.14,1.15] and De Bondt [1.16].

Notwithstanding the considerable work carried out to characterise and solve reflective cracking problems, mechanisms still remain somewhat undefined. For instance, reflective cracking occurring in Lane 2 but not on the adjacent Lanes (Lanes 3 and 1) or on the hard-shoulder (that was not trafficked) has been reported. This type of occurrence highlights the complicated nature of predicting pavement performance in the field, even with 'normal' pavement constructions. As described later, the addition of non-asphaltic interlayer materials within asphalt layers serves to give more difficulties in analysis and performance prediction.

1.2.4 Rutting

Rutting occurs as a consequence of deformation of the visco-elastic bituminous materials and/or permanent deformation of materials supporting the bituminous surfacing. An illustration of the stress-strain-time response of bituminous materials is given in Figure 1.4. The permanent deformation of bituminous material is due to viscous flow of the bitumen, which in turn is a function of loading time, stress level and temperature. With repeated loading, permanent strains accumulate and manifest as surface deformation, recognised typically through the appearance of raised shoulders. Deformation of the whole pavement structure does not produce shoulders. The two different mechanisms are illustrated in Figure 1.5.

To help prevent 'excessive' rutting, analytical flexible pavement design generally uses relationships between traffic-induced vertical strains at the top of the subgrade, and the accumulation of permanent deformation to determine layer thicknesses [1.8 and 1.9]. No deformation within the bituminous material is therefore explicitly taken into account. The permanent deformation of materials supporting the bound layers is due to excess stress being transmitted through the bound layers and is a deficiency of the overall pavement design.

1.2.5 Field Mechanisms

The mechanisms by which rutting and cracking occur in the field are now summarised.

Flexible pavements may fail due to excess permanent deformation, cracking or a combination of both. Crack formation often begins in wheel tracks, and as the pavement becomes progressively trafficked, spreads irregularly over the surface. Eventually, the surface may be covered in a lattice of interlocking longitudinal and transverse cracks often termed 'alligator cracking'.

Failure of **flexible composite pavements** typically occurs when cracks in the supporting cement bound (CBM) roadbase are 'mirrored' on the surface or reflect through to the surface. At this stage, the mode of failure is typified by quite regularly-spaced transverse cracks. Cracking of the CBM occurs through initial shrinkage of the material (and is therefore dependant on the cement content used), and due to daily and seasonal changes in temperature. The changes in temperature induce contraction and expansion in materials which largely depends on the type of aggregate used.

With prolonged or heavy trafficking, flexible composite pavements may also develop a network of irregular cracks as the CBM gradually breaks into smaller pieces which reflect cracks into the surfacing. It is therefore possible to estimate the degree of distress of a CBM by the nature and severity of surface cracking.

The structural strength of the CBM usually precludes the formation of 'whole structure' rutting, so permanent deformation observed on the surface is likely to be due to permanent deformation of the bituminous material only.

Overlaid rigid pavements typically fail through reflective cracking, as slabs tend to move or 'rock' under traffic loads due to support under the ends of the slabs weakening. Surface cracks are then opened up over joints which can allow the ingress of water. If this occurs, further loading can force water and fine material out of the cracks which is termed 'pumping'. Pavement failure can then become very much quicker as support is progressively reduced and rocking is intensified, which opens cracks and gives access to more water, thus making the situation worse. With reduced support, concrete slabs may crack irregularly, especially close to corners and edges, which can often be seen as diagonal cracks.

1.3 Maintenance treatments

It follows therefore that any economic maintenance treatment should result in the number of load repetitions required to initiate and subsequently propagate cracks being increased, and the rate of permanent deformation reduced. Where overlays are to be used there are several possible approaches to enhance their performance. These include increasing the thickness of bituminous material, placing a Stress Absorbing Membrane Interlayer (SAMI), modifying the properties of the overlay (by addition of a polymer perhaps), or adding a reinforcing interlayer between the surfacing and the cracked/jointed layer.

The options are now briefly described.

An **increase in the thickness of bituminous material** has two main functions, namely, to reduce the strain on the lower interface and increase the time of crack propagation due to the increased distance from the point of crack initiation to the surface. This has been well-used historically but can be expensive, and is not compatible with the increasing environmental concern. Also, in situations where no further increase in levels can be allowed, this option is not feasible.

A **SAMI** is a relatively soft layer placed between the old pavement and the new construction. The function of a SAMI is to reduce stresses generated by movement in the cracked pavement to a level that can be accommodated by the overlay. A SAMI is typically 4-8mm thick and often comprises a blend of rubberised bitumen. A summary of the findings of a study into the effectiveness of SAMIs has been given by Mukhtar and Dempsey [1.17] which (*inter alia*) states that SAMIs have been more effective when used with flexible pavements than rigid pavements and that SAMIs perform better the thicker they are and lower stiffness they have. Also, full-width pavement treatment has been more successful than local treatment directly over the joint/crack area.

Modified binders in bituminous mixtures [1.18] have been used for more than a decade to improve resistance to rutting and cracking. Probably the most common way of bitumen modification is with the addition of Styrene Butadiene Styrene (SBS), Styrene Butadiene Rubber (SBR) or Ethylene Vinyl Acetate (EVA) polymers. The addition of these polymers helps to enhance the performance of the bituminous mixture at high and low temperatures, which in general means that mixtures are less susceptible to rutting at high temperatures and cracking at low temperatures.

Design rules that can reliably quantify the subsequent performance of pavements incorporating these materials are however not presently well-developed, and field trials [1.19, 1.20] give a range of effectiveness. Work aimed at quantifying the properties of polymer-modified bitumen and the subsequent effects on mixture performance is reported in Reference 1.21.

The inclusion of **grids, fabrics and composites** within a pavement construction is another option that has been tried in various forms since the 1950s. Although the first use of (steel) reinforced asphalt occurred in the 1950s, it is only in the past 20 years or so that a range of different glass and plastic materials have been used in any significant quantity. Early work carried out at Nottingham [1.22] helped to confirm the potential of reinforcing pavements with grids. Conventional engineering philosophy would normally suggest that there are two main reasons for placing interlayer materials within a pavement. First of all, if the interlayer material is stiffer than the asphalt, it will reinforce the layer (if adequately bonded to the asphalt above and below) by carrying load that would otherwise be carried by the pavement. Otherwise, if the interlayer stiffness is less or of a similar magnitude to asphalt, then, to enhance the properties of the pavement it must provide stress-relief or similar to protect the pavement. This may also include a 'crack-stitching' quality where crack initiation is not prevented, but crack propagation is delayed.

Note that for the remainder of the document, the term 'reinforced asphalt' refers to any sheet, grid or combination of the two, within layers of asphalt.

Although it would appear relatively simple to define which mechanism applies and what contribution it makes to engineering performance, few reliable guidelines exist. In fact, basic questions regarding the type of material to be used as an interlayer, and where it should be placed in a pavement still remain. It also follows that more detailed questions, such as the nature of the bonding (adhesion or interlock) and the effect it has on pavement performance and how to achieve it in the field are also poorly defined.

The main focus of the work described here was to investigate the principal factors affecting the use and performance of reinforced asphalt. Once these factors were defined, it was reasoned, this knowledge would be used to predict pavement performance and make it possible to choose appropriate treatments on the basis of sound engineering principles.

As cracking has been a major contributor to pavement failure in the past, the project is primarily directed towards reducing or solving the problems of cracking. However, as is later seen, the possible role for reinforcement in slowing the development of rutting is also addressed.

It is recognised that the best method of establishing field performance is to monitor the behaviour of full-scale in-service pavements. This of course is not feasible for every experiment due to practical reasons such as the number of test variables which need to be investigated which would result in excessive cost. Accelerated testing of full-scale pavements is another option but would also be expensive even if the apparatus was available. Physical and mathematical modelling is on the other hand affordable and possible with the facilities at hand, and offers some advantages over large-scale testing.

More discussion of the approach chosen to investigate reinforced pavement mechanisms is given in Chapter 4.

1.4 Aims of the Project.

A summary of the aims of the project is given:

- To investigate the use of reinforced asphalt in the UK in particular, and around the world in general.
- To carry out tests on samples of reinforced asphalt to define operating mechanisms.
- To model laboratory test results and to apply these models to field situations
- To summarise findings and produce guidelines for the selection and use of reinforced asphalt.

1.5 Structure of the Thesis

The thesis is structured as follows:

Chapters 2 and 3 comprise a general introduction to reinforced asphalt, how and where it has been used, and with what success. Chapter 4 summarises the results of the desk study and discusses possible approaches to the problem. These range from monitoring in-situ applications of reinforced asphalt under real traffic loading, to laboratory testing of each of the components (asphalt, reinforcement and the bond between them).

Chapters 5 and 6 describe laboratory testing of the components of reinforced asphalt i.e. reinforcement and interlayer bond.

In chapters 7 and 8 the development of test apparatuses used to test reinforced asphalt beams and half-scale reinforced pavements is described. Chapter 7 gives details on beam tests, and Chapter 8 describes wheel tracking tests.

The numerical modelling of reinforced beam and pavement structures using Finite Element Analysis (FEA) is described in Chapter 9. The potential benefits in terms of reducing crack propagation is illustrated.

Economic appraisal of reinforced asphalt using the Whole Life Costing (WLC) approach is given in Chapter 10. In particular, the appraisal shows that careful consideration of the field situation is required before reinforced asphalt will be an economic solution to rutting or cracking.

Chapter 11 provides guidelines for the use of reinforced asphalt derived from both work carried out in this project, and from results of full-scale trials described in the literature.

Chapter 12 gives an overall summary and conclusions of the project, and Chapter 13 gives suggestions for future work to be carried out to investigate some of the 'unknowns' discovered during the present work.

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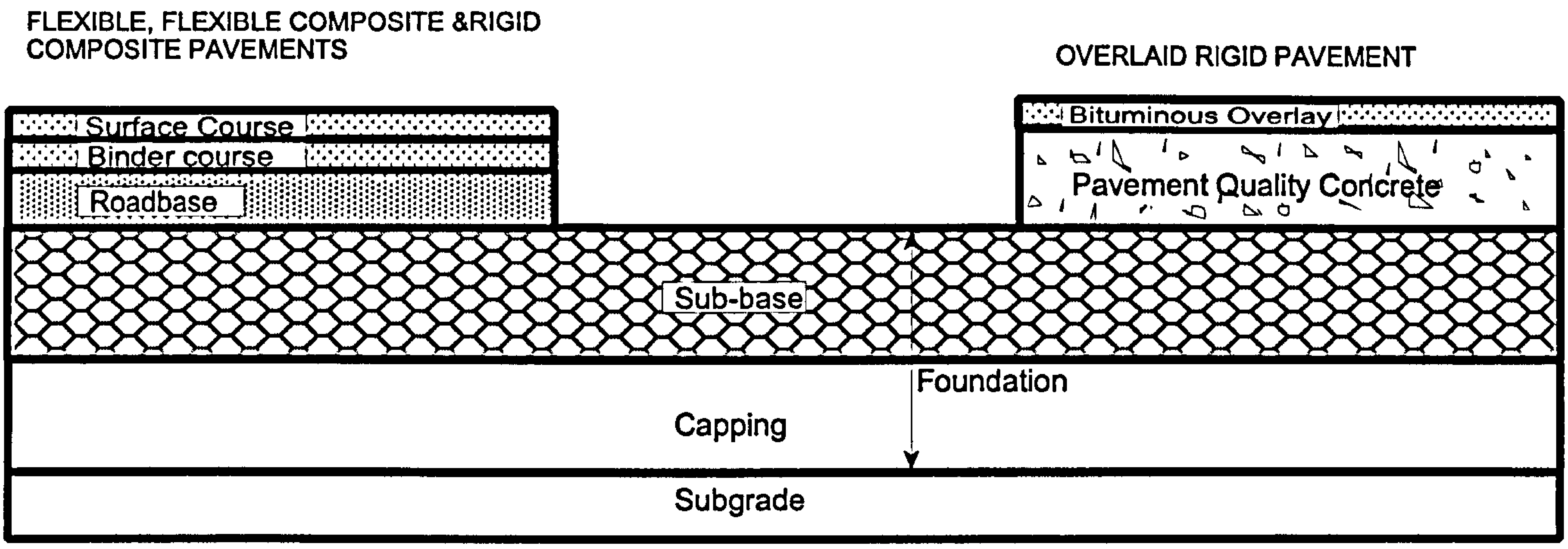
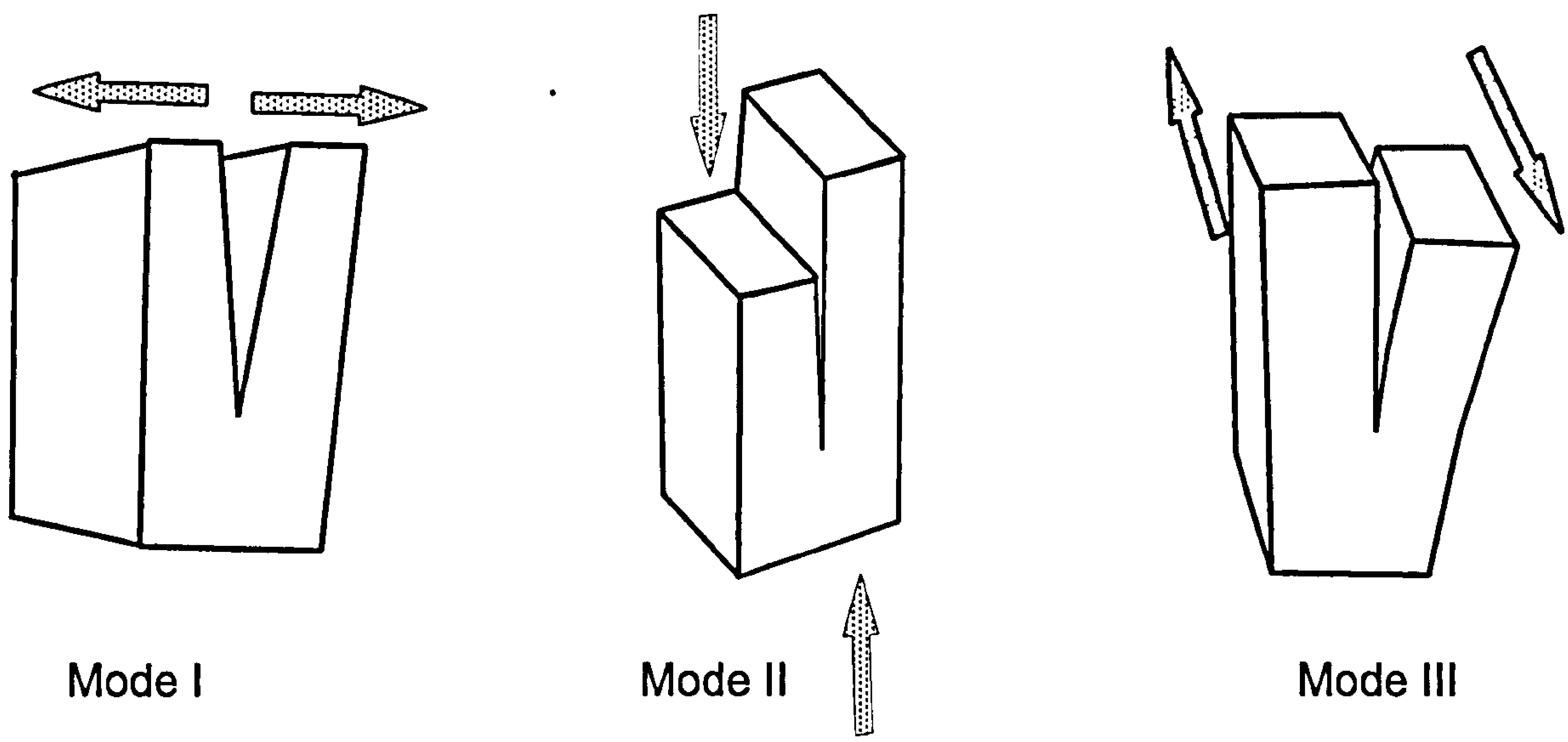
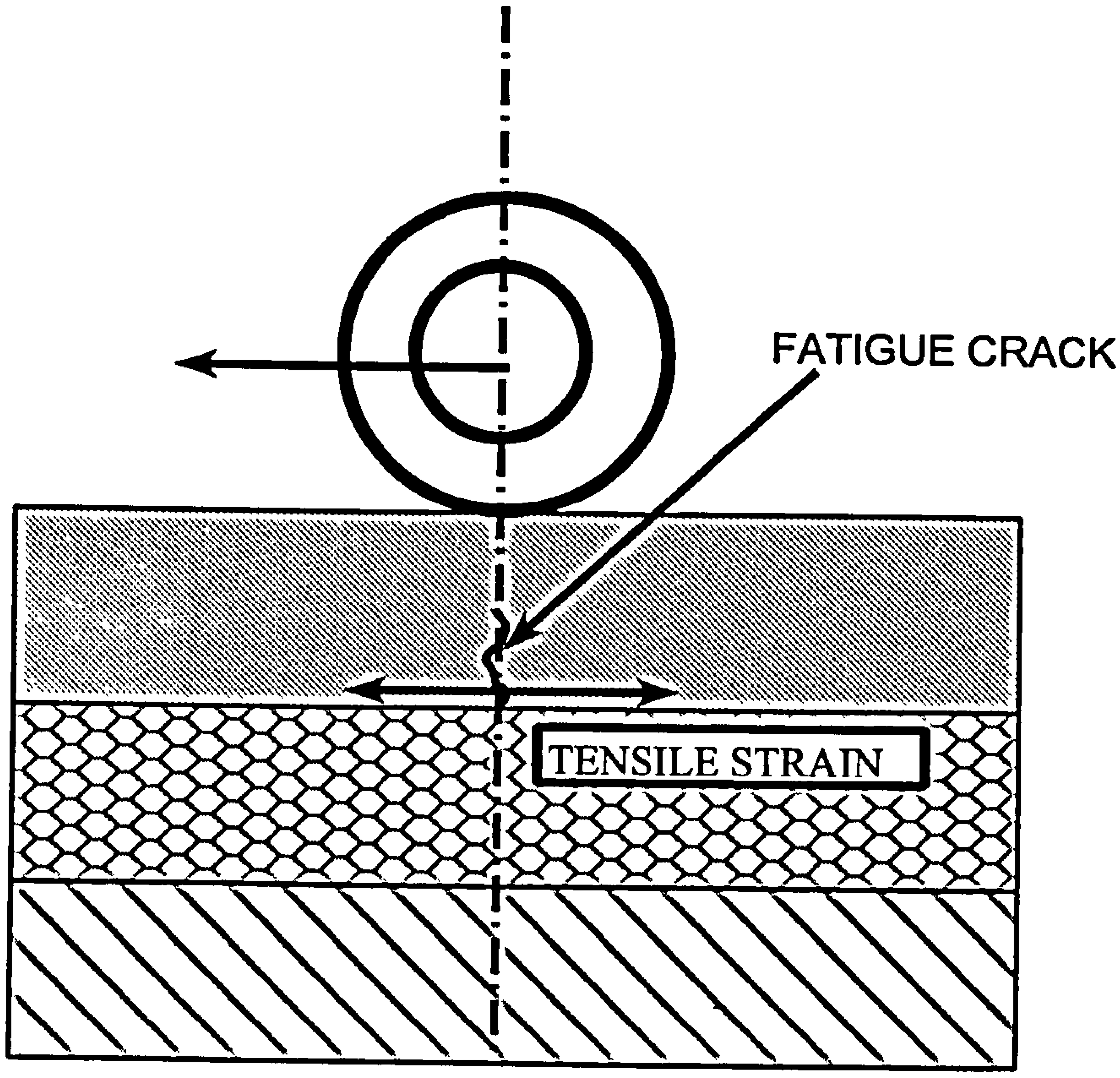


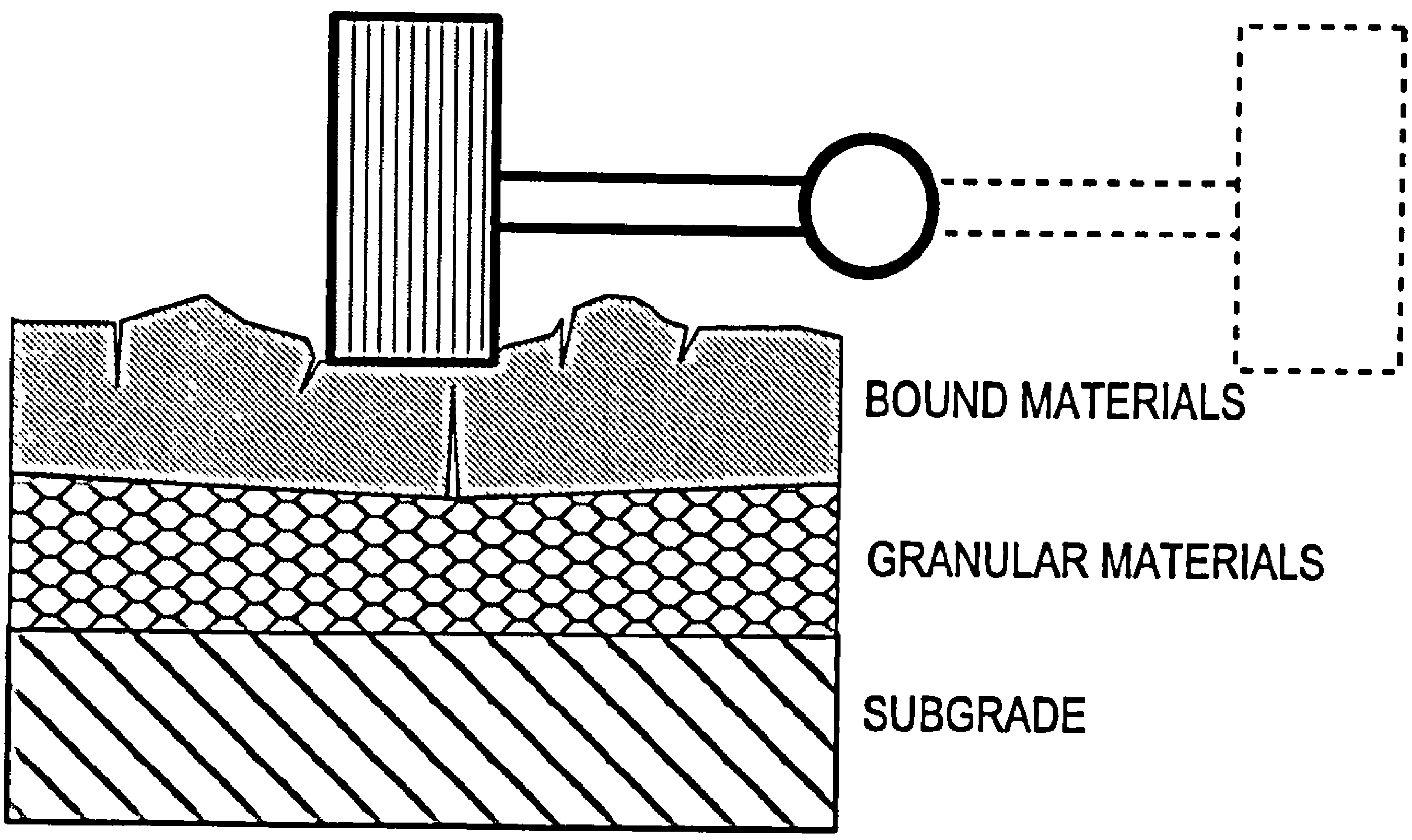
PAVEMENT STRUCTURES
FIGURE 1.1



MODES OF CRACKING
FIGURE 1.2



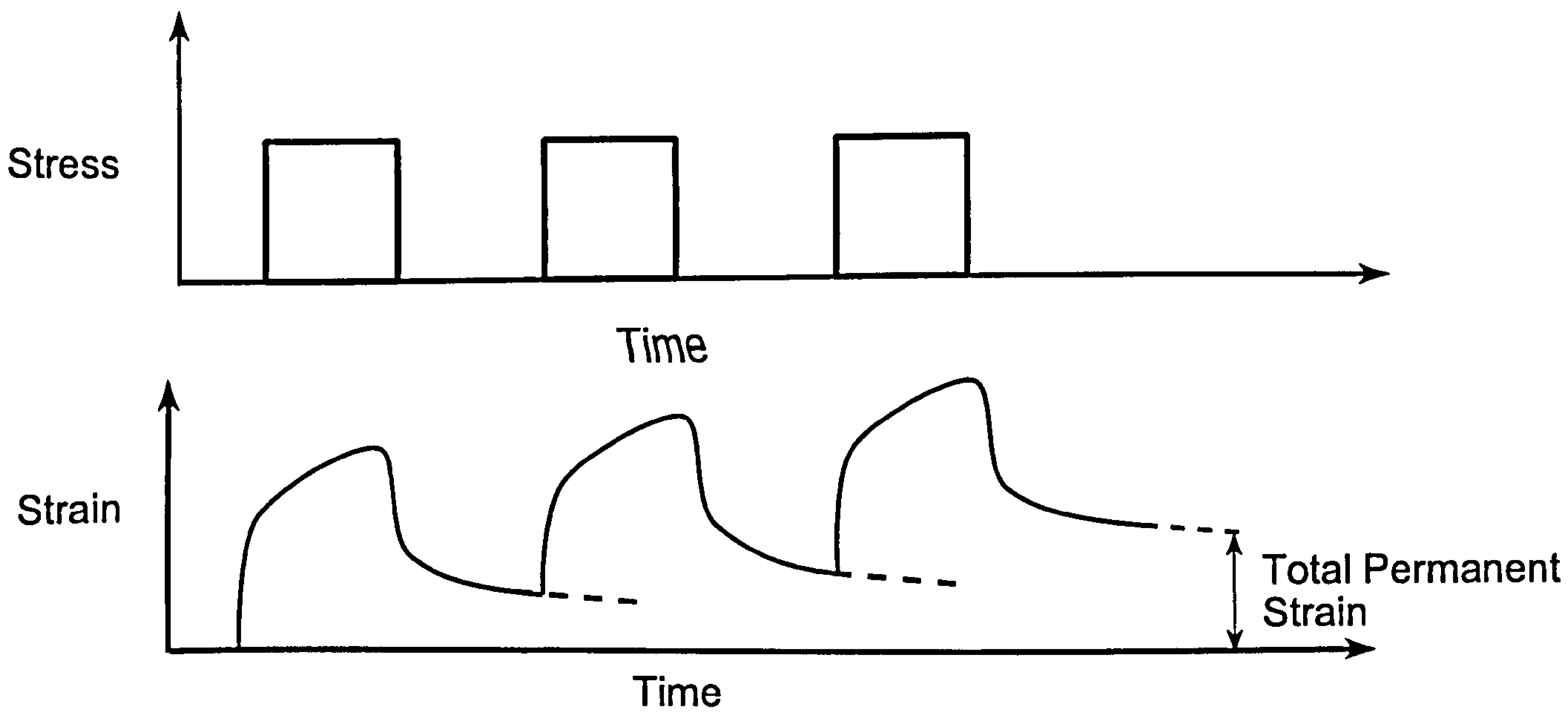
(a) FATIGUE CRACKING AND CRITICAL STRAINS



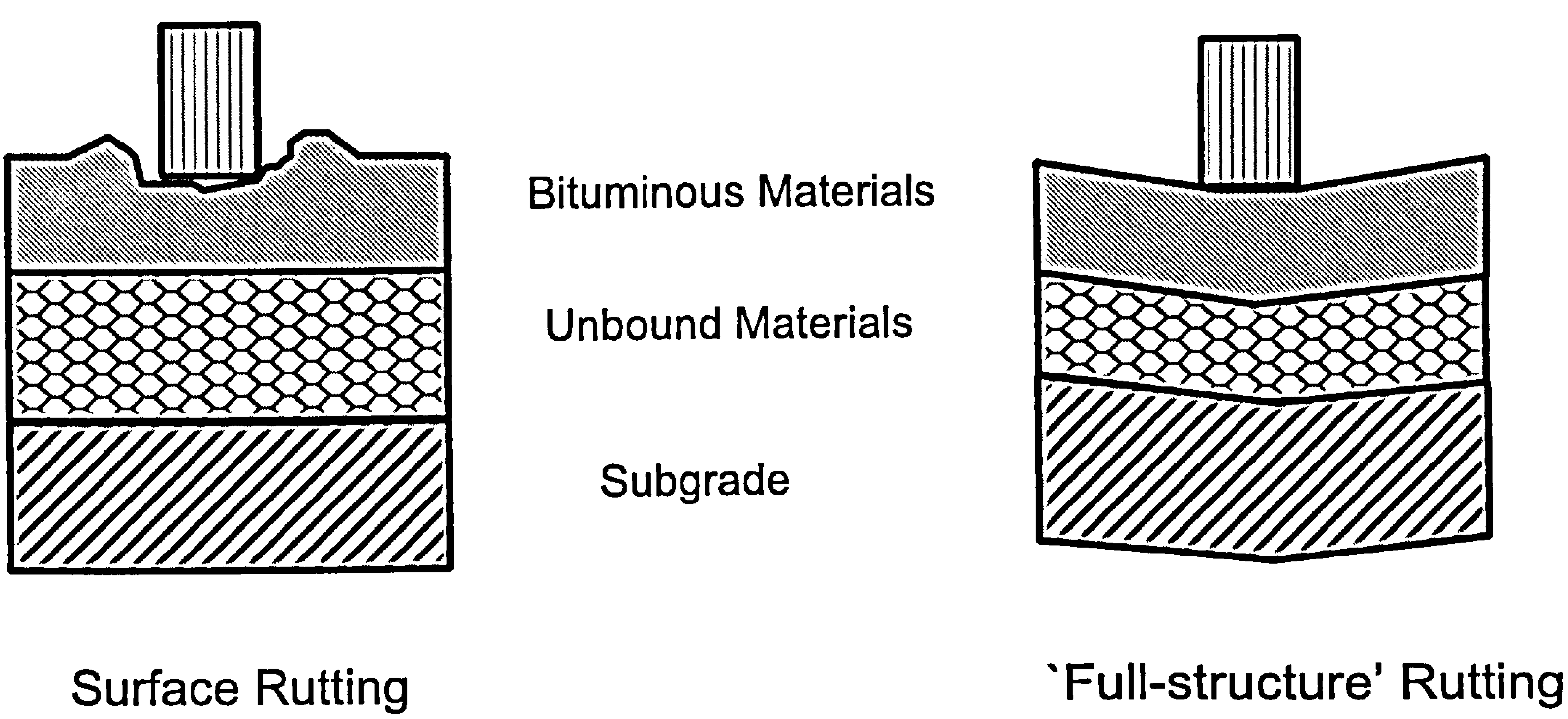
(b) RUTTING AND CRACKING

FAILURE MODES AND CRITICAL STRAINS

FIGURE 1.3



DEVELOPMENT OF PERMANENT STRAIN
DUE TO REPEATED LOADING
FIGURE 1.4



TYPES OF RUTTING
FIGURE 1.5

CHAPTER 2

PROBLEM DEFINITION - DESK STUDY

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CHAPTER 2

PROBLEM DEFINITION - DESK STUDY

2.1 Introduction

Ultimately, the use of a treatment in practice depends on whether it is economical or not. Therefore, to begin the investigation of the use and performance of reinforced asphalt, discussions were held with manufacturers and agents responsible for maintaining the English Trunk Road and Motorway network. This yielded some seemingly contradictory views on the performance of reinforced asphalt in general, and of specific products in particular. From the discussions it seemed that personal prejudice and extrapolation of limited experiences to the overall use of reinforced asphalt tended to give bias to the 'true' situation. In an attempt to resolve this issue, confirm that the overall aim of the work was indeed appropriate, and possibly help to define ways of addressing the main issues raised, more objective information was required. Accordingly, two main approaches were followed;

(i) a survey of organisations using or designing reinforcing products for reinforced asphalt in Britain, and
(ii) a literature review.

Results of the survey are now provided and discussed, and the literature review is given in Chapter 3.

2.2 Survey

A questionnaire was sent to 185 organisations within Britain, listing a range of information required. The questions included

- whether the organisation had indeed used reinforced asphalt, and if so,
- what type of reinforcement was used
- whether problems were experienced during placement of the reinforced asphalt
- performance rating of the reinforced asphalt layer

The organisations selected were thought to represent a cross-section of pavement-related organisations and included county and borough councils, consultants and contractors.

The format of the questionnaire was intended to help define the use of the main types of pavement interlayers, i.e. fabrics, polymer grids, glass-reinforced grids and steel grids. Also, definition of how frequently grids or fabrics are used, in what situation they are used, and whether they have been successful was sought.

The form of the questionnaire used is given below:

	Geotextile (e.g. Polyfelt)	**Polymer Grid or composite (e.g. NETLON AR-1 or AR-G)**	**Glass Reinforced Grid or composite (e.g. Glasgrid/ Rotaflex)**	**Steel Mesh (e.g. Roadmesh)**
Frequency of usage				
Never				
Once				
Rarely				
Often				
Application				
New flexible roads				
Overlaid jointed concrete				
CBM base roads				
Overlaid STIFF flexible road				
Overlaid WEAK flexible road				
Success rating				
Caused problems				
No clear benefit				
Clear benefit				
Good performance				
Too early to tell				

The results of the questionnaire are summarised in Table 2.1 and Figure 2.1. Details of the type of reinforcement used were obtained from subsequent follow-up discussions.

Table 2.1 Summary of questionnaire returns

Category	Geotextile	Polymer		Glass grids	Steel Mesh
		AR1[A]	AR-G[A]		
No. of affirmative replies	12	17	3	13	10
Problems experienced	2	5	1	1	3
Clear benefit	1	9	-	6	3
Good performance	4	7	-	6	2
Too early to tell	2	3	1	9	3
No clear benefit	3	1	1	1	-
Note Reference	B	C	-	D	E

Notes

A Distinction between AR1 and AR-G was made in the returns.

B Good performance' was reported on rigid pavements (x2), new flexible pavements (x1), and with overlays on weak flexible pavements (x1).

C 'Good performance' was reported on weak flexible pavements (x5), and on new flexible pavements (x2). It appears that in some cases although problems were experienced during placement of AR1, once the material was installed, good performance was obtained.

D 'Good performance' was reported on weak flexible pavements (x4), on CBM base pavements (x1) and on overlaid rigid pavements (x1).

E 'Good performance' was reported on weak flexible pavements (x1), and on overlaid CBM-base pavements (x1).

Of the 29 organisations that reported using, installing or designing reinforced asphalt, 23 were County and Borough Councils, two were consultants and four, Contractors.

The survey provided some interesting results, and trends in the performance of particular types of reinforced asphalt became more obvious, especially during subsequent discussions with respondents to the survey. However, a shortcoming of the investigation was that in many cases it was difficult to clearly define what was meant by 'clear benefit' or 'good performance' from either the survey sheets, or from subsequent follow-up discussions. One of the main reasons for the lack of clear definition seemed to be, that unless problems are reported, engineers do not, as a rule, closely monitor sections that are not trial sections. Indeed, it appears that in general, rating of the performance of reinforced asphalt sections tends to be subjective and little information additional to that supplied with the questionnaires was found to exist. Also, in many cases, 'control' sections adjacent to the reinforced sections were not constructed, so objective comparisons of performance were difficult to make.

Figure 2.2 portrays the data given in Figure 2.1 as percentages. It is interesting to note that problems were only experienced on 4% of glass grid installations whereas the rate for steel grids is 28%. However, this may be misleading as the 'Problems Experienced' category generally refers to problems experienced during installation. Installation techniques have developed considerably in the past few years and have become more reliable, thus probably reducing the incidence of problems. If the categories 'Good Performance' and 'Clear Benefit' are combined, polymer grids have the highest percentage. However, the returns also showed that 'problems' were experienced with this category of reinforcement 21% of the time. It is noted that the percentage of returns in the category 'Too early to tell' is significant, particularly for the glass grids. Depending in which category these will eventually fall, the present distribution might be considerably different.

A summary of the most significant points found in the survey is now given:

- The number of organisations (1 in 6) using steel reinforcement was thought surprising as steel grids had not long been commercially available when the survey was carried out in 1996. This contrasts with polypropylene grids, for example, that had been used for a decade or more.

- 'Good performance' was reported with each type of reinforced asphalt, i.e. polymer grids and fabrics, steel products and glass-reinforced products, in particular situations. This is considered particularly interesting, as for this to be the case, it is reasoned, different mechanisms of asphalt-reinforcement interaction must exist.

- Polypropylene grids tend to perform well on 'weak' foundations.

- Polypropylene grids and composites and steel grids were prone to giving problems during construction.

- Glass-reinforced materials were found to be quite popular and generally performed well, but were apparently not as effective as the polypropylene materials on pavements with high deflections.

  Of the glass products, the proprietary product 'Glasgrid' was popular, partly due to its general performance, and partly due to the ease of installation (on account of the self-adhesive backing).

- In addition to the above observation regarding polypropylene grids on weak foundations, there was a perception that polypropylene grids and composites were more effective than glass products where differential (vertical) movements at joints in concrete slabs were relatively large.

- Geotextiles seemed to be effective over jointed concrete pavements. It was thought that this could be due to the bitumen-soaked geotextiles' ability to undergo high strains and reduce stress concentrations.

- From discussions during the survey it appears that the relatively few returns from consultants seems to indicate that the perceived risks (the absence of a recognised design method) in specifying reinforcement in asphalt are considered to outweigh the possible benefits. Also, there seemed to be a general lack of awareness (or interest) among consultants concerning the use of fabrics or grids in pavements. In addition, regional offices of the Highways Agency are often reluctant to take responsibility for accepting reinforced asphalt as a reliable option in maintaining the trunk road network.

2.3 Results and Implications of the Survey

Assessment of the survey results and follow-up discussions with respondents is described as follows:

a) A considerable number of highway-related organisations have had some contact with reinforcement in bituminous pavements.

b) Each of the 'main' groups of reinforcement is used in Britain.

c) In general, limited knowledge exists of the performance of the treatments used to date.

d) Linked to c), measures of cost-effectiveness are almost always subjective.

e) Understanding of the way that reinforced asphalt 'works' is largely anecdotal.

f) Before reinforced asphalt is routinely used and accepted by designers and highway authorities as a valid maintenance treatment they must be shown to be effective, reliable and affordable.

g) For reinforced asphalt to be accepted by highway authorities, reliable design methods are required.

h) To develop the design methods in g) good understanding of the manner in which grids and fabrics influence pavement performance is required.

i) Results of well-documented full-scale (field) trials would help provide confidence to organisations considering the use of grids and fabrics as maintenance treatments.

j) The relatively high incidence of 'poor' performance appears linked to problems during installation. This is a practical issue that must be resolved.

As stated in f), for reinforced asphalt to be used routinely, the clients (highway authorities) need to be convinced that reinforced asphalt is an appropriate maintenance solution. To provide this reassurance full-scale trials are preferred, but if this option is chosen, several years trafficking are normally required. During this period, i.e. before the results of the trials are known, maintenance still has to be carried out, and so funding may be provided for unsuitable treatments. Alternatives to quicken the production of results, and so implement results sooner may be considered. These include testing of small-scale samples in the laboratory or large-scale wheel tracking tests, which may be carried out in the field or in the laboratory. Although it may be expedient to test small-scale samples in the laboratory, it is difficult to apply test results to full-scale situations. Accelerated testing of test sections of pavements, on the other hand, provides test results that can be more easily applied to 'live' pavements more directly and with more confidence. Benefits of using this approach include:

- several years trafficking can be applied in a few weeks or months,
- the magnitude of the load is known and can be varied
- the effects of temperature and moisture ingress can be accurately monitored and (depending on the test configuration), controlled.

Accordingly, for the reasons given above, accelerated testing of 'pilot-scale' sections of reinforced asphalt pavement were carried out, and are described in Chapter 8. Whilst not the preferred full-scale test, the Pavement Test Facility (PTF) was considered a worthy compromise, able to provide valuable insight into the performance of reinforced asphalt under wheel loading. To supplement the relatively large-scale wheel tracking tests (by providing information on constituent materials), small scale laboratory tests were also carried out, as described in Chapters 5 to 7.

Appendix

Notes on Follow-up Discussions

Where possible, additional information to that supplied in the survey was obtained through telephone discussion. Table 2.2 gives a summary of the main points of these contacts.

Table 2.2. Summary notes from 'follow-ups'

Organisation	Type of Reinforcement	Observations
City & County of Swansea	Steel grid	'Ruffling' and rippling appeared on the surface of the (possibly 80mm) overlay.
Scottish Borders	Poymer: Tensar AR1	Cracking and separation at interface with a 40mm overlay. No problems with 100mm overlay on grids.
Hampshire County Council	Geotextiles	Can cause 'fatting-up' – probable migration of binder to surface.
	Grids	Polymer grids were found to be better than glasgrid in situations of 'high' deflection.
Dorset County Council	Steel grid	The grid 'moved' during installation of the wearing course – possible problems in future.
Birmingham City Council	Glasgrid	Loss of adhesion due to placement in wet conditions.
Powys County Council	Polyester Grid	Good performance over a road with high deflections (the road was over a bog).
Derbyshire	Polymer Grids	Applications have performed well on an area of haunch settlement and as a large patch repair.
	Glass grid	Placing difficulties led to subsequent debonding. Also, under heavy quarry traffic over a haunch, a 'thin' basecourse 'slid' on the grid.
Staffordshire	Steel and polymer grids	After initial problems with placing grids (keeping materials flat), good performance was obtained.
Wrekin construction (Shropshire)	Polymer grids Geotextiles Steel grids Glass grids	All products were seen to slow-down but not prevent deterioration. See note 1 below.

Note 1 Problems were experienced with laying geotextiles, as they tended to 'pick-up' on the wheels of construction traffic.
Glasgrid was found to be the easiest to apply due to the self-adhesive backing.
All products present problems on roads with tight bends.

Table 2.2. Summary notes from 'follow-ups' (continued).

Organisation	Type of Reinforcement	Observations
Suffolk County Council	Poymer: Tensar AR1 (Grid)	Difficult to lay, but 'very effective in maintaining the integrity and serviceability of the overlay'
	Poymer: AR-G (Composite)	Difficult to lay and overlay cracked immediately.
	Glasgrid	Worked well where a flexible pavement was widened with a rigid haunch. Used to remedy the problems noted above.
Cumbria County Council	Glasgrid	Mixed success on flexible composite pavements.
	Polymer grids	Very effective on pavements with weak subgrades. See Note 2 below
Highland Council	Polymer and Steel grids	Grids were placed on single-track roads subjected to very heavy forestry vehicles. Of the polymer grids, Tensar AR1 was found difficult to lay, Hatelit was easier, and performed as well. Steel mesh is the other 'preferred' solution.

Note 2: It was accepted that it was not possible to stop the pavements deflecting, but the polymer grids hold the bituminous materials together.

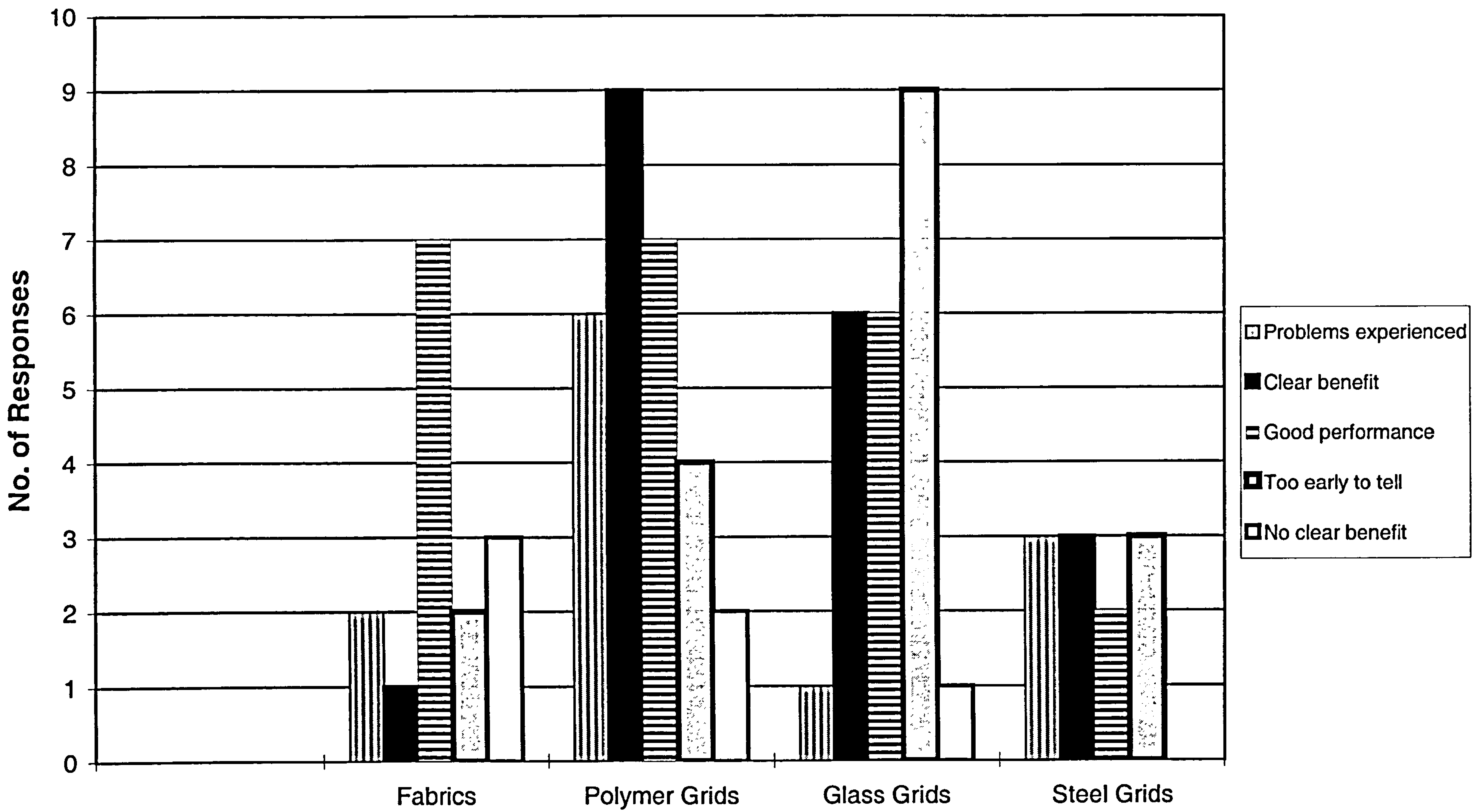

FIGURE 2.1
RESULTS OF QUESTIONNAIRE

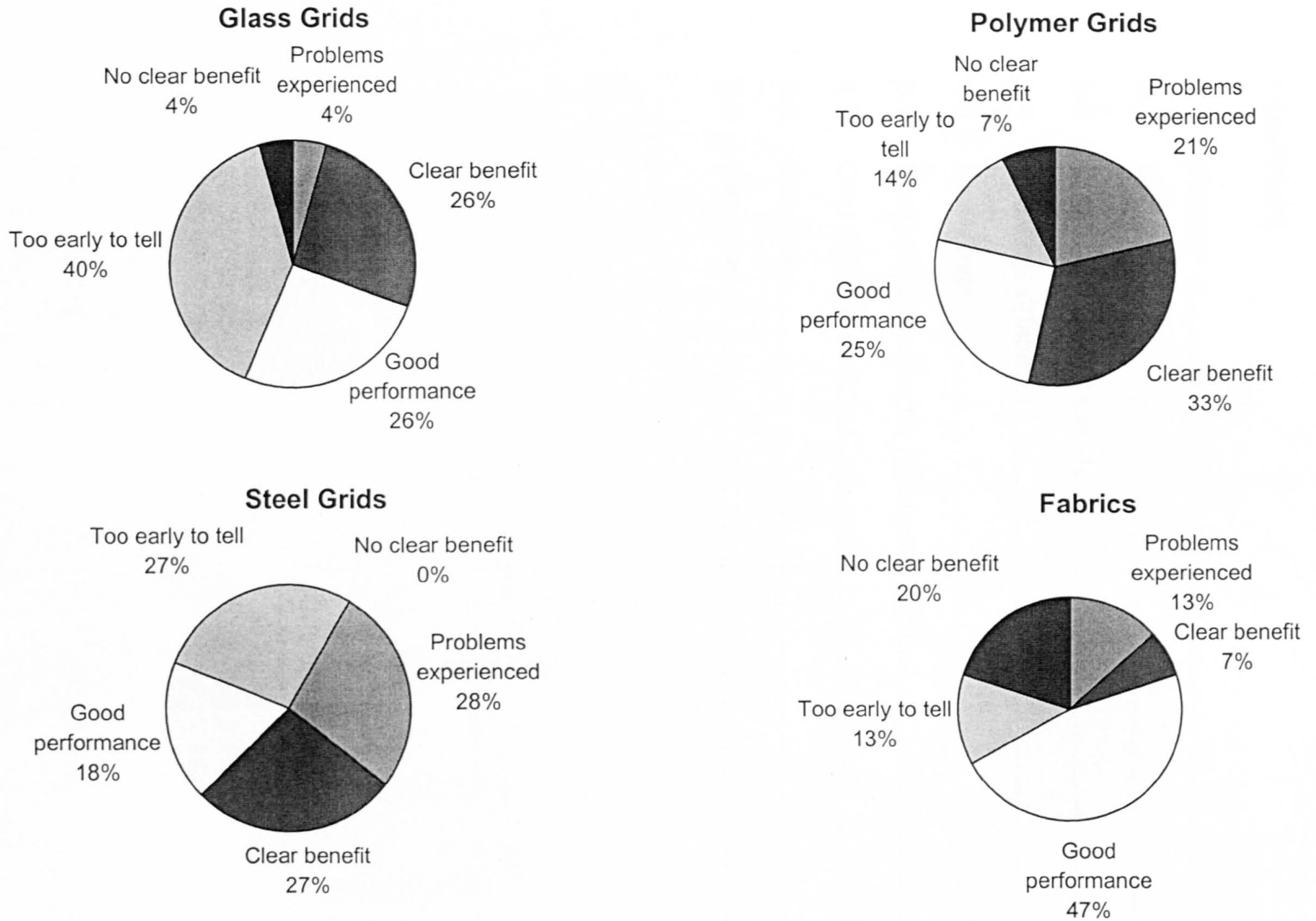

FIGURE 2.2
ANALYSIS OF QUESTIONNAIRE RESULTS

CHAPTER 3

PROBLEM DEFINTION – LITERATURE REVIEW

CONTENTS

CHAPTER 3

PROBLEM DEFINITION - LITERATURE REVIEW

3.1 General

From results of the survey described in Chapter 2, it was obvious that more information on the performance of reinforced asphalt was required before an effective investigation to establish design principles could be formulated. Also, the survey results suggest that there is no clear-cut answer to the question of whether reinforced asphalt is cost-effective, and it is this question that ultimately needs to be answered before reinforced asphalt can be used with confidence in practice.

The main purpose of the literature review was therefore to further define the field performance of reinforced asphalt, and so help:

(i) establish the main influences on performance, and
(ii) investigate the cost-effectiveness of reinforced asphalt.

In addition to these findings, information on suitable test techniques and design approaches was also sought.

Accordingly, information describing the behaviour of reinforced pavements is summarised below, and information relating to the investigation of laboratory performance of reinforced asphalt is provided in Chapters 5 to 8.

In a similar way to the results of the questionnaire, the literature survey showed that there are a limited amount of well-documented 'complete' case histories that permit a full and fair comparison of the performance of different reinforced asphalt solutions to be made.

The main reason for the lack of detailed case studies is typically that of cost, as to obtain sufficient data from which to draw reliable conclusions, trial sections require expensive instrumentation and careful monitoring. Also, considerable time is normally required to ensure sufficient trafficking has occurred for reliable comparisons with other reinforced, and unreinforced sections to be made. In addition, where traffic is the prime cause of distress, it is often difficult to determine the 'effective' traffic load that the section has experienced, bearing in mind factors such as traffic wander and variations in speed, axle load and dynamic effects.

Some of the largest full-scale trials reported in the literature were carried out in America. These include; for example, the studies carried out in New Mexico [3.1], Pennsylvania [3.2], and Illinois [3.3]. These studies summarise the assessment of around 13 different interlayer treatments on a large number (literally hundreds) of sites.

Monitoring and evaluation of the trials indicated that reinforced asphalt could be beneficial and cost-effective when incorporated into an overlaid pavement, although this was by no means true in all cases. In one reference, [3.2] the authors actually comment that 'paving fabrics and fibrous treatments to retard reflective cracking are not recommended on the current analysis of life cycle costs'. Similar studies in Europe [3.4 and 3.5] also show mixed results, with grids and fabrics performing both well and poorly in a variety of circumstances. Accordingly, from this evidence the cost-effectiveness of reinforced asphalt is not obvious.

It appears that the main reason for difficulties in determining the performance, and hence accurate appraisal of the cost-effectiveness of reinforced asphalt is the number of variables involved and their possible combinations. The main variables include:

- **pavement type** (e.g. flexible, or composite)
- **traffic volume**
- **traffic wheel loads**
- the **condition of the pavement** before overlaying with reinforced asphalt
- pavement **foundation support,**
- **differential movement of cracks and joints**
- the **type of fabric or grid** used
- the **position** of the reinforcement in the pavement
- the **type of bituminous mixture** applied,
- the **thickness of the bituminous mixture** applied,
- the manner and degree by which the reinforcement is **bonded** to the asphalt
- **climate,** (in particular annual and daily temperature changes).

Inevitably, each parameter will have a different influence on the overall performance of a reinforced asphalt pavement depending on the situation, so the number of possible combinations is large. In assessing documented trials of reinforced asphalt, it is intended to identify the most important variables and thus enable a suitable investigation to be carried out.

A summary of findings relating to the main types of reinforcing products is now given.

3.2 Polymer Grids

Nunn and Potter [3.6] give the results of a road trial site built on an overlaid cracked concrete pavement to compare the performance of asphalt reinforced with grids with a control section. After four years, the section built with a polypropylene Tensar grid was seen to have around 50% of the number of cracks reported on the control and HaTelit(polyester)-reinforced sections. The principal cause of cracking was 'low' temperature (cracks were reported to have initiated in the winter months).

Herbst et al [3.7] present the findings of trials on overlaid concrete pavements in Austria where the performance of layers reinforced with geotextiles, geogrids, and a steel mesh, was compared with a control section and one treated with a SAMI (in this instance a layer of granulated asphalt beneath the binder course). The authors report that of the 60mm-thick asphalt sections trafficked, the Tensar-reinforced pavements (where a levelling course was used) behaved well, although the trial was inconclusive due to the limited (3 years) data. The steel-reinforced sections cracked early in the trial, which was thought due primarily to installation (nailing) problems. Early performance of the geotextile-reinforced section was good, although later in the trial, more cracking was noted.

A simple economic appraisal was made comparing the cost of maintenance treatments, i.e. the difference between a 120mm overlay and 60mm overlays with the various treatments. This showed that the geotextile solution to be cheapest, with the geogrid in second place followed by the conventional 120mm overlay. However, performance was not explicitly taken into account in the appraisal, which makes the comparison less useful. The principal cause of cracking was not stated, but as the asphalt was placed on concrete, thermal effects were suspected.

Gilchrist et al [3.8] describe the performance of a polypropylene grid-fabric composite (Tensar AR-G) used in a pavement on a soft foundation. The composite was placed over a regulating layer with Hot Rolled Asphalt (HRA) as a binder course. After five years, the pavement was free of cracks, which compared well with the control section which cracked within four months. The principal cause of cracking was that of relatively large movement of the foundation.

Two examples from Germany [3.9], generally agree with results from the survey described in Section 2.1, i.e. that polymer grids seem to be effective in pavements with weak foundations.

The first site was an approach road built over clay and peat in Schleswig-Holstein which was mainly trafficked by heavy vehicles. Tensar grid was fixed to the old pavement by nailing, and protected from construction traffic using a chip-and-spray treatment. This was overlaid with an 80mm bituminous layer. After 7 years of heavy trafficking no cracking was reported. Previous to this, cracks reappeared in the pavement surface within two years of overlays being applied. The principal cause of cracking was that of traffic-induced movements on a soft foundation.

The second pavement described was built on a high embankment over peat and clay. The road had been trafficked for 6 years to allow for the majority of settlement to occur before the reinforced overlay was placed, and was badly cracked. The principal reason for using the reinforced overlay was to keep the thickness (and hence vertical load) to a minimum to reduce further settlement. 80mm of bituminous overlay was then placed, and to improve the bond between the grid and the old pavement, a polymer-modified binder was used in a chip-and-spray process. Monitoring over four years subsequent to placement of the reinforced overlay showed the pavement to be uncracked. As for the previous case, the principal cause of cracking in the unreinforced pavement was that of traffic-induced movements on a soft foundation.

The performance of a range of maintenance treatments applied to a flexible composite pavement was compared by Silfwerbrand and Carlsson [3.10] The treatments included polymer-modified binders (in the binder course), geogrids and a geotextile. Results showed the section containing the geogrid to be the poorest performer, which was partially attributed to a poor placement technique. The authors observed that treatments comprising 'homogenous asphalt layers' or of two layers of similar properties (stiffness) cracked least of all.

Although the cause of cracking of the initial asphalt layer was not defined in the article, with flexible-composite pavements, a combination of shrinkage cracks in the roadbase and traffic loading is suspected.

The performance of trial sections of the M6 motorway in England is described by O'Farrel [3.11]. As for the previous case history, the pavement treated had a flexible composite structure, and the treatments under investigation were combinations of fabric, geogrids and polymer-modified asphalt. *Inter alia*, results of the trial showed that

- Rutting increased where a geotextile was present.
- Placing the reinforcement between the roadbase and bindercourse was more effective in inhibiting cracking than if the reinforcement was placed between the binder course and the surfacing.
- GlasGrid was more effective laid in 'larger' continuous areas than strip. treatments.
- The GlasGrid was easier to place than the polypropylene grid.

As for the previous case history, the cause of cracking in the asphalt surfacing is thought to be a combination of traffic loading and shrinkage cracks in the CBM roadbase.

A road trial comprising 17 sections in Sweden, reported by Johansson and Ancker [3.5], compared the performance of geogrids, geofabrics and control sections on a thin pavement (350mm from top of subgrade to the surface). The construction was that of a grouted macadam base under a 50mm surfacing, which had suffered extensive longitudinal and alligator cracking. The geogrid suffered from problems with installation, which led to considerable cracking early in the trial. Other treatments, on the other hand were monitored for six years and showed (*inter alia*) that the polyester geotextile performed considerably better than the polypropylene fabric. This was attributed to the larger amount of binder used with the polyester, which suggests that the binder-soaked geotextile functioned as a stress-reducing layer, similar to a SAMI. This might suggest that the main function of the geotextile is a bitumen reservoir.

The alligator cracking in the 'original' pavement was attributed to binder aging, but no reason for the longitudinal cracking was given.

Yaromko et al [3.12] report the findings of maintenance treatments applied to roads in Belarus that employed geotextiles, grids and control sections. It appears that the geotextile-reinforced sections fared better than the polymer grid-reinforced sections, although all grid and geotextile treatments were found to reduce cracking.

In summary, although little detail is given in most of the references, there is a suggestion that polypropylene grids are more effective in crack reduction when used in a situation where vertical movements are 'high', such on a soft foundation. This contrasts with situations on rigid or flexible composite pavements where movements are often expected to be thermally-induced, and thus tending to be horizontal. However, although often performing well once installed, polypropylene grids were generally found to be more difficult to place successfully than some other reinforcement types.

3.3 Glass-reinforced Materials

Five case histories giving examples of the application of glass-reinforced composites and grids to fully flexible, flexible composite, and granular base pavements have been described by Doligez and Coppens [3.13]. The composites were positioned below the wearing course, i.e. covered with between 20 and 80mm of asphalt, and to gauge the effect of the maintenance treatments, Benkleman Beam and LaCroix Deflectograph tests were carried out before and after maintenance was carried out. Rut measurements on some of the sites were also quoted.

Test measurements taken before and after application of the composites showed both rutting and transient deflections to be less than the unreinforced pavements. In addition, deflections actually reduced on two of the pavements as time passed, suggesting that the condition of the foundation had improved. The authors quote an engineer responsible for the road as saying that 'it seems that the stiffness of the glass fibre distributes the stress and helps the soil structure to become stable', but no detailed suggestion was given as to how this actually occurred.

From the article it appears that the glass-reinforced interlayer is effective on relatively thin roads subjected to heavily loaded axles. However, as a cautionary note, the authors make the point that vertical shear must be considered before specifying glass-reinforced grids. Although not specifically stating why, it is assumed that these relatively brittle grids are susceptible to this type of failure. Also, (and common to many of the references encountered during the literature review) correct placement of the grids was stated as being very important to obtain good performance.

Reference 3.12 reported that glass-reinforced grids generally performed well on Roads in Belarus. However, it is not obvious on what road structures the materials were placed, or what the mode of cracking was.

As mentioned in Section 3.2, trials on the M6 motorway in England [3.11] showed that GlasGrid applied in 'continuous' layers was more effective than

GlasGrid applied in strips. Also, the application of this adhesive-backed product was found to be more straightforward than the use of polypropylene grids.

3.4 Geotextiles (fabrics)

Examples where geotextiles have been used successfully can be found on most pavement types, and they are more widely used than any other type of interlayer product. This is largely due to their relative ease of application, their (perceived) generally 'good' performance, and 'low' price per square metre.

A key difference between fabrics and grids is the absorption properties of the fabrics for soaking-up bitumen. This results in a tendency to act like a SAMI (stress absorbing membrane interlayer), absorbing stresses, rather than adding strength to the pavement. This being the case, it is understandable that geotextiles have been reported as not performing well under load on soft foundations. However, as is seen below, this is not always the case, as geotextiles have been successfully used as part of thin surfacings on 'low volume' roads in Australia, although for this usage, the primary function is more to waterproof the pavement, than to act as a SAMI.

A selection of references describing the use of geotextiles is summarised and given below.

A study in New Mexico [3.1] compared 7 different interlayer treaments on six sites on a mixture of fully flexible and flexible composite pavements. The treatments investigated were non-woven fabrics and rubberized asphalt, and were compared to the performance of control sections. Results showed that in general, interlayers can retard the rate of reflective cracking and therefore make savings on maintenance costs. However, the benefits of the treatments were not clear-cut. Mention was made of the importance of good construction control, as problems were experienced in laying some of the products, largely because of insufficient preparation of the pavement surfaces and incorrect amounts of 'bonding coat' (binder) being applied.

A field experiment in Pennsylvania [3.2] was set up to compare the relative performance of four different fabric interlayers, a fibre-asphalt interlayer, a polyester fibre-reinforced asphalt overlay and a control section. The pavement had a mixture of fully flexible and overlaid concrete construction. Problems were encountered during placement of materials, due mainly to 'contractor inexperience' but also to the nature of a heatbonded fabric which tended to 'wrinkle' and move under site traffic.

Significantly, after 44 months none of the treatments were considered cost-effective, (compared on the basis of surface cracking), but it is fair to say that this area is prone to low temperatures and wide thermal cracks against which geotextiles are (reputedly) not effective.

In the UK, Walsh [3.14] reported good performance for geotextiles over cracked concrete slabs, especially when used with an HRA overlay. A

potentially important point is made concerning the need to limit the amount of vertical movement of the slabs before application of the geotextile. In this case, it was achieved by grouting voids beneath the slab.
A comparison of the costs of geotextile (£1.30/m^2) and an 'equivalent' 40mm overlay (£4.00/m^2) was given. However, it was also reported that until both the untreated and the geotextile-treated sections had cracked, a fair comparison could not be made.

Other instances where geotextiles have been used over concrete or flexible-composite pavements are given in references 3.15 to 3.21, and in general, the results of the trials indicate good performance. However, there are some exceptions, such as Reference 3.20, where more rutting was recorded in the fabric-reinforced section, and in Reference 3.19 where cracks were reported as reflecting through the fabric-reinforced layer relatively quickly. This was attributed to the concrete (over which the reinforced asphalt was placed) not being cracked and seated prior to laying the reinforced overlay.

Apart from the stress-reducing function of a geotextile, the waterproofing properties may be very important, as the papers by Dumont and Decoene [3.16], Van Deuren and Esnouf [3.22] note. A paper by Phillips [3.23] also describes the use of bitumen-impregnated geotextiles to seal pavements with thin structures on expansive soil subgrades. These papers serve to highlight another positive feature of fabric behaviour – their ability to undergo large strains without 'failing'. Also, if cracks do reflect through the wearing course, the bitumen-soaked material still remains watertight. Use of geotextiles to stabilise moisture contents of low volume roads could also be extended to higher grade roads where water-susceptible materials are present.

Barksdale [3.24] collated and summarised a large amount of information from field trials in the USA and concluded the following:

In general fabrics were found to be more effective in reducing the incidence and severity of reflection cracking in temperate or warm climates. In some cases the waterproofing qualities of bitumen-soaked fabrics were noted as being the most important benefit of applying fabrics.

For flexible pavements:

- Moderate to significant levels of reflection cracking could be delayed by 2 to 4 years by using a full-width fabric interlayer.
- Fabrics were found to be most effective where tight closely-spaced cracks were found (similar to 'alligator' crack patterns).
- The limit of crack width for successful application was 10mm.
- Fabrics were not effective where thermal cracking was a problem (as these cracks may often be greater than 12mm wide).

Mukhtar and Dempsey [3.25] note that geotextiles tend to perform better on flexible pavements that exhibit distress via closely-spaced alligator (fatigue) cracking, rather than on pavements with large cracks and/or large deflections.

Limiting values for use of geotextiles are given in Table 3.1.

Table 3.1 Recommended Limits for Geotextile use [3.25]

	Vertical Deflection (mm)	**Horizontal Displacement (mm)**	**Maximum Crack or joint width[1] (mm)**
Geotextile not required	<0.05	<0.5mm	--
Suitable Range	0.05-0.2	0.5-1.8	3.0 – 10
Geotextile not suitable	>0.2	>1.8	>10

Note 1. If cracks are greater than 10mm, it is recommended that a joint filler is used.

Barksdale [3.24] agrees with the point made above, that geotextiles tend to perform better on flexible pavements with alligator (fatigue) cracking than where large thermal movements are present, and also provides limiting values for the use of geotextiles. These are given in Table 3.2

Table 3.2 Recommended Limits for 'Lightweight' Geotextile use [3.24].

	Vertical Deflection (mm)	**Maximum Crack or joint width[1] (mm)**
Best performance	<0.03	3-10
Geotextile not suitable	-	>10

Note 1. If cracks are greater than 10mm, it is recommended that a joint filler is used.

A minimum overlay thickness of 50mm was recommended for applications of reinforcement in temperate regions and between 75 and 100mm for areas with an 'Average freezing Index' of between 0-500 'degree-days' (Celsius).

For rigid pavements (according to Barksdale [3.24]), both transverse and longitudinal reflection cracking could be delayed by 2 to 4 years if the following conditions were applicable:

- Vertical joint movements, as measured by the Benkleman Beam, should be between 0.05mm and 0.19mm. For movements greater than 0,19mm, a fabric did not help, and for movements less than 0,05mm, a fabric was unnecessary.
- Horizontal joint movements should be less than 1.2mm.

Barksdale [3.24] suggests that geotextiles are not as effective for concrete pavements as they are for flexible pavements. This is due to the (typically) wider joint and crack openings in rigid pavements, and larger differential movements at concrete joints, as compared to the more dispersed movements associated with alligator cracking, for instance. However, where recommendations are made regarding the application of reinforcement on overlaid rigid pavements, both 'heavy-duty' membranes and paving fabrics are referred to. Heavy-duty membranes are normally applied as strips of around 200 to 600mm over cracks, and consist of composite material comprising layers of fabric bonded to other bituminous or rubberised layers. The intended function of these materials is (similarly to the lightweight fabrics) to waterproof the pavement, and in addition provide a stress-reducing layer. Use of these membranes was found to provide up to 5-7years delay for reflective cracking if used with an overlay of at least 60mm.

Limits of crack or joint movements for placing fabrics on rigid pavements are given in Table 3.3.

Table 3.3 Recommended Limits for Fabrics over Rigid Pavements [3.24]

	Vertical Deflection (mm)[1]	**Horizontal Movement (mm)**
Geotextile not required	<0.08	<0.76
Best performance	0.08 - 0.2	0.76-1.78
Geotextile not suitable	>0.2	>1.78

Note 1 Deflection measured by the Benkleman Beam across a joint.

Regional variation of performance due to climatic factors was also noted by Barksdale. In particular, geotextiles were found to perform better in more temperate climates, than in areas that experience very low temperatures. Barksdale also noted that for overlays greater than 130mm, geotextiles had no influence on performance.

One possible problem with heavy duty membranes was the likelihood for cracks to form over the edges of the membrane if it is particularly stiff, and this has also been noted with glass-reinforced strip repairs. However, no recommended solution has been given except to increase the overlay thickness, which tends to invalidate part of the reason for using a geotextile in the first place.

Before reinforcement is applied, preparation of the concrete surface was considered to be very important. In particular, joint movements need to be reduced to an acceptable level. This may be done thorugh replacing and refixing dowels, under-slab grouting or cracking and seating. Repairs to spalling joints also need to be effected and cracks must be sealed, especially if they are greater than 6mm wide.

In terms of cost-effectiveness, Barksdale considered initial construction costs, and the likely maintenance strategies with-or-without geotextiles. Although difficult to determine, as scarce reliable data exists, the finding was that there was little or no clear benefit in using geotextile layers.

The key points taken from case histories relating to geotextiles are now summarised:

- Geotextiles do not reinforce pavements in the traditional sense, as their stiffness is considerably lower than the asphalt to which they are bonded.

- Geotextiles tend to perform better on flexible pavements where cracking is more dispersed and generally narrower than on rigid pavements.
- Geotextiles do not perform well where they are placed over joints or cracks that are subject to large vertical movements. In this regard, cracking and seating of concrete pavements has been found to be an effective solution before the application of geotextiles.
- Due to their bitumen-absorption properties, Geotextiles help to keep a pavement protected against the ingress of water, even after cracks have passed through it. Also, it follows that modified bitumens can also be used to impart additional beneficial properties to the geotextile-bitumen interlayer.
- Following from the above point, geotextile-reinforced seals can prove an effective surfacing option for low volume rural roads, especially those built on expansive materials.
- A bitumen-soaked geotextile can act similarly to a SAMI.
- Little hard evidence exists to show geotextiles are a cost-effective treatment, although there seems to be a general acceptance of some degree of benefit amongst users of these materials.
- Geotextiles may reduce rut resistance in the surfacing.
- Procedures used to install geotextiles can have a significant influence on subsequent pavement performance. Use of an experienced contractor is important in this regard.

3.5 Steel Grids

The possible use of steel as reinforcement for asphalt pavements has been considered seriously (and trialled) since the 1950s in the form of continuous meshes or smaller wire mesh strips [3.26]. Wire reinforcement was often in the form of welded mesh [3.27] and was tensioned during construction to remove bulging and bowing of the grid which tended to occur during paving. Performance of the sections was mixed and it was not obvious which factors determining pavement performance had the greatest influence. However, it was noted by Brownridge [3.27], that wire-reinforced pavements performed better in regions where daily and annual temperature extremes were not 'excessive'. In this case the temperature variations referred to were around 50°C. with a minimum of -23°C in southern Ontario and variations of 58°C, and a minimum of –33°C. in the north.

Thirty years later, wire mesh is still being used with varying degrees of success, but with a general tendency to use products with smaller, more flexible profiles. In particular, twisted wire has largely taken the place of welded mesh. Veys [3.28] gives a summary of ten projects where steel mesh was used on roads in Belgium. Of the ten projects,

- Five describe overlaid concrete slabs,
- Three describe overlays on lean-mix concrete, and
- Two give descriptions of overlays on asphalt roads.

Since the maintenance treatments were carried out (three sites in 1989, one in 1990, four in 1991 and two in 1992) good pavement performance has been observed. The author reports that six years later, both cracking and rutting were substantially reduced or eliminated.

The reasons for the reduction of rutting were given as (a) increased support and load spreading ability (which in turn reduce stress on supporting layers), and (b) the interlock of aggregate and the large aperture wire mesh which helped to prevent lateral movement of the asphalt mixture.

After treatment, the only cracking reported was attributed to the 1mm vertical movement (rocking) of the overlaid concrete slab, and the relatively thin (40mm) asphalt overlay.

In an update to the work described in Reference 3.28, Vanelstraete and Francken [3.19] note that the steel-reinforced overlays on concrete slabs performed best where slab rocking had been reduced by cracking and seating, and thicker overlays were used. In particular, it was noted that overlays greater than 50mm should be used if steel is fixed using slurry, with the thickness increasing to 60mm when nails are used to fix the grids. Where these measures were taken, the overlays were reported to perform well after 10 years of service. It is noted that a reduced minimum overlay thickness is allowed if a slurry fixing is used, due to the slurry bond between the grid and the existing pavement being more uniform than is obtained with nailing.

3.6 Design Approaches for Reinforced Asphalt

Although limited in number, a wide variety of approaches to reinforced asphalt design was found, ranging from 'educated estimates' to stress analysis using Finite Element Analysis (FEA). This wide range of complexity reflects the difficulty in dealing with combined variations in pavement conditions, interlayer materials and wheel loading. For practical reasons (i.e. to simplify the design procedure), design methods inevitably tend to focus on a limited number of parameters, and in doing so, may not (explicitly) take into account some important influences.

Although several design approaches exist, assessment of the accuracy and applicability of the methods is not straightforward. This is because the comparison of predicted behaviour and subsequent performance is often not done. This in turn may be due to:

(a) The unique characteristics of each site, coupled with the fact that design approaches are almost always developed from 'back-analysis' of field observations (and/or laboratory test results). Where predictions of future performance are given, the treated lengths are normally in the early stages of trafficking and long-term pavement behaviour is unknown.

(b) It is difficult to measure traffic loading in the field, and beyond simple observations of crack and rut measurements, movements of overlaid concrete slabs, or thermally-induced changes in pavement layers (for instance) are difficult to quantify.

(c) Due to the long-term and expensive monitoring that is normally required to determine the performance of a pavement section, recording often lapses after a year or two. Differences in performance may not be obvious in the initial years, and may mean that the time at which differences in performance between reinforced sections and control sections become obvious are missed. This tends to happen when personnel involved in monitoring leave the organisation, or if resources are reallocated to other projects depending on new priorities.

The uncertainties regarding loading and monitoring of pavement performance may be resolved using large scale accelerated tests. In these tests wheel loading is controlled, and instrumentation can be used without the danger of breakages due to traffic or environmental effects. This approach is described later.

A summary of design approaches is now given.

3.6.1 Reinforcing Design : Multi-layer Linear Elastic Theory (MLLET)

This approach uses 'standard' multi-layer linear elastic theory and (normally) assumes full bonding between layers. Typically, strain at the bottom of a bituminous layer is computed and used for calculating the pavement life to crack initiation. Thereafter, the time required for the crack to propagate through a layer, is often taken into account simply by using a 'crack propagation factor'. The value for this factor may be derived from observation and experience, or alternatively, from a relationship such as the Paris Law [3.29] which relates stresses at the crack tip to the rate of crack propagation. However, if a FEA approach is used to develop values for this crack propagation factor, then the use of the less-sophisticated MLLET approach may be redundant.

To calculate tensile strain at the bottom of the asphalt for a reinforced pavement, properties of the fabric or grid (elastic modulus, Poisson's ratio and thickness) are input together with other layer data for the 'conventional' pavement layers, into a multi-layer, linear elastic programme, such as CHEVRON [3.30]. The calculated strain is then compared with appropriate relationships relating tensile strain to pavement life (for fatigue cracking).

Although this approach seems logical and straightforward there are a number of associated drawbacks, including assumptions of

1. Conventional reinforcement mechanisms,
2. Full interlayer bond,
3. Bottom-up cracking,
4. Homogenous layers, i.e. no cracks and joints in the layers and therefore no stress concentrations.
5. Reinforcement to be a uniform continuous layer.

The assumption of Point 1, that the reinforcement takes load from the asphalt is true only if the reinforcement is stiffer than the asphalt or if the crack has passed beyond the reinforcement. Higher stiffness is not the case, though, with many of the interlayer materials, particularly the non-woven fabrics.

If there is some degree of slip between adjacent layers, the assumption of full inter-layer bond in Point 2, can lead to large differences between predicted and actual stresses. A situation where this may occur is where layers of significantly different stiffnesses lie adjacent to each other, and the difference in strain can cause bonding between the layers to be excessively strained. However, if the assumption of full bond can be shown to be applicable then this approach can be used, especially if the anticipated mode of failure is cracking initiating at the bottom of the pavement.

Another drawback to the use of conventional multi-layer theory is its inability to model local effects of discontinuities (cracks or joints) in layers. From intuition, observation and literature (see the RILEM conferences referred to in references 3.3, 3.4 and 3.6, for instance), it is obvious that discontinuities have an effect on stress distribution.

One approach used to design an overlay on a 'uniformly cracked' pavement is to reduce the stiffness of the existing pavement. This leads to the thickness of bituminous overlay material being increased to take into account the reduced support. Obviously, this approach does not take into account the stress concentrations induced by discontinuities which are prime contributors to reflection cracking. For this reason, therefore, stresses and strains calculated by this method may be underestimated.

An alternative approach to overlay design over cracked pavements has been given by Eckmann [3.31]. First of all, the pavement is divided into 'many layers' (often ten or more), then, for each layer, strains are calculated and a crack initiation relationship applied, followed by a crack propagation relationship based on Paris' Law. Although this approach is an improvement on the 'usual three to five layers used in an analysis, it is still limited by the other limitations mentioned above.

A design example using MLLET for steel grids has been obtained from a supplier [3.32] which used the following steps: First of all the BISAR program [3.33] was used to calculate tensile strain at the bottom of an unreinforced overlay, using the procedure outlined above. The procedure was then repeated with a reinforced overlay. The calculation of allowable traffic was carried out for both cases using a fatigue law derived using laboratory data:

$$\log N_{lab} = a + b \log \varepsilon_{allowable}$$

Where

N_{lab} = the number of loads to failure in the laboratory, ('equivalent' to 80kN axle loads)

$\varepsilon_{allowable}$ = permitted strain for a given design traffic

a and b are factors depending on mix type, temperature and speed of loading.

The values of a and b used in the example supplied were -9.38 and -4.16 respectively.

In the example N_{field} was taken as equal to N_{lab}, and a factor of 10 used to take into account lateral traffic wander and rest periods (between traffic loads), which are not reflected in laboratory testing. Note that a 'perfect' bond (i.e. no slip) was assumed between the asphalt and the grid, which, through its relatively large stiffness modulus, reduced the tensile strain in the asphalt. A comparison was then made between the cost of the reinforced and unreinforced overlay design and a decision made on this basis.

3.6.2 The Meshtrack/Bitufor Approach [3.34]

This approach has been developed for twisted wire (steel) grids that are installed using a bituminous slurry. The grid dealt with in this method is essentially the same as the one referred to in 3.6.1, but in this later approach, analysis was carried out using FEA to calculate tensile strain from either thermal movements or vertical wheel loads. Four main cases were considered in the analysis–

1) Thermal movements on concrete slabs,
2) Concrete slabs under traffic loading
3) Thermal and traffic loading on a flexible composite pavement, and
4) Traffic loading on a fully flexible pavement.

The ratios of reinforced to unreinforced pavement life derived from the analysis are shown in Tables 3.4 and 3.5.

Table 3.4 Life Improvement Factors- Temperature-Induced Crack Initiation

Overlay Thickness (mm)	Height of Grid in Overlay (mm)	Transverse Cracking	Longitudinal Cracking
60	10	6.5	16
60	20	1.3	Not given
100	10	8.8	24
100	20	1.9	Not given

Table 3.5 Life Improvement Factors – Crack Initiation induced by slab 'rocking'.

Overlay Thickness (mm)	Transverse Cracking	Longitudinal Cracking
60	3	2.4
90	3.3	4.3
120	3.5	7.8
150	4.9	Not given

As an alternative measure of the effect of the Bitufor grid in delaying cracking, reductions in asphalt thickness of between 25 % and 30% were calculated for an equivalent life to crack initiation. It was noted that as the overlay thickness was increased, the presence of Bitufor was predicted to become more effective, although the reason for this was not apparent.

3.6.3 The 'Nottingham & Netlon' Approach

Research into the behaviour of grid interlayers in bituminous mixtures has been carried out at the University of Nottingham since 1981 [3.35 – 3.42]. Principally work has been carried out using polypropylene grids, although testing was also carried out on polyester and glass-reinforced materials. Various aspects of grid and asphalt behaviour were investigated and tentative design guidelines were produced. In addition to work carried out to investigate the cracking and rutting behaviour of reinforced asphalt, the field installation process was also considered [3.40]. Satisfactory grid installation was found to be difficult to achieve due to movement of the grid during paving. The grid was found to require anchoring and tensioning, and this was perceived to be crucial to subsequent pavement performance. Satisfactory anchorage was however shown to be difficult to achieve consistently in the field.

Early in the research programmes the influence of temperature and cyclic loading on the performance of the Tensar grid was investigated. The object was to establish whether any significant change of properties occurred when laid with a hot bituminous mixture, and then subjected to traffic loading in the field. This work led to heat setting of the polypropylene grids during manufacture to prevent excessive shrinkage and loss of stiffness. A range of tests were carried out in the laboratory to simulate field conditions. These included loading asphalt beams supported by plywood sheets over a rubber 'foundation', as well as by using a larger wheel tracking slab testing facility, and a pilot-scale pavement test (the pavement test facility-PTF). Test results brought to light various points:

- **Crack initiation.** The grid did not alter the stiffness of the pavement significantly, thus not reducing the time to crack initiation [3.35].

- The rate of **crack propagation** could be reduced significantly with the inclusion of grids [3.35].

- **Reflection cracking** could potentially be reduced significantly, if not eliminated, if the grid was placed close to the bottom of the bituminous overlay material [3.35].

- Also, it was shown that the **fatigue life** of beams tested on an elastic support could be improved by a factor of 10.

- **Rutting.** With a grid placed at a depth of 0.25 x the width of the loaded area, rutting could be reduced by a factor of 3.

- An attempt to simulate cracking due to **thermal effects** made use of an apparatus that simulated the effect on an overlay of the slow opening and closing of a crack or joint in an existing pavement [3.35]. Results of this testing were inconclusive.

- The **influence of interface properties** on the shearing characteristics of a pavement was investigated using a large-scale shear box. Test results showed large variations in shear capacities of reinforced slabs (generally between 60 and 90 percent of control specimen values). Also, in general results suggested that grids reduced shear strength less than was the case with fabrics. However, from the large scatter of test results, it appeared that there were a variety of factors influencing test results and not simply whether reinforcement was a grid or a fabric. These apparently include the different types of interlayer (grids or fabrics), tack coat types and application rates, test control (stress or strain), the magnitude of normal loads applied and shearing rates.

- The effect of **debonding** of the grid adjacent to the existing crack or joint was considered by Brown et al [3.41] who showed that local debonding helps to reduce the stress intensity around the discontinuity, and hence the tensile strains, thus improving the fatigue life.

- Linked to the design of reinforced asphalt is the consideration of **installation techniques** [3.38 and 3.42]. This is an important consideration as it was found that if the grid was not securely anchored to the pavement surface, it would tend to lift during the paving operation. Various methods to prevent this from happening were developed included padcoats, pre-tensioning the grids and using surface dressings to hold the grid in place.

Using this method for design of reinforced asphalt layers, the main three points are:

- Fatigue life is increased by a factor of 10,
- Resistance to reflective cracking increases by a factor of 3, and
- Rate of rut development is reduced to a third of that of unreinforced material.

However, these factors rely on grids being placed 'appropriately' i.e. in the correct position, and with good workmanship. In this regard, to prevent reflection cracking, grids should be placed near the bottom of the bituminous layer, whereas for rutting (in the surfacing), a position near the top of the pavement is more suitable. This requires the designer to anticipate which is the most likely failure mode that will occur.

In addition, it was noted that the type of mixture to be used as an overlay may also have an influence on the best position to place the grid within the layer. Hot Rolled Asphalt, for example, is generally more susceptible to rutting than Dense Bitumen Macadam (DBM), and may suggest that grids used with HRA overlays would be more effective if placed closer to the surface that would be the case with DBM. This of course could reduce the effectiveness the grid in limiting cracking, if cracks initiate at the bottom of the layer and move upwards.

3.6.4 The Nottingham Approach – OLCRACK [3.43].

The OLCRACK design programme was developed partly as a consequence of the testing and numerical modelling carried out during the project herein described. The programme allows a designer to select and design reinforced overlays against top-down, and bottom-up cracking taking into account

- reinforcement strength and stiffness,
- load magnitude,
- crack spacing,
- foundation strength and
- grid-asphalt interlock, or
- interface (bond),

The programme was developed using test results and observations from the following:

- beam tests (for measurements of crack growth - see Chapter 7).
- trafficking of half-scale pavements on jointed concrete slabs (for resistance to reflective cracking – see Chapter 8),
- large-scale shear box tests (for interface conditions – see Chapter 6), and
- strength and stiffness testing of reinforcement – (see Chapter 5).

To calculate rates of crack propagation, tensile strain is first calculated at the surface and at the lower asphalt interface using adaptations of beam-bending theory. Then, and similarly to the Paris law (that describes the relationship between rates of crack growth and stress at the crack tip), OLCRACK uses a relationship between tensile strain and crack growth rate:

$$\frac{dc}{dN} = A\varepsilon^n$$

Where dc/dn is the crack growth rate,

A and n are taken from fatigue lines from laboratory testing, and

ε is a measure of strain in the 'cracked zone'.

The effects of slip between reinforcement and asphalt are taken into account using a relationship between the interface shear stiffness and applied stresses. Also, the 'crack-stitching' effect of the reinforcement across the crack is calculated. The 'stitching' effect serves to reduce stresses and strains within the crack region as the reinforcing action of the grid tends to hold the crack closed.

To calibrate predictions of crack growth with test measurements, a 'fatigue factor' is included to take into account the apparent effect of reinforcement on crack growth, even before the crack reaches the level of reinforcement in some cases.

Initial development of the program was restricted to modelling results of the beam testing. The applicability of the programme was then extended to 'pilot-scale' pavements by modelling test results obtained from the Pavement Test Facility (PTF). Assuming that the PTF is representative of full-scale test results, typical savings of between 20mm and 50mm are predicted.

Table 3.6. Approximate savings from use of grids.

Reinforcement type	Saving on asphalt (mm)
Polymer grid	30
Glass grids	18
Steel grids	40

3.6.5 The Carleton (Ottawa) Approach [3.44].

This approach assesses the different components of the overall mechanism acting under thermal (tensile) loading. Four mechanisms were defined:

- Interlock
- Bond
- Confinement and
- Membrane.

The **interlock** mechanism was defined as the percent of the mobilized strength of the interface due to the anchorage provided by the aggregate within the grid aperture. Interlock was identified as the main component of the reinforcing mechanism with optimum interlock achieved when the ratio between the grid aperture and aggregate size was between 3:1 and 4:1. In addition to the aperture size effect, thickness of the grid strands and the ratio of the strand area to the overall grid area was also found to be important. Also, it was noted that even after peak stresses have been reached and cracks have formed, the grid-asphalt interlock can still provide up to around 80% of peak resistance, especially with grids formed from continuous materials. This is compatible with field observations of polymer grids on pavements that undergo high deflections, i.e. that although the pavement may crack, the pavement remains intact and retains considerable strength.

Bond is the portion of the interface strength provided by adhesion between the reinforcement and asphalt. As the area of reinforcement increases, so does the strength, (providing the bitumen content remains the same). An important difference between this mechanism and the interlock mechanism is that if applied stress exceeds bond strength, interface strength was reported to reduce significantly.

The laboratory test results suggest that the bond mechanism is dominant at small strains whereas the interlock mechanism contributes more strength at higher strains.

The component of strength due to the **confinement** mechanism (horizontal stress, predominantly) is provided through the relationship between the overall width of the reinforcement and the road width. If a strip of reinforcement significantly narrower than the road width is applied, the strength mobilised by this mechanism would be small.

The **membrane** effect is a measure of the stress distribution from the reinforced layer to the layer below. It is influenced by the elasticity of the reinforcement, its position within the layer, the type of underlying layer, and the horizontal distance of the reinforced layer beyond the paved area to be reinforced.

The above mechanisms seem closely linked, and no explicit means of taking each effect of the mechanisms in a pavement design approach was given. In reality their incorporation into a design model will probably be implicit and partly determined by other practical issues, such as the manufactures sizes (widths) of reinforcement.

3.6.6 'Component Design' Approach

This is the most fundamental approach found during the literature survey and has been developed in differing degrees by the Technical University of Delft (TUD) [3.45], and the Texas Transportation Institute (TTI) [3.46].

The basic approach is to characterise the various components of a pavement interlayer system and combine them in a model that allows a rational design procedure to be used. Both methods use fracture mechanics to model the relationship between crack growth through a reinforced asphalt overlay and load applications, albeit in different ways. An outline of the principles of fracture mechanics used in the two approaches follows.

To predict the rate of crack growth through a bituminous layer, the Paris law [3.29] is used:

$$\frac{dc}{dN} = Ak_i^n$$

Where
c = crack length,
N = number of load applications
dc/dN = rate of crack growth,

A and n are parameters dependant on the fracture properties of the bituminous mixture
and k represents a stress intensity factor (SIF).

A SIF gives a measure of the energy that causes the material to crack. The stresses causing the cracking might be due to traffic loading (bending and shearing), thermal loading or a combination of both.

To use fracture mechanics for crack growth prediction, appropriate values for the parameters A and n are required. To obtain these values, investigators at the TTI used a combination of asphalt beam testing and Finite Element Analysis [3.46]. The asphalt beams were tested in 4-point beam bending apparatuses and an 'overlay tester' which was used to simulate thermal displacements. From the analysis of test results general relationships between A and n for bending, shear and thermally developed stresses were developed [3.46]. Recommendations for obtaining good estimates of these parameters (without having to carry out difficult and sophisticated testing), were also made. In particular the TTI work found that

$$n=2/m, \text{ where}$$

m is the slope of the log creep compliance versus log time curve.

For traffic-associated cracking and typical (Texas) asphalt concrete mixtures,
$n = -2.2-0.5\log(A_f)$, and
For thermally-induced cracking with the same bituminous mixture,
$n = -0.92-0.42(\log A_T)$

Bending, shearing and thermal cracking modes were investigated from which SIF values were computed. Factors found to influence cracking included

- tyre pressures,
- bituminous thicknesses, (existing and overlay)
- overlay stiffness and Poisson's ratio
- the thermal coefficient of expansion of the cracked surfacing,
- crack spacing in the existing surfacing, and
- the maximum change of temperature in the original pavement layer (after overlaying).

The design equations finally derived were calibrated for 6 climatic zones in the USA, using extensive data on pavement condition, support, bituminous material and temperature changes.

The method has been developed using a mixture of theory, numerical modelling and observation and calibration. Therefore, as use of the approach away from the regions used for calibration is not recommended, before the approach is used, calibration for local conditions is required.

The more recent development of a 'component analysis' by TUD [3.45] has followed a largely analytical approach using detailed Finite Element Analysis (FEA) and laboratory testing to develop a model for the calculation of required overlay thicknesses. To calibrate the analytical approach, data from sites in the Netherlands was used during the development and calibration of the model. The approach is less dependant on climatic data than the TTI design guide and thus may be more useful for general conditions.

The FEA programmes used to analyse the behaviour of asphalt pavements with interlayers are the CAPA-2D, and CAPA-3D programs. Special interface elements have been incorporated into the FE program to allow automatic computation of Stress Intensity Factors (SIFs) as a crack progresses through a layer.

The main input parameters required by CAPA 2D are

- Mesh geometry,
- Elastic properties (stiffness modulus and Poisson's ratio) for the asphalt and reinforcement materials, and other pavement layers.
- Shear and normal stiffnesses for the interface bond,
- Load type (point or line load) and position,

To calculate the expected pavement life for a given interlayer system, a procedure using CAPA has been outlined by Scarpas et al [3.45]:

Average (or equivalent) Stress Intensity Factors (SIF - k_{eq}) are calculated for each crack increment, by placing the load at different distances from the crack and computing values of K_I and K_{II} (SIFs for Mode 1-'bending' , and Mode II.- 'shearing').

k_{eq}, can be calculated from values of K_I and K_{II} as follows:

First the angle of crack extension is calculated from

$$K_I \sin\theta + K_{II}(3\cos\theta - 1) = 0$$

Then using the value determined for θ, an equivalent SIF value can be computed

$$K_{eq} = K_I \cos^3\frac{\theta}{2} \sin\frac{\theta}{2}$$

Crack growth speed (dc/dN) through the overlay elements is then calculated using k_{eq}, A and n (material constants), and the Paris law.

An alternative means of calculating the number of loading cycles to failure is through the use of Miners rule:

$$\eta = \frac{N}{N_{Tot}}$$ where

η is the proportion of pavement life already used,
N is the damage due to a number of loads 'N' and
N_{Tot} represents the total number of load cycles to failure at a given stress level.

The number of repetitions of load required to crack a given element is calculated by both methods and the lowest value is taken.

The calculation is then repeated for all elements of the overlay through which the crack passes. The total lifetime of an overlay is then the sum of the load repetitions required to crack each element of the overlay.

The fatigue life of the interface bond region, is incorporated into the procedure through knowledge of the characteristics of the bitumen i.e. stiffness variation with temperature and frequency. If calculated stresses indicate that interface bond failure has occurred, the programme is rerun with bond stiffnesses set to zero for the element in question. Then, if in subsequent FE runs the 'failed' element is found to be in compression the original value of stiffness is reassigned to take into account friction.

After each FE run, bond stresses are used to calculate the fatigue life of the bond by applying a damage law such as:

$$\text{Damage} = N_{present}/N_{total}$$

Values of N can be calculated by comparing calculated shear stresses with bitumen shear data from Janssen and Molenaar [3.47].

The component modelling approach appears to be the most fundamental approach encountered, as it uses input of individual component material properties and a variety of loading and support conditions. This implies that it can be used for modelling laboratory experiments, and then for field conditions by modelling pavement support conditions and wheel loading in place of laboratory test conditions. With this approach, sensitivity analyses can be carried out to optimise the properties of any given component and hence the performance of the composite pavement structure.

This approach, although fundamentally based has some practical drawbacks, relating to the complexity of the procedure, suggesting that it is not suited to a routine design office application. In particular:

- considerable time is required to set up a FE mesh to calculate stresses,
- 'complicated' laboratory testing is required to measure parameters to provide input data,
- if CAPA-2D is used, realistic modelling of field loading conditions is difficult. CAPA-3D, on the other hand, can model a field situation much more 'realistically' but is considerably more difficult to use.

Notwithstanding the drawbacks mentioned, this approach can be used effectively to investigate 'what-if' scenarios for specialist design situations. For instance, the effect of poor bonding between layers (due perhaps to installation problems) can be simulated, as can the effect of 'large' deflections or crack widths.

As a practical measure it seems that this fundamental technique could be effectively used to compliment and 'calibrate' the more 'simple' approaches such as the MLLET technique. Specifically, a 'library' of meshes could be set-up for typical design situations, to allow the effect of variations in loading, bond stiffness and overlay thickness, for instance, to be readily assessed.

3.6.7 'Estimates' of Improvement and General Points

In some situations specification of reinforced asphalt may be desirable where a 'design' thickness of bituminous material cannot be placed. This may occur, for example, in urban situations where property thresholds limit thicknesses, where load restrictions apply (e.g. on bridges or culverts), or where headroom restrictions exist.

In place of a formal design procedure, empirically-derived logic is used that reasons that the addition of a grid or fabric must result in 'some improvement'. Little or no detailed investigation is normally carried out and recommendations are often made by a supplier of reinforcement products.

Notwithstanding the unscientific nature of specifying a treatment with little or no engineering data, this approach can suffice if a design situation is very similar to that where a previously successful application of reinforced asphalt has been made. In general, however, where this is not the case, and the pavement structure, traffic loading or climate is distinctly different, this approach cannot be recommended.

3.7 Construction Practice

Every design must be linked to practical application and the importance of site practices such as the preparation of the pavement surface for the application of reinforcement has been emphasised in many references. In particular the design of reinforced asphalt solutions assumes that wide cracks are filled and measures are taken to reduce horizontal and vertical deflections to limits that can be tolerated by reinforcement. Although these considerations are similar to those taken for any other overlay treatment, the limits may be different. Also, factors such as handling of certain products (especially brittle products such as glass-reinforced materials), have been pointed out as being possible reasons for subsequent poor performance of reinforced asphalt.

A summary of important issues relating to 'practical' considerations to be borne in mind when designing reinforced asphalt is now given. These include recommendations on preparing a pavement for the application of reinforced asphalt from references 3.4, 3.9 and 3,19:

- Surfaces to be free from potholes and cracks of more than 2 or 3mm wide.
- Any grooving left by planers must be less than 4mm deep.

- Surfaces must be clean and dust free, especially where fabrics are to be fixed with tackcoat.
- Appropriate fixings (that suit the surface to be nailed into) need to be used if grids require nailing.
- Minimum overlay thicknesses should be provided, especially with grids. A minimum of 40mm is suggested [3.9].
- Prior preparation of the pavement was noted as being critical to performance, especially with regard to the correct amount of tack coat being used. The formula given below can be used to estimate the amount of tack coat required:-

  $$RTC = 0.05\ (TW)^{0.3}$$

  Where RTC = recommended tack coat rate (gallons/yard)
  T= Geotextile thickness (mils) and
  W= Geotextile weight (oz/yard)

Where reinforcement is laid on bends, problems arise as without exception, materials are produced for application on straight sections and cutting and lapping is therefore required. This can result in problems if the reinforcement is not lapped and fixed properly, or it results in inadequate asphalt cover.

As noted earlier in the literature review and in the findings of the survey in Chapter 2, many of the negative points raised (regarding sub-standard performance) can be related in some way to the manner of installation of the reinforcement. Accordingly, every supplier of reinforcement has a set of instructions relating to placement of the product being marketed. It is important that these are adhered to. Also, if possible, a specialist contractor with experience in placing grids and fabrics should be used.

3.8 Cost Effectiveness

The issue of economics and cost effectiveness is becoming increasingly important in the consideration of alternative maintenance treatments and strategies. This is due to the competing needs for maintenance budgets which are often barely adequate to provide the service required by the public and demanded by politicians.

The two main factors to be considered when comparing the cost-effectiveness of alternative treatments are

- Initial treatment costs,
- Future maintenance and user costs over a specified time period.

Initial treatment costs are relatively easy to define and compare, but maintenance costs over a given time period are often more difficult to forecast. This is often due to practical issues such as poor records being kept of pavement condition, and even what maintenance was actually carried out. Also, even if good records of pavement condition are kept, it is often not clear why sections may deteriorate more quickly in one area than another. For instance, unless reinforced and unreinforced sections are adjacent to each other, relevant traffic records are required, which are often not available. Other factors, even more difficult to determine, such as the way the sections are built, might also be very important in affecting subsequent performance. For example, if reinforced sections are not constructed correctly, (which may happen due to the contractor being unfamiliar with particular construction techniques), a fair comparison of performance is difficult to make. In addition, to compare treatments fairly, an understanding of why treatments were required in the first place is needed. For instance, in some cases reinforcement might be placed to deal with a cracking problem. If the treatment performs well and delays the incidence of cracking but has little or no effect on rutting, a condition rating system giving high priority to rutting, would underestimate the benefits.

User costs are costs borne by the road user principally through delays due to roadworks, increased fuel consumption through poor riding quality, and accidents, either at the roadworks or due to poor skidding resistance, potholes and ruts, for example. Often these costs are not taken into account in budgeting at a project, regional or local level but are a concern for central government. The information used to quantify these factors is usually obtained from government statistics.

Relatively few references were found specifically addressing issues of cost-effectiveness, due possibly to the reasons given above, plus the fact that where grids have not performed well, detailed costing of longer-term maintenance was not relevant. Note too that estimation of economic benefits based on extrapolation of laboratory test results (often simply beam tests) to field performance may not be reliable, as without proper calibration with field performance this approach has not proved to be accurate. References relating to the cost-effectiveness of reinforced asphalt treatments are now summarised.

A paper by Bozkurt et al [3.3] reports on the detailed assessment of 52 projects in Illinois which focussed on overlays with non-woven paving fabrics over jointed concrete pavements. Climate was found to be one of the most important aspects of paving fabric performance, with fabrics in the warmer areas of Illinois performing better than in more extreme climatic zones. Life cycle costing showed a marginal benefit using paving fabrics, although on 'small' projects, this was not the case due to the higher costs of the fabric (and the relatively small amount of fabrics used). However, it was also pointed out that even after areas treated with the paving fabrics had cracked, pavements remained waterproofed. These benefits, i.e. of protecting the lower pavement layers from moisture, were not taken into account, and so the financial advantages of using paving fabrics were probably underestimated.

Walsh [3.14] makes a direct comparison of the cost of applying geotextile at £1.30/m^2, to £4.00/m^2 for the 'equivalent' 40mm overlay. However, the overlay costs are probably underestimated, as the costs of consequential works such as raising kerbs, gulleys and footways were not included. At a simplistic level, the economic advantages of using the geotextile seem clear, although it was pointed out that a true comparison of costs was not possible until the performance of each section could be evaluated fairly. Once again, the advantages of waterproofing the pavement are not explicitly taken into consideration.

Van Deuren and Esnouf [3.22] infer large cost savings as a consequence of using geotextile-reinforced chip seals, although costings are not given. The authors state that '*all works treated with geotextile chip seals have performed beyond expectations with little or no reflection cracking evident in any of the works carried out over the past 10 years'.* This is a remarkable observation as it refers to use of the maintenance treatment on all types of roads ranging from unsealed roads to major freeways.

As a counter to the positive benefits reported in References 3.14 and 3.22, however, Button [3.48] notes that in Texas paving fabrics showed '*no economic benefits*' when compared to other maintenance treatments. Also, Maurer and Malasheskie [3.2] report that '*based on the extent of cracking after 44 months, and considering the current and proposed crack sealing costs, none of the treatments used on the project were considered cost-effective'.* This statement referred to fibre-reinforced asphalt and four different paving fabrics used on both rigid and flexible pavements in a trial in Pennsylvania.

Barksdale [3.24] discusses the issue of life cycle costs and gives a rule of thumb (for North American conditions) that the cost of a full-width paving fabric is equal to around half an inch of asphalt overlay. Barksdale notes that complete cost comparisons should also take into account ride quality and aesthetics (reduction in surface cracking) and waterproofing, although these are difficult to quantify. Also, the probability of success of a paving fabric ought to be taken into account, and an estimate of 60 to 65% for a successful outcome is given, although, as noted, this should increase as the level of understanding of the mechanisms of asphalt-reinforcement interaction improves.

In addition to the above, Barksdale also points out that even if it seems possible to reduce the thickness of an overlay through use of reinforcement, the structural integrity of a pavement must be taken into account, as reinforcement generally has no effect on pavement strength. In terms of life-cycle costs, it was recommended that cost comparisons be made over the life of an equivalent unreinforced overlay, rather than over the 'remaining pavement life'. Finally, Barksdale notes that the cost of crack treatment was found to be relatively cheap when compared to paving fabrics, and concludes that '*fabrics with overlays on flexible pavements appears to be cost-effective only if quantifiable benefits are derived from other factors such as aesthetics, improved ride quality or reduced water infiltration.*'

Haas [3.49] gives cost-benefit ratios and estimates of cost savings to quantify the benefits of (a) extending the period between construction and overlaying, and (b) treating cracking when it does occur.

a) Assuming costs of $100 000 for overlaying 1km of 2-lane carriageway, by extending the period before an overlay is required by five years estimated savings of $25 000 are quoted.

b) The benefits of carrying out crack treatment were quantified using an estimated life extension of at least two years for a routed and sealed crack. A figure of $7000 per lane-km was given, which, over the project considered (21km x 2 lanes) gave a saving of around $300 000. Although the savings appear substantial this needs to be compared to other treatments to give it perspective.

No figures for reinforced asphalt treatments were given, but if it is accepted that reinforcement can extend the life before an overlay is required by 5 years (as in (a)), the additional treatment costs could be added to the analysis quite simply.

To summarise the appraisal of cost-effectiveness, it appears that no single method identifying the most economic treatment is commonly accepted. However, it is obvious that simple comparison of treatment costs at the time of initial maintenance is not adequate, and that careful consideration of all possible factors in the 'life' of a pavement, or at least in the design period of a 'traditional' maintenance treatment, needs to be carried out. An approach that takes into account both works costs and costs to the user (and therefore the national economy) is given in Chapter 10. This technique is the whole life costing approach and is described in Chapter 10, together with examples of cost-benefit analyses.

3.9 Summary

The literature review has been useful in confirming that reinforced asphalt is found in many forms and is used in many parts of the world with varying results. The review also shows that a large variation exists in the type of materials used to 'reinforce' asphalt, and differences in reinforcement types are not well correlated to full-scale performance. Also, it is obvious that the traditional reinforcement mechanism, (where stiffer reinforcement carries tensile load), is not normally the case with reinforced asphalt, as materials that are weak compared to asphalt (e.g. non-woven fabrics) have been shown to help suppress or delay cracking. The combination of variations in full-scale and laboratory performance, and reinforcement type implies that investigation is required to help define when and how reinforcement can be effective in reducing or eliminating pavement rutting and cracking. This includes the need for an economic appraisal, which ultimately decides whether or not a reinforced asphalt treatment is viable.

It is obvious from the literature review that considerable work has been carried out in the field and in the laboratory to define the properties of the asphalt-reinforcement composite, generally by monitoring 'in-service' pavements for full-scale behaviour, and by testing beams in the laboratory. It is also seems clear that the interface bond is a very important component and plays a large part in determining the performance of reinforced asphalt. Accordingly, the components of asphalt, and particularly the interface bonding, should be investigated as part of the work.

The literature review appears to confirm some of the findings of the UK review on the use of reinforcement asphalt where installation procedures are considered to be important in determining final performance. In some cases this probably over-rides other more definable factors such as reinforcement and asphalt stiffness or strength.

Using the findings of the literature review and the survey, Chapter 4 sets out the way in which the laboratory investigation is to be carried out. In addition to testing, the need for numerical modelling is taken into consideration and is incorporated into the overall plan of investigation.

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CHAPTER-4

PROJECT STRATEGY

CONTENTS

4 PROJECT STRATEGY

4.1 General Considerations – Findings from the Desk Study

From the results of the survey described in Chapter 2 and the literature search in Chapter 3, several key points have emerged. Namely:

- Various asphalt reinforcement product types are already used in the UK, including polymer, steel and glass-reinforced products and geotextiles.

- For common use of reinforced asphalt, it must be shown to be cost-effective.

- Reinforced asphalt can be effective in the control of both cracking and rutting. Definition of when and why this is the case needs to be given, as there are cases where reinforced asphalt has not been successful.

- In the UK, few guidelines exist on the use of reinforced asphalt, except those provided by manufacturers or suppliers of reinforcing products.

- Worldwide, a wealth of experience on the use of reinforced asphalt exists, but due to different climates, pavement structures and traffic loading, only general principles may be applied from one country to another.

- Notwithstanding the name, some reinforcing products do not act in a 'traditional' reinforcing mode, as they are less stiff than asphalt. Two main mechanisms of reinforced asphalt behaviour are commonly recognised. These are:
  (i) Reinforcing, and
  (ii) Stress relief.

The 'reinforcement' mode (typically associated with grids) may or may not act in the true sense of reinforcement, depending on the stiffness and cross-sectional area of the product installed. If the reinforcement cannot reinforce asphalt in the true sense, but still has a positive effect, it seems that the product must help to keep the asphalt together sufficiently to retain interlock across any cracks that form. Thus, load-carrying abilities are retained by the pavement.

Conversely, stress relief is provided through a bitumen soaked fabric as is provided by a Stress Absorbing Membrane Interlayer (SAMI). The relatively soft interface layer absorbs stress through deformation, and helps to 'blunt' cracks.

- The waterproofing role of fabrics is potentially important, especially where reinforced asphalt is placed over pavements with moisture susceptible layers. However, there appears to be no explicit way in which this important attribute is taken into account in any of the design approaches encountered.

- Reinforced asphalt is a composite system with three main components: asphalt, reinforcement and the bond between asphalt and the reinforcement. The relative importance of each needs to be appreciated before reinforced asphalt can be applied in appropriate situations.

- To show that reinforcement can be effective in practice, experiments are required whose test findings are more readily applicable to field situations than laboratory tests on 'small' samples, due to loading and environmental factors often being quite different. Ideally, full-scale testing under live loading is preferred, but testing in this fashion normally takes several years for a satisfactory result, and is expensive. The alternative procedure of accelerated wheel tracking on 'large' samples is a possible solution.

4.2 Behaviour of Thin Reinforced Asphalt Against Reflective Cracking: A Proposed Investigation

To achieve the overall aim – guidelines on the use of reinforced asphalt against reflection cracking, a combination of approaches is required. On the one hand the mechanisms of reinforced asphalt need to be understood at a fundamental level, and on the other hand, the investigation needs to provide appropriate answers to problems in the field. Various options for the development of design guidelines exist, including the empirical approach of monitoring full-scale pavement performance, and numerical modelling with laboratory testing. An appraisal of the advantages and disadvantages of some of the options are now discussed.

Option A Performance monitoring of full-scale pavements under 'real' traffic loading.

Advantages
- An ideal solution if sufficient time and funding is available, as findings can be directly applied with no further calibration required.

Disadvantages
- Findings are difficult to apply to other lengths of pavement unless they are similar in construction, climatic zone and traffic loading.
- These tests are expensive to monitor due to safety considerations (traffic management) and the need to reinstate the pavement after any coring is carried out or test pits opened.
- Instrumentation (if used) is often difficult to maintain due to the typically 'harsh' testing environment.
- Traffic loading can vary daily, weekly and seasonally and be difficult to quantify.
- With any empirical approach, design rules developed from results are only applicable within the data set developed. Any new materials or significant departures from the conditions used to develop the original data can make the application of design rules difficult, and may require a repeat test programme.
- The nature of loading or environment cannot be changed easily.

Option B Accelerated trafficking of full-scale pavements.

Appraisal of this option assumes that the test facility is available i.e. the capital costs of the equipment has already been met.

Advantages

- Test results can be generated in a matter of months as compared to years with the Option A.
- Testing can be carried out in the field or in large-scale test tracks where traffic loading can be controlled.
- If testing is carried out within large 'hangers', environmental factors can be controlled to a reasonable degree.
- To help develop a more fundamental understanding of behaviour, instrumentation can be installed and monitored easily.
- Testing can be carried out in-situ [4.1], or in semi-laboratory conditions [4.2].

Disadvantages

- Construction of a full-scale pavement is expensive.
- The running costs of a large test facility are high.
- Test results may be criticised due to traffic loading often not incorporating effects such as traffic 'wander', variations of speed and the spectrum of loads found in a field situation.
- The pavement construction may also be criticised as being artificial (too well-controlled by the contractor) and not representative of in-situ conditions.
- Test results are not applicable to sections of pavement having a different construction.

Option C Accelerated trafficking of reduced-scale pavements

As for B, it is assumed that the basic test facility is available.

Advantages

- Testing with this facility takes place under laboratory conditions [4.3] thus allowing careful control of environment, loading, instrumentation and monitoring procedures.
- The accurate measurement of applied load, temperature and deflections made possible in the controlled laboratory environment, lends itself to numerical modelling, more so than full-scale pavements.
- It is less expensive than Options A and B.

Disadvantages

- Criticisms may be made concerning the loading and pavement construction, and how it relates to full-scale pavements. This may be especially true for scale effects, if the intention is to apply test results directly to the field.

Option D Measurement of Component Properties and Numerical Modelling.

This is the most fundamental of approaches and should be used to some degree to compliment the other options given above. It should also be noted that even with a 'fundamental' approach, results need to be calibrated with data from 'live' pavements before they can be applied in practice. This is because it is very difficult to take into account all significant influences likely to affect pavement behaviour.

Advantages
- All testing and monitoring can be strictly controlled giving high quality test results.
- The option is relatively inexpensive if compared to the above options.
- Once a numerical model has been set up, variations in loading and material properties can be made easily, thus reducing the number of tests to be carried out.

Disadvantages
- The development of numerical models can be time consuming.
- Significant amounts of testing are required which may be expensive.
- It is difficult to anticipate, test and quantify details of the influences that may be significant in the field.
- Calibration is needed using at least a reduced-scale pavement under realistic wheel loading.

Considering the most effective balance of approaches and the practical constraints of time and budget, a combination of Options C and D was selected. In particular, the tests named in Table 4.1 were considered to be suitable in providing the necessary information required for modelling, but require calibration with the PTF.

Table 4.1 Selected Test Modes.

Parameter Investigated	Test mode
Rate of cracking	4-point Beam
Reinforcement properties	Dynamic tensile tests
Interface bond properties	Large-scale shear box
Asphalt properties	Nottingham Asphalt Tester (NAT) [4.4].

The numerical model chosen was the CAPA-2D package, a 2-dimensional linear elastic programme incorporating a special crack-propagation routine [4.5]. This programme was chosen mainly to help in the analysis of laboratory beam experiments. It was then intended to apply the information thus derived to full-scale pavements.

As resources were limited it was not possible to test all of the diverse range of grids, fabrics and composites that are commercially available. Activities were therefore focussed on one main product in the following categories:

- Glass-reinforced products, (in particular the Rotaflex products supplied by Chomorat in France),
- Steel grids (supplied by Maccaferri), and
  Polymer products (supplied by Netlon).

In addition, it was decided to test control (unreinforced) samples alongside the reinforced asphalt to give direct comparison.

The overall procedure chosen was as follows:

- First of all, for each of these products, investigation into the material properties was to be carried out (Chapter 5).

- Secondly, products were to be built into asphalt samples for testing in the following manner:

  Beams to investigate crack propagation, (Chapter 7),
  Large (320mm x 200mm) shear-box samples to determine interface bond properties in a dynamic shear mode (Chapter 6).
  A Pilot-scale pavement (4.8m x 2.4m) in the Pavement Test Facility (PTF) (Chapter 8).

- To compliment the above tests, asphalt samples were to be cored from beam, shearbox and PTF pavements to determine asphalt quality in terms of density and stiffness (in the Nottingham Asphalt Tester). Direct tension tests were also to be carried out on cored samples to determine the adhesion between the asphalt and reinforcing product.

- Numerical modelling was to be used to apply test results to field situations. (Chapter 9)

Figure 4.1 illustrates the structure of the proposed investigation.

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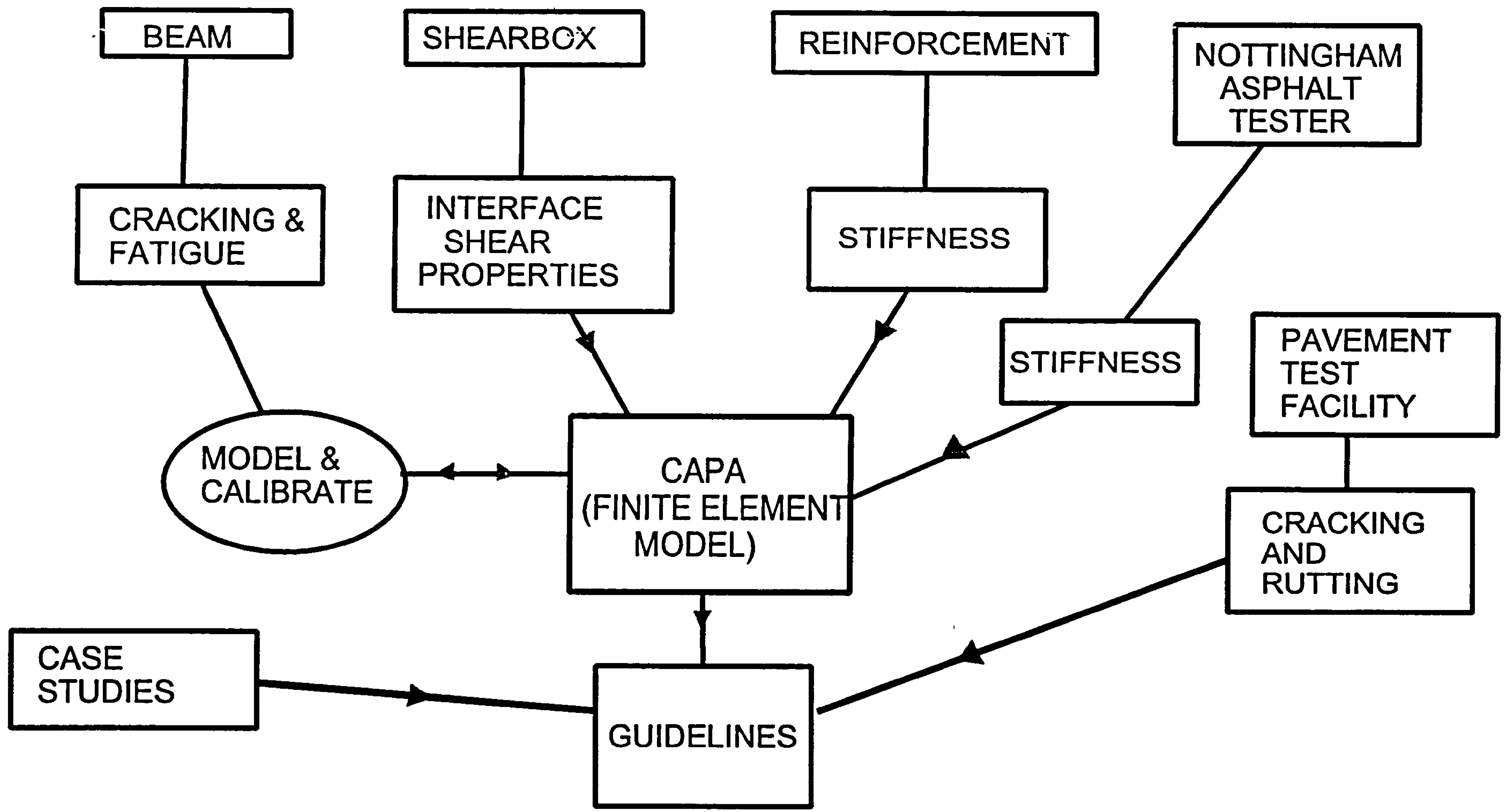


FIGURE 4-1
REINFORCED ASPHALT: PROPOSED INVESTIGATION

CHAPTER 5

TESTS ON REINFORCING MATERIALS

CONTENTS

CHAPTER 5 – TESTS ON REINFORCING MATERIAL

5.1 Introduction

The engineering properties of the reinforcement materials used in Pavement Test Facility (PTF) and beam tests were required to analyse and model test results, using the CAPA-2D Finite Element programme described in Chapter 9 and Reference 5.1. In particular, values of the stiffness modulus are needed for Finite Element Analysis.

Reinforcement materials selected for testing were:

Tensar AR1	a polypropylene grid with apertures of 63mm x 63mm, rated at 20kN strength (transverse and longitudinal).
Tensar AR-G	a Tensar AR1 grid backed with a polyester fabric.
Rotaflex 833	a multi filament PVC coated glass fibre grid backed with a polyester fabric. The glass-fibre grid has dimensions of 26mm x 40mm.
Road-Mesh	a hexagonal double twist galvanised steel wire mesh with an aperture opening of 80mm.

These materials were selected for testing as their properties were to be used in the analysis of the accelerated loading (wheeltracking) Pavement Test Facility (PTF) test. A more detailed description of the materials is given in Appendix 5.A.

5.2 Previous Test Results

Previous testing of grids at Nottingham [5.2] was largely carried out on polypropylene grids supplied by Netlon, where a selection of tests was carried out to investigate ultimate strength, elastic properties, and the susceptibility of these properties to temperature.

Stiffness tests were carried out on samples 430mm long by 380mm wide, giving an average grid stiffness of 0.9MN/m, (or 12.9GPa, if the minimum grid cross-sectional area is used). Grid stiffness was found to reduce with temperature by around 50% for an increase of 40°C. However, it was reasoned that because grid stiffness reduced less than that of the stiffness of the asphalt mixture tested (for the same temperature difference), the grid would still have a beneficial effect, i.e. being stiffer than the asphalt.

The effect on stiffness of placing hot asphalt on grids during paving was also investigated, by exposing grids to high temperatures in both retrained and semi-restrained conditions, and measuring their stiffness. That study concluded that if shrinkage occurs, as is the case where grids are not restrained, stiffness reduces as the magnitude of shrinkage increases. Where the grid was fully restrained, however, the change in stiffness was negligible.

The effect of loading frequency on stiffness was also investigated with tests being carried out at frequencies between 1 and 30 Hz on samples of 500mm x 430mm.

Results showed that stiffness did not alter significantly, although some doubt was expressed at the capabilities of the test apparatus at higher frequencies.

Ultimate strength tests were also carried out on specimens of similar size used for stiffness testing, giving an average ultimate strength of 26kN/m at between 8% and 11% strain. Failure occurred at nodes where, even though the material is thicker, the grid is weakest. The reason for this is that the strength of polypropylene is largely affected by its 'draw ratio' during manufacture i.e. the amount the material is elongated, which serves to orientate the polypropylene structure. As the draw ratio increases, so does the strength and stiffness of the material. At the nodes, material is not elongated, particles remain randomly orientated, resulting in weaker material than the ribs.

Fatigue testing of the material showed that up to at least 500 000 load applications, stiffness remained relatively constant, although the material tended to creep, and exhibited load relaxation, typical of a visco-elastic material.

A summary of the results is given in Table 5.1.

Table 5.1 Summary of Test Results for TENSAR Grids [5.2]

Test Type	Test results
Strength	Average = 26kN/m
Stiffness	0.9MN/m (peak-to-peak measurements of stress and displacements)
Temperature susceptibility	If restrained, stiffness remains constant. If unrestrained, shrinkage occurs, and stiffness reduces.
Fatigue	No change in stiffness up to 500000 cycles.
Frequency susceptibility	Negligible between 1 and 30Hz.

As stated above, to model results from the Pavement Test Facility (PTF) and beam testing, properties of the reinforcement were required. The results given above, provided useful background information, but were not sufficient for input into the CAPA-2D programme.

5.3 Testing - Current Project

As materials in the PTF and beam tests were both subjected to cyclic stresses, it was initially thought appropriate to test samples of material with this type of loading. Also, for convenience, testing in this fashion was hoped provide results that would be readily comparable with the earlier work carried out by Hughes [5.2]. In addition, it was also reasoned that, if relatively large samples were tested, test results should be more applicable for the analysis of PTF data than results on small samples of material would be. Initially, therefore, cyclic tests were carried out on multi-aperture grid samples using an INSTRON universal testing machine (see Figures 5.1 and 5.2). This approach was halted, however, when problems were experienced in applying uniform stresses across the sample. The final test configuration and method used was simpler, consisting of monotonic testing of single strands of material. Richardson and Schiavone [5.3], found that this approach gave more reliable results than multi-strand testing, and was relatively easy to use. As is seen below, this was also the case with these tests which were found to provide data that was both relatively consistent and useful for Finite Element Analysis. To represent the stiiffness of the reinforcement, the Finite Element package CAPA-2D [5.1] requires values of stiffness expressed in units of force per unit strain, i.e. the slope of the force-displacement relationship (providing the gauge length is known). The way in which this measure was obtained is given below.

5.3.1 Test Procedure

To derive values of stiffness, only the applied force and the resultant displacement and gauge length need be measured (see below), which is relatively straightforward. A further advantage of representing stiffness in the form of 'force per unit strain' is that measures of cross-sectional area do not need to be measured. This can be an advantage where representative values for this property can be difficult to obtain due to the nature and geometry of some reinforcement materials:

$$EA = \frac{\sigma}{\varepsilon}A$$

$$EA = \frac{\left[\frac{F}{A}\right]}{\left[\frac{\delta}{L}\right]}A$$

and

$$EA = F\left[\frac{L}{\delta}\right] \text{ i.e. } \frac{F}{\varepsilon}$$

Where

E represents Youngs modulus,
A represents cross-sectional area
ε represents tensile strain
σ represents applied stress
F represents applied load
δ represents extension of the material
and L represents the gauge length of the sample.

In addition to the Finite Element programme requiring stiffness in the form of E*A, an advantage of using this approach is that values of stiffness can be obtained without measures of cross-sectional area, (which can be very difficult to obtain in some of the materials).

5.3.2 Cyclic Testing of multi-aperture samples

Three samples of each material were tested in tension at three different rates. Grids were tested in the Instron test machine using jaws supplied by NETLON. The jaws were designed for gripping these materials and connections were manufactured to connect the jaws to the test machine. The configuration seen in Figure 5.1 was used.

Previous tests on polypropylene grids at Nottingham [5.2] used both controlled strain and controlled stress tests to determine grid properties. However, for the tests carried out in this project, only load control was used, as position control proved to be difficult with the glass-reinforced materials and led to damage to some of the samples. CAPSTAN jaws were supplied by ABG Ltd to enable the testing of composite materials, and were used in the configuration shown in Figure 5.2.

Initially, with the multi-aperture samples, an extensometer was used to measure displacement of the materials. However, trials showed that if the extensometer was mounted in different positions across the samples, different values of displacement were recorded. Also, with composite materials (e.g. AR-G), it was found difficult to mount instrumentation to give repeatable results. To avoid this problem, four LVDTs were mounted on the test jaws (as shown in Figures 5.1 and 5.2) which allowed average measures of displacement across the samples to be calculated.

Dimensions of the samples tested in the Instron machine are given in Table 5.2.

Table 5.2 Details of Grid Samples Tested

Material[1]	Width(mm)	Length (mm)
AR1	263	380
AR-G	300	383
Road-Mesh	300	352

Note 1 Rotaflex was not tested under cyclic loading due the nature of its stress-strain characteristics.

After cutting the samples to size and placing them in the jaws, the spacing between the jaws was adjusted and the grid/fabric samples put under sufficient tensile load to straighten them. A load was then applied to each sample (the 'mean load') around which cyclic loads were applied.

For the first AR-G sample, a series of load magnitudes and frequencies were used to investigate test rate effects before standardising on three test frequencies, i.e. 1, 5 and 10Hz. Twenty seconds of load and deflection data were then recorded for each sample. This procedure was repeated for the other grids giving the test results seen in Table 5.3.

Test results for the Road-Mesh are only given for a test at 10 Hz, as testing was discontinued when it was found that without lateral restraint, the diagonal mesh closed as tensile load was applied. Allied to this, forces in each of the wire strands are very unlikely to be constant across the sample, as twisted wire 'nodes' seem to offer variable resistance as the grid distorts. In a real application, the tendency of the mesh to close up when loaded would apply a degree of confining pressure to material within the aperture and could possibly enhance the crack-suppression qualities of the grids. This should be investigated further, as it may have a significant influence on the grid performance. On consideration of the difficulties in making appropriate measures that represent stiffness of this type of grid, (and possibly all reinforcing products), materials should ideally be tested after being cast within the medium where they are to function in practice (i.e. asphalt). A possible way in which this could be carried out is given in Chapter 13 'Recommendations for future work'.

Table 5.3 Cyclic Testing of Multi-aperture Samples

Sample	Test Frequency (Hz)	Peak to peak values:		EA (kN)
		Strain (%)	kN/Strand	
AR1	1 5 10	1.07 0.33 0.18	0.19 0.10 0.10	18 30 50
AR-G	1 5 10	1.95 1.03 0.57	0.68 0.36 0.18	35 35 32
Road-Mesh	10	0.18	0.1	50

Given that the AR1 and the AR-G grid components are similar, the variation in test results is considered surprising. The apparent stiffening of the AR1 grid with frequency, compared with the relatively constant values of AR-G at all test frequencies do not appear compatible.

5.3.3 Monotonic testing of single strands

As stated above, following the difficulties experienced with testing multi-aperture samples, and the difficulty of measuring the cross-sectional area of the materials tested, (whether they were grids or composites), an alternative approach was required. The method needed to be relatively simple to carry out, but also be capable of giving reliable results that could be easily interpreted. The testing of single strands (or strips in the case of composite materials), rather than larger, multi-aperture samples, was therefore undertaken, using the same equipment as shown in Figures 5.1 and 5.2. For each material, 3 tests at test rates of 0.5, 5.0 and 50.0 mm/minute extension were used, and samples were tested to destruction, to compare failure loads with manufacturers' data.

5.4 Test Results

Figures 5.3 to 5.5 show plots of load versus deflection for AR1, AR-G and ROTAFLEX 833 samples, and Figures 5.6 to 5.11 show the relationship between the test rate and stiffness EA, and the test rate and failure load. Road-Mesh samples were not tested as single strands as it was not thought appropriate, due to the grid configuration - i.e. the interaction of the angled strands 'twisted' or woven together make it difficult to model with CAPA-2D. As pointed out above, to obtain more 'usable' parameters, it would appear likely that testing grids cast in asphalt would yield more appropriate results.

A summary of the test results is given in Table 5.4.

Table 5.4. Test Results (average of 3 test results)

Material	Test Rate (mm/minute)	Failure Load (N/strand)	Failure Load (kN/m)	Failure Strain (%)	EA per strand (kN) Average	EA per strand (kN) Range
AR1	0.5	1113	17.1	11.1	25.0	17-33
	5.0	1421	22.0	12.7	27.2	24-33
	50.0	1451	22.5	9.3	29.8	28-31
AR-G	0.5	1051	16.3	13.6	20.2	17-23
	5.0	1134	17.6	12.5	25.7	24-27
	50.0	1220	18.9	12.1	27.9	27-28
ROTAFLEX 833	0.5	729	28.4[1] 18.2[2]	2.6[3]	13.5	11-14
	5.0	691	26.9[1] 17.3[2]	2.7[3]	14.1	12-15
	50.0	772	30.1[1] 19.3[2]	3.2[3]	13.2	9-15

Notes: 1 Transverse direction
2 Longitudinal direction
3 Corrected strain values-obtained by resetting the zero strain reading by extending the steepest gradient to the x-axis.

5.4.1 AR1 Test Results

Figure 5.3 shows test results of 9 samples, all of which show non-linear behaviour, and roughly fall into three groups, according to the test rate. Tests carried out at 50mm/minute tend to give higher values of stiffness than do test results from tests carried out at 0.5mm/minute, which is consistent with the effects of visco-elasticity. The plot that appears different to the others (i.e. sample AR1e) shows a relatively flat portion near the beginning of the test curve which was due to some movement of the grid in the jaws. With translation, however, the plot fits the other curves well. Failure of the grids occurs typically between 8 and 12% strain which agrees with other test findings,[5.4] and manufacturers specifications and quality control measurements (see the Appendix). Figures 5.6 and 5.7 both show a tendency for values of both stiffness and strength to increase with test rate, suggesting that care would need to be taken if the grids were to be used to reinforce asphalt under sustained loading, such as for a lorry park for instance.

Ultimate strengths of the samples tend to agree with manufacturers rating, i.e. 20kN/metre, although samples tested at 0.5mm/min tend to be low, giving an average of around 17kN/m.

5.4.2 AR-G Test Results

Test results (see Figure 5.4) generally appear to be consistant, except for samples AR-G8 and AR-G9. These samples differ from the rest in that AR-G 8 initially shows a steeper gradient than the rest, and AR-G 9 shows a shallower gradient than the others up to around 3% strain, and at failure (around 13% strain). It is not certain what the reason for the differences was although there did appear to be some distortion of one of the nodes in the clamp during the AR-G 9 test. The initial low gradient section of the plots could be due to some compression of the fabric and the welded bond between the grid and fabric at the jaw. It is also noted that these tests correspond to the slowest test rate. It is not known, however, if these factors are linked to the test results. More samples should be tested to investigate the possible effects of clamping.

To calculate values of EA, the steepest part of the curve was taken, as it is reasoned that when placed, the reinforcement would be totally confined and have no initial 'play', as exhibited in the tests. This assumption needs to be investigated further, as if the reinforcement requires a strain of around 3% before its effect becomes significant, then the bituminous materials would be strained excessively and would be likely to fail prematurely.

Material strength test results are more than 20% lower than for AR1 results, which suggests that (a) more samples should be taken to confirm this apparent discrepancy, and (b) if there is a significant difference between AR1 and AR-G, then more investigation is required, possibly in the technique used to bond the fabric to the grid.

5.4.3 ROTAFLEX 833 Test Results

These test results (see Figure 5.5) differ from the AR1and AR-G polypropylene curves, and have curves that tend to be concave, i.e. stiffening with load. This was due to the material gradually stiffening as it tightened around the CAPSTAN jaw. Also, failure of the samples appears in some points to occur in stages, with a peak followed by a trough, followed by a second higher peak, (see for example sample 833-E in Figure 5.5). This is attributed to the progressive brittle failure of glass filaments, i.e. not all filaments being stressed to the same degree at the same time. The second peak occurred after the remaining glass filaments slipped slightly and then tightened up.

The strength test results appear low, i.e. around half or less of the manufacturers' specifications of 35kN/m for warp and 70kN/m for the weft. De Bondt [4], carried out similar tests on a range of materials, and obtained similar values for ROTATEX WG2303, which is the grid component of the ROTAFLEX 833 composite. De Bondt attributed the unexpectedly low values to unrealistic estimations of the material's capability by the manufacturer, who computes the strength of the strand by summing the individual strengths of the glass filaments, assuming all filaments fail simultaneously. This was seen not be the case as fiaments could just be seen breaking by the naked eye, and were quite obvious with the aid of a magnifying glass. It follows that to confirm or deny these limited results additional testing should be done on other rolls of the material. It is of couse possible that the material supplied was uncharacteristically weak through a manufacturing flaw, or had been damaged in transit, perhaps. There was however, no obvious sign that damage had occurred.

Further investigation of these grids is required, especially the nature of the plastic covering of the filaments and the interaction between filaments under load.

5.4.4 General comments on test curves

Test results show two characteristic shapes, namely, (a) the polypropylene curve with gradients gradually reducing with strain, and (b) the gradient of the curve for the rotaflex tending to increase as strain increases up to failure. The reason for the latter seems to be the effect of the material becoming more tightly wound around the capstan jaws as load increases and the plastic sheathing of the glass filaments becoming more compressed.

There is a notable difference too in the manner of failure, i.e. as the polypropylene breaks, the load reduces to zero quickly, whereas the ROTAFLEX fails more gradually, exhibiting the more 'saw-toothed' appearance seen on the graph. Results for the ROTAFLEX materials, when scaled-up for metre widths, are lower than those cited by the manufacturer.

5.5 Concluding Comments

Tests have been carried out on polypropylene, glass and steel reinforcement to obtain material properties appropriate for analysis of beam and Pavement Test Facility results. After trials with cyclic and monotonic test configurations, a configuration testing single strands with monotonic tensile loads was used. This was due to the cyclic-loading tests on larger multi-aperture samples being difficult to analyse due to a variation of stress across the samples. Steel grids were not tested in the apparatus after trials with samples of mesh showed the apertures to close-up when load was applied, thus giving very high extensions.

Nine samples from three types of reinforcement were tested monotonically, i.e. three samples at each of three test rates.

A summary of test results is given in Figures 5.6 to 5.11 and Table 5.3 and show stiffness values and failure loads obtained at three loading rates. For these rates, results are similar to those obtained by de Bondt [5.4] and agree in general with manufacturers recommendations, although for the ROTAFLEX 833 composite, strengths and stiffnesses appear low. This was attributed to the nature of failure, i.e. progressive brittle failure of the glass filaments making up the reinforcing bars.

Representative test results could not be obtained for the steel Road-Mesh grid due to the configuration of the grid. In particular the hexagon-shaped apertures and the twisted wire connections means that the Finite Element modelling package used for analysis (CAPA-2D) would be difficult to apply. To obtain representative values it is thought that this grid needs to be tested after being cast into asphalt. General suggestions of how this could be carried out are given in Chapter 13 but the approach will require more development, especially in detailing how the reinforcement is gripped by jaws, and what configuration of instrumentation is required to measure displacement.

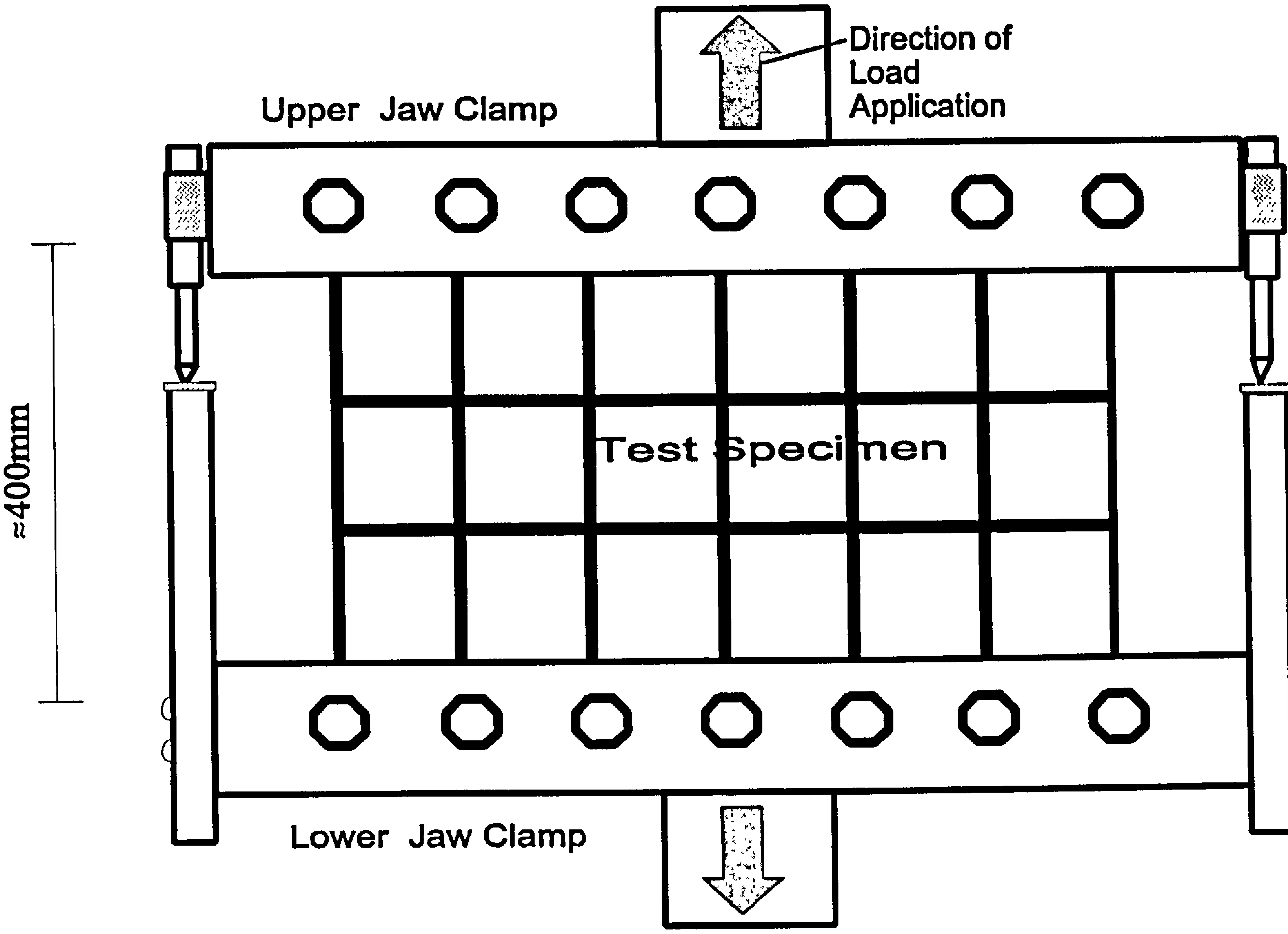
5.6 References

5.1 Scarpas, A De Bondt, A H , Molenaar, A A A and Gaarkeuken, G (1996). Finite Element Modelling of Cracking in Pavements. Proceedings of the Third International RILEM Conference on Reflective Cracking in Pavements: Design and Performance of Overlay Systems. Maastricht, Holland, .

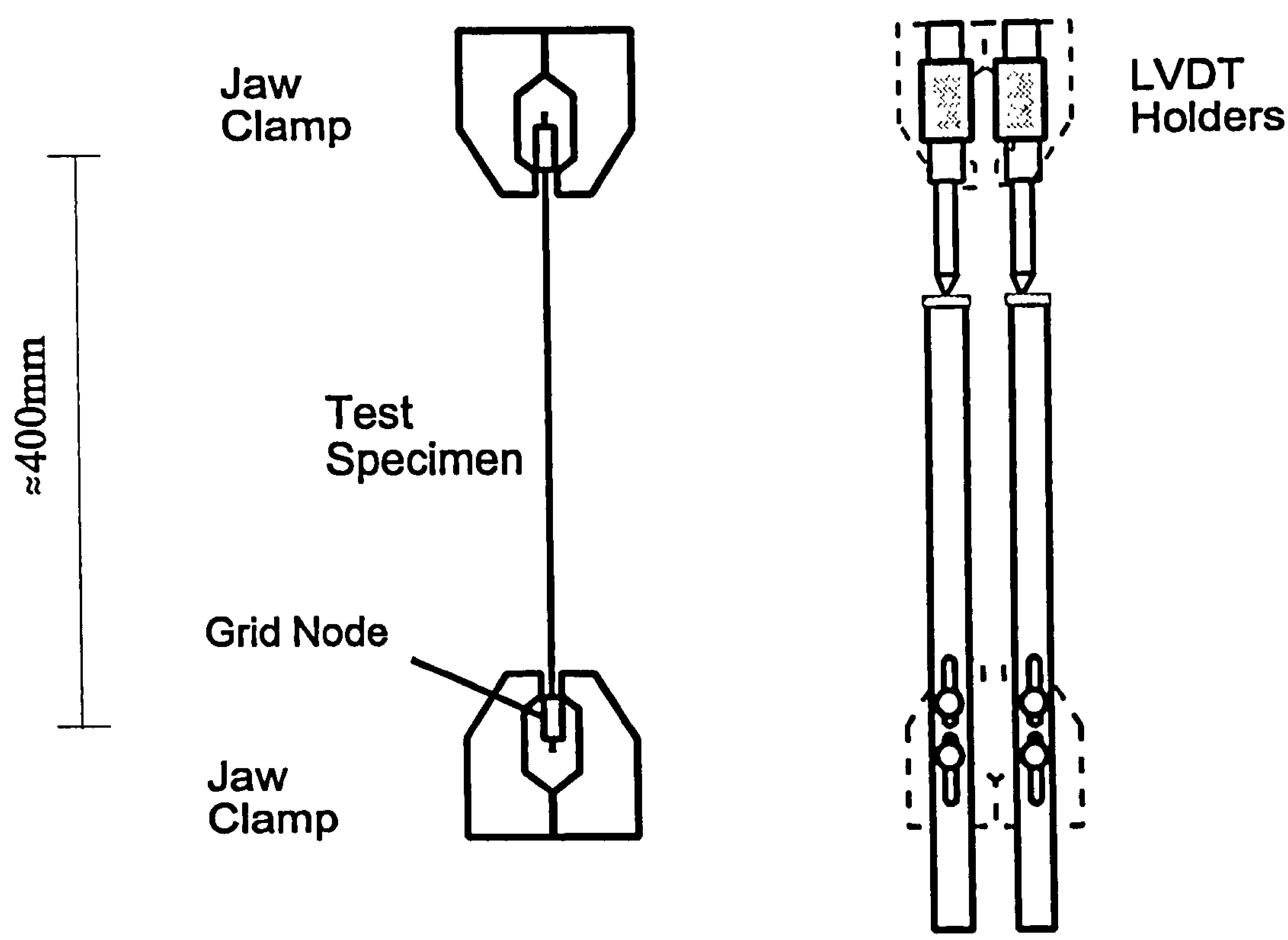
5.2 Hughes, D A B, (1986). Polymer Grid Reinforcement of Asphalt Pavements. PhD Thesis, Department of Civil Engineering, University of Nottingham.

5.3 Richardson, G.N. and Schiavone, P.E. (1995). Investigation of the Tensile Properties of Fibreglass based Reinforcements. Civilsynthetic Engineering, Raleigh, North Carolina, USA.

5.4 De Bondt, A (1999). Anti-Reflective Cracking Design of (Reinforced) Asphaltic Overlays. PhD Thesis, Department of Civil Engineering, University of Delft, The Netherlands.



(a) General layout



(b) Side Elevation : Jaws and LVDT holders

Figure 5.1
Grid Testing Configuration

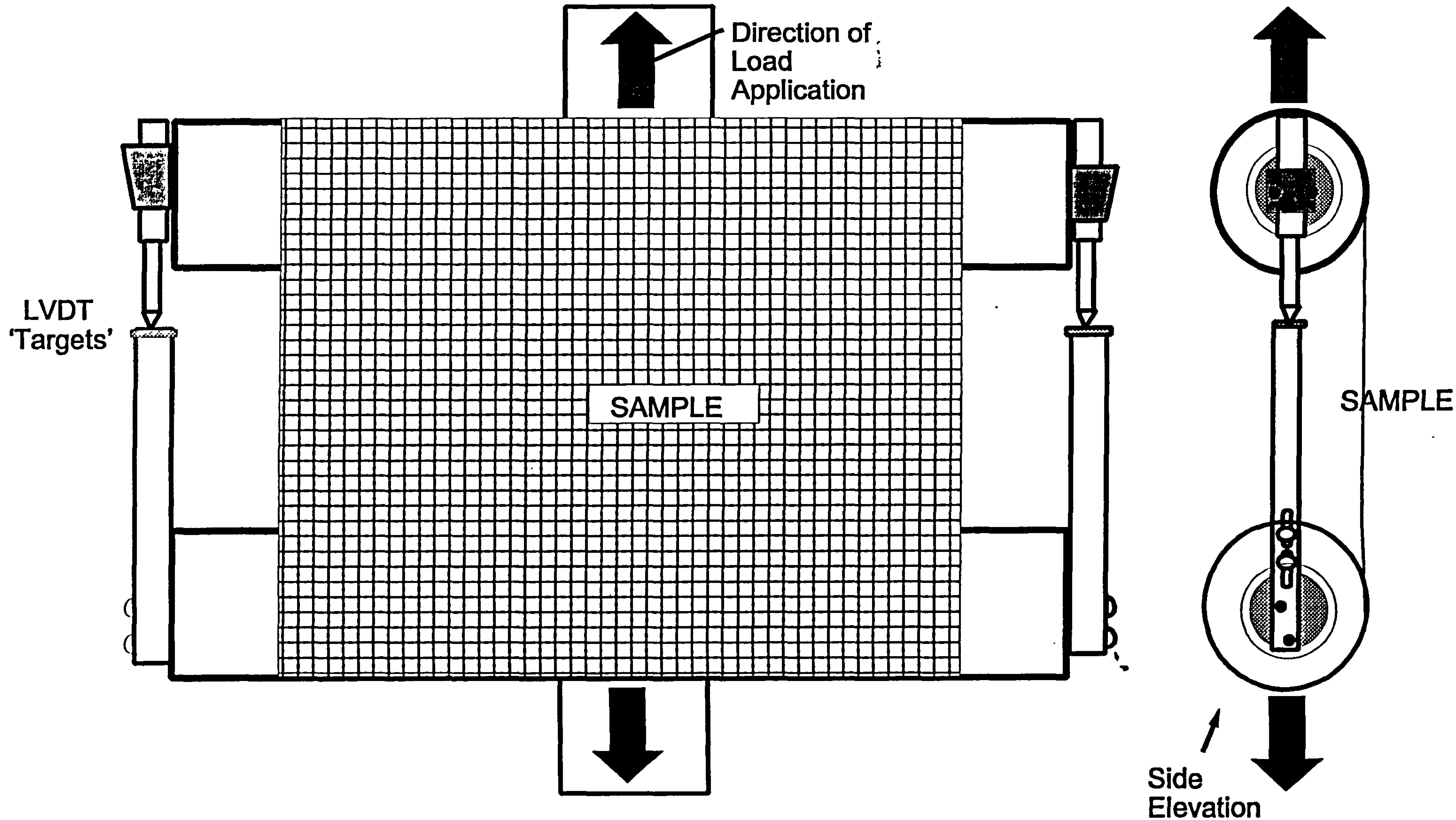


FIGURE 5.2

COMPOSITE REINFORCEMENT TESTING:
GENERAL ARRANGEMENT

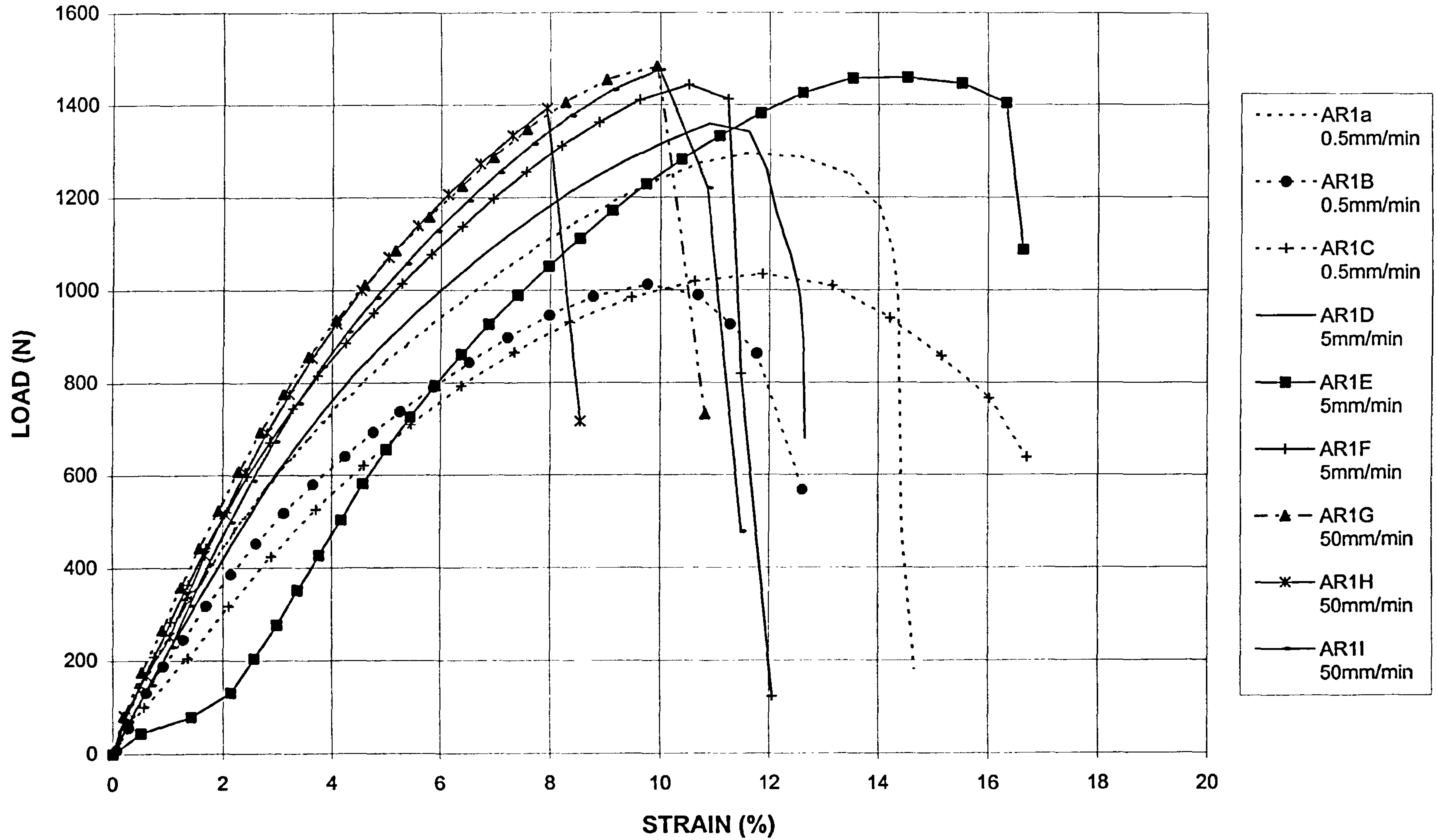


FIGURE 5.3

REINFORCEMENT TESTING: AR1 SINGLE STRAND

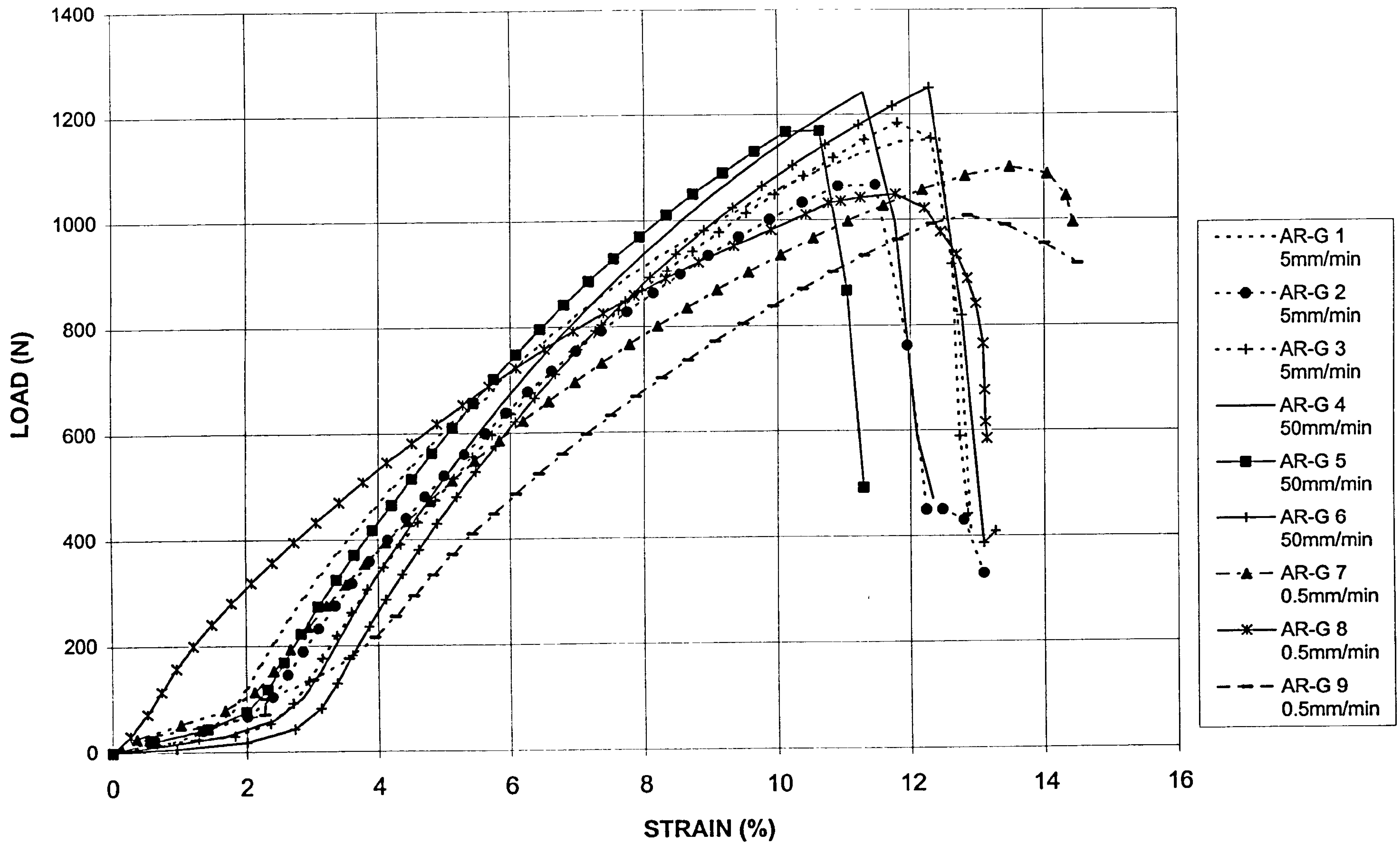


FIGURE 5.4

REINFORCEMENT TESTING: AR-G SINGLE STRAND

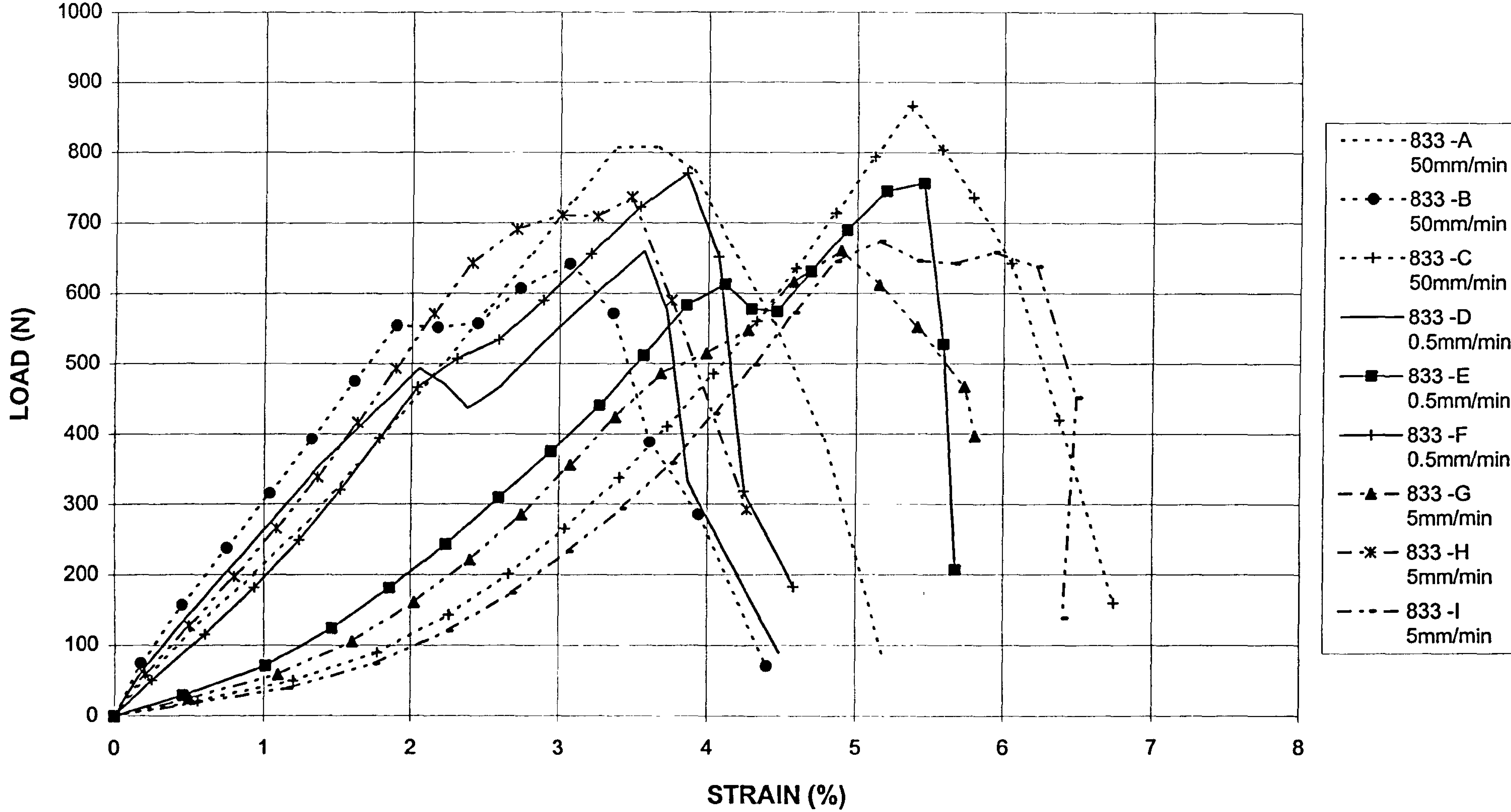


FIGURE 5.5

REINFORCEMENT TESTING: ROTAFLEX 833 SINGLE STRAND

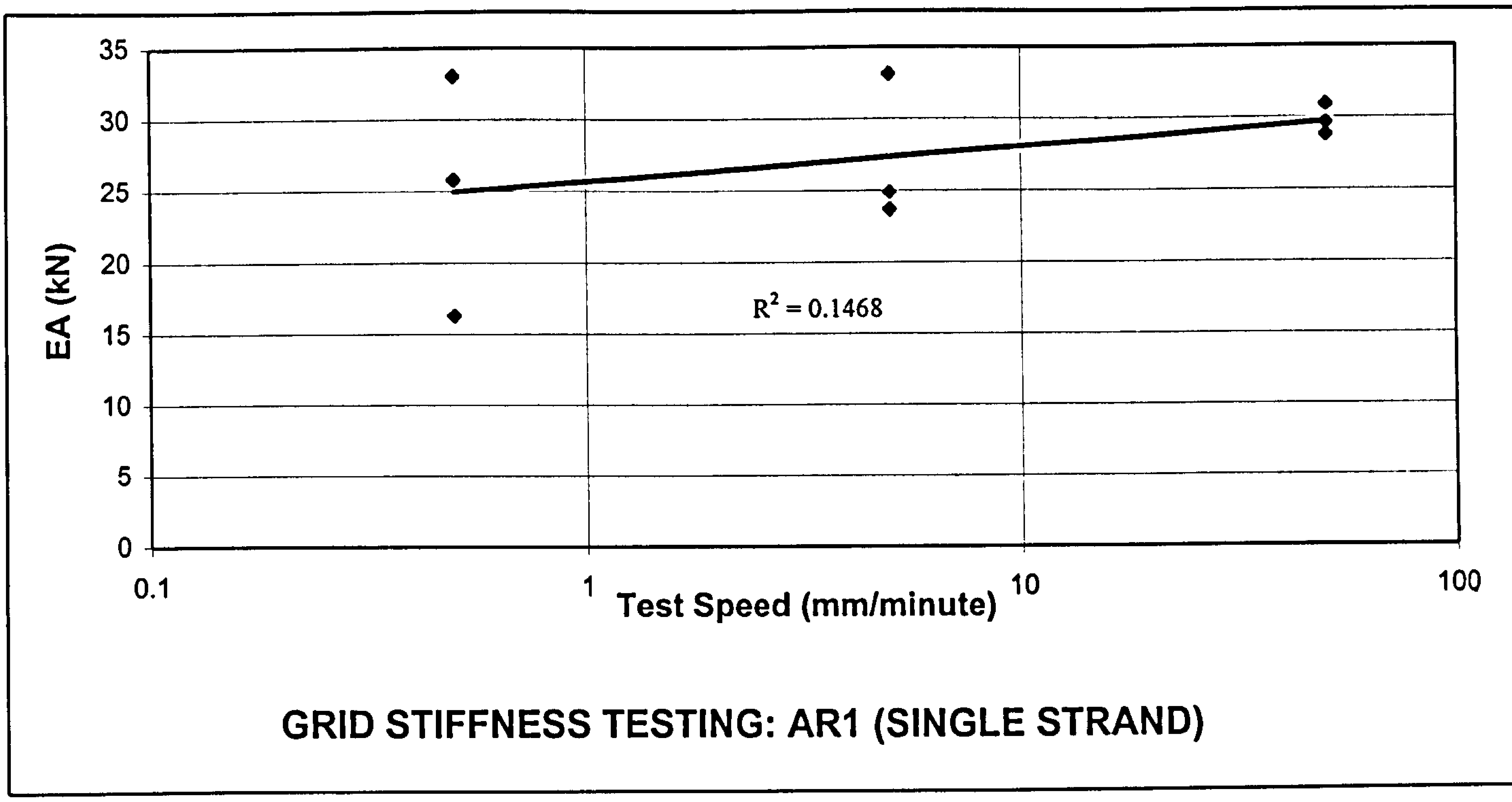


FIGURE 5.6

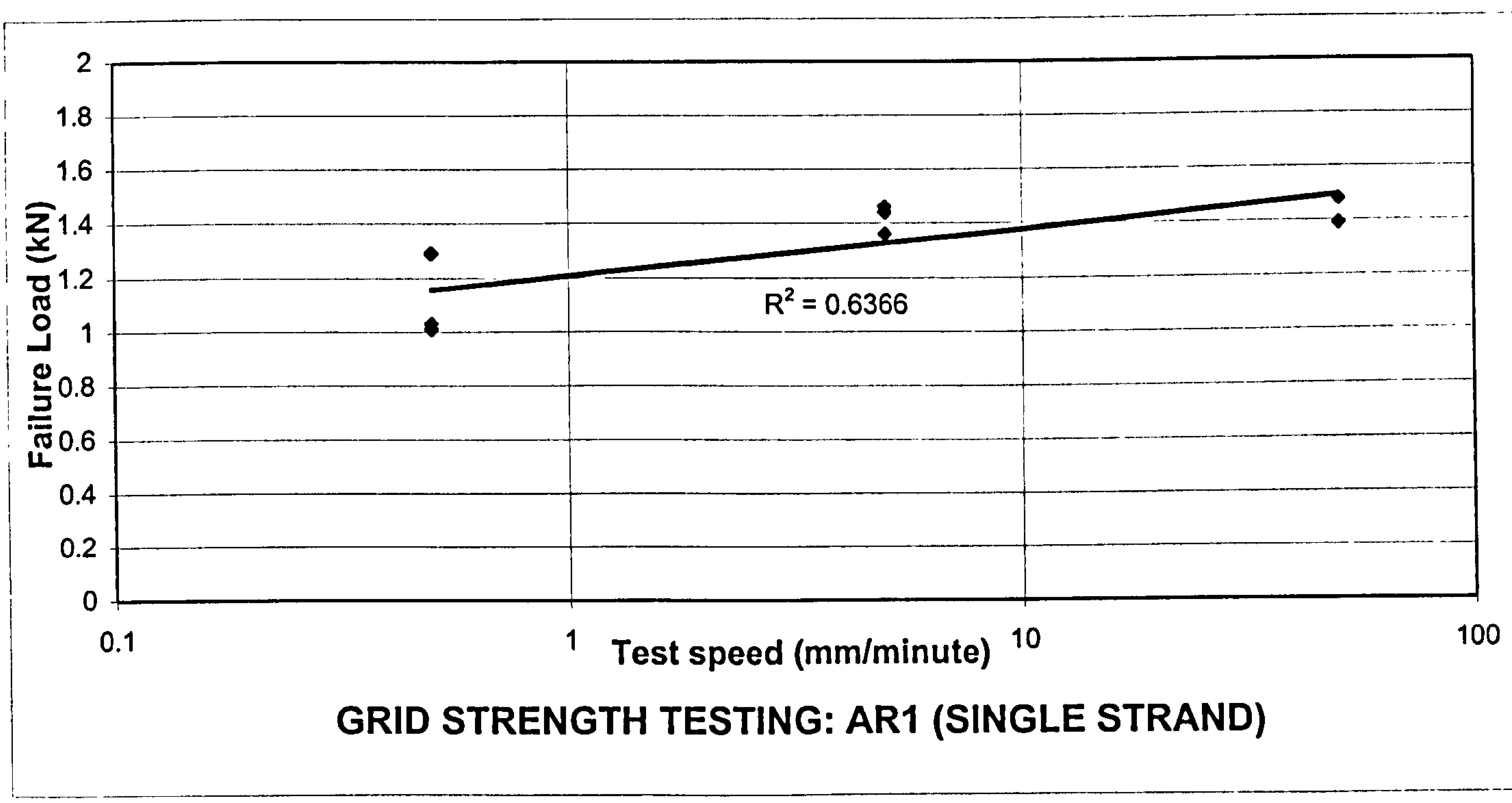


FIGURE 5.7

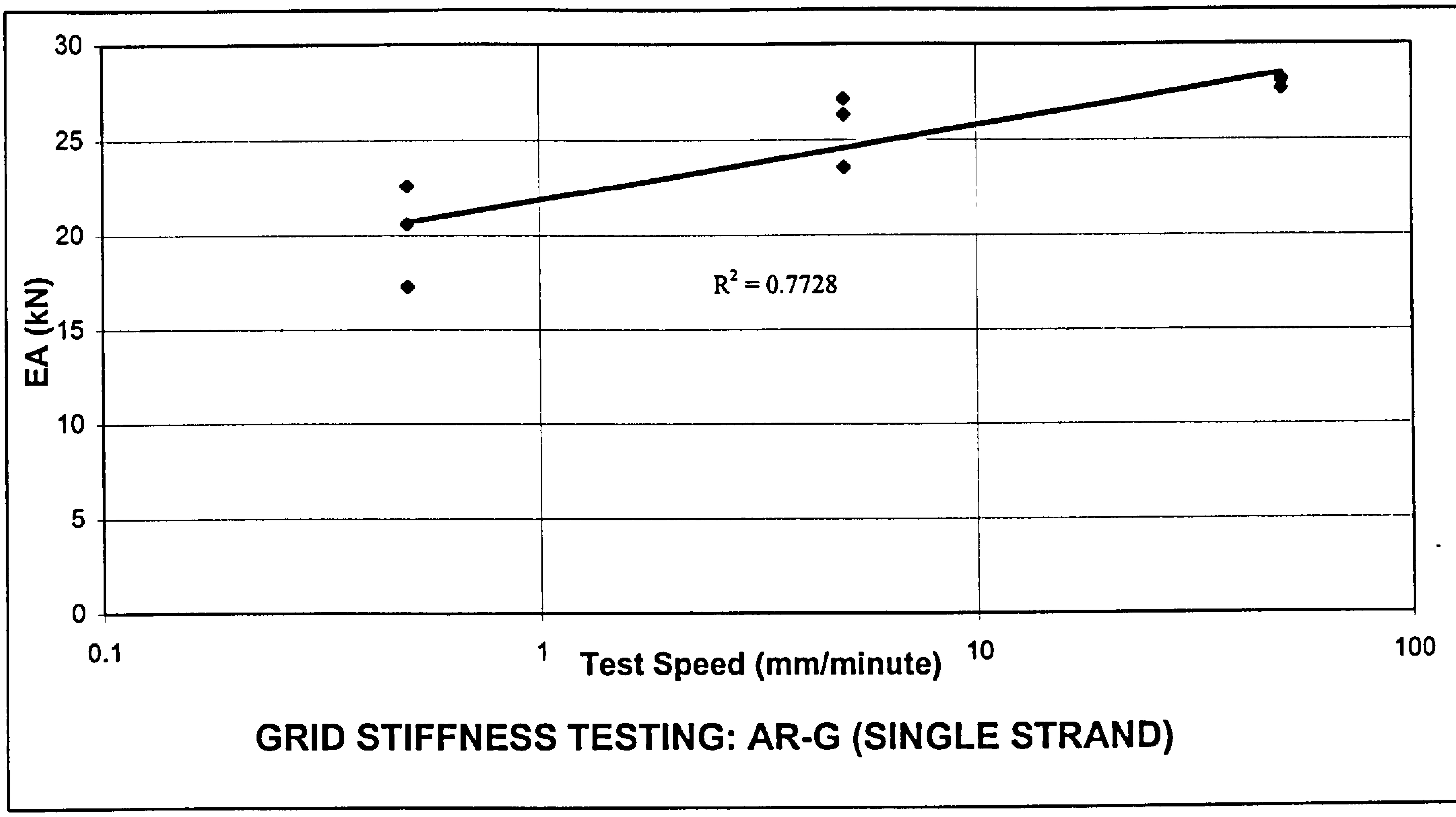


FIGURE 5.8

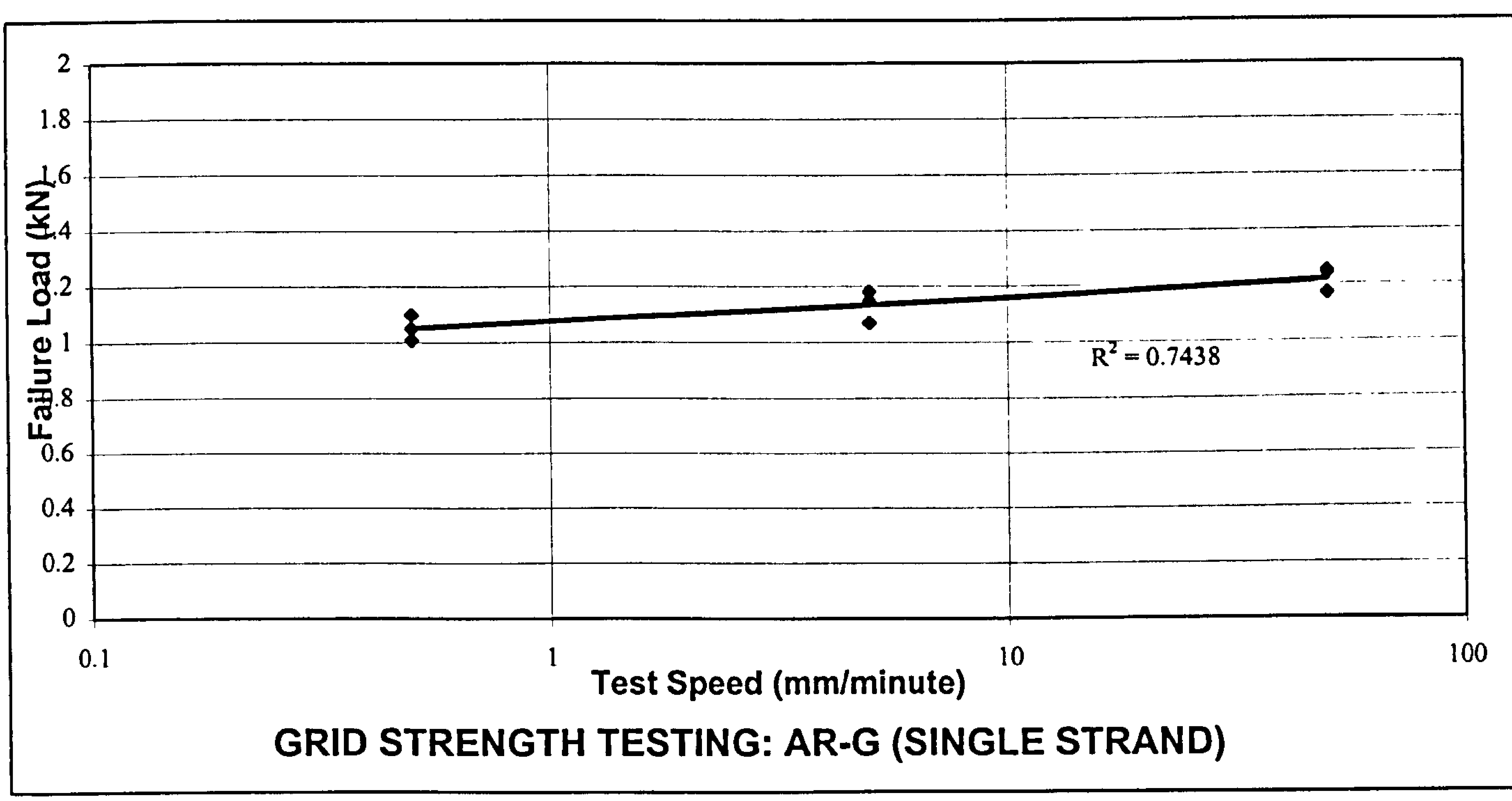


FIGURE 5.9

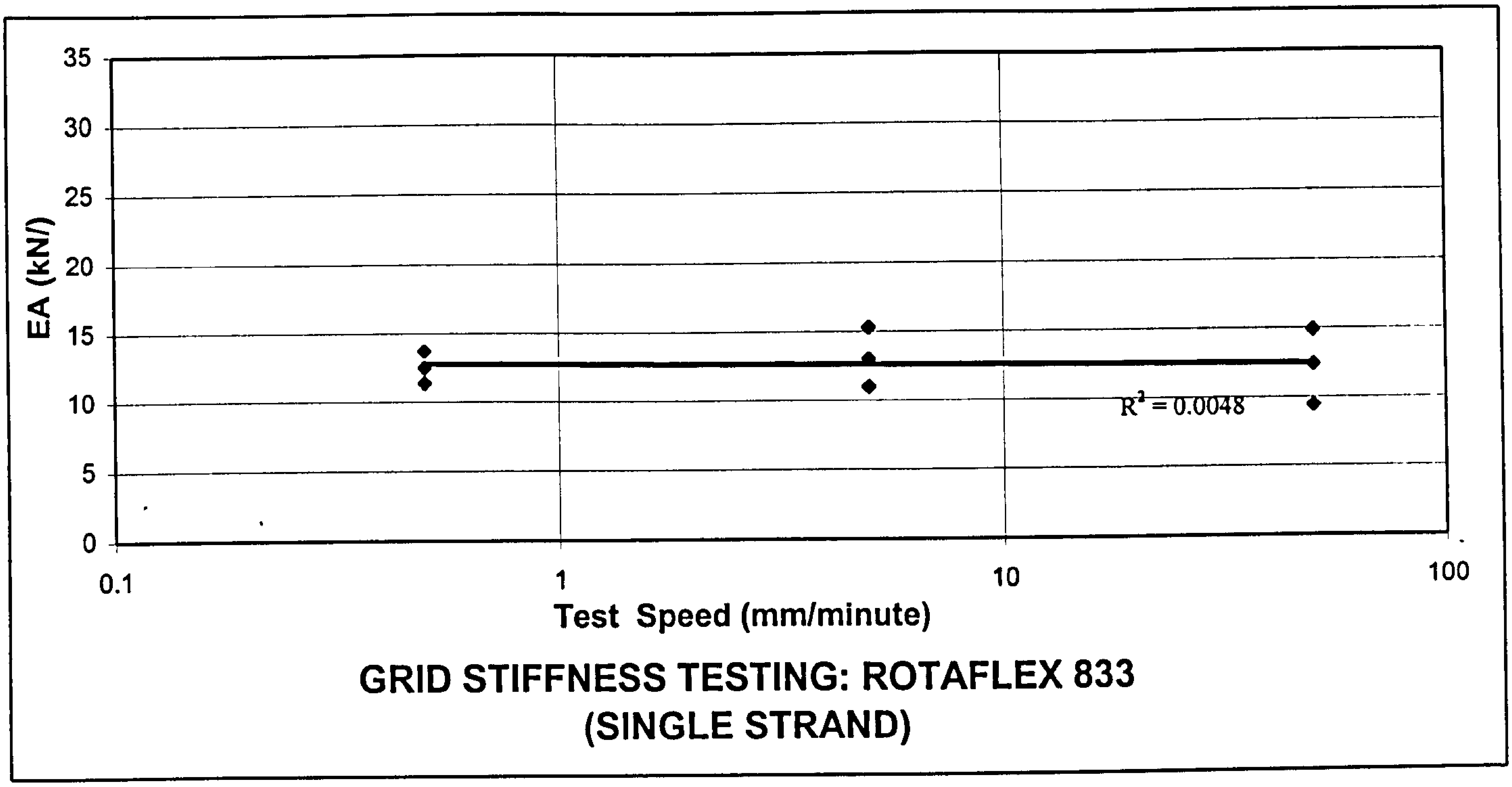


FIGURE 5.10

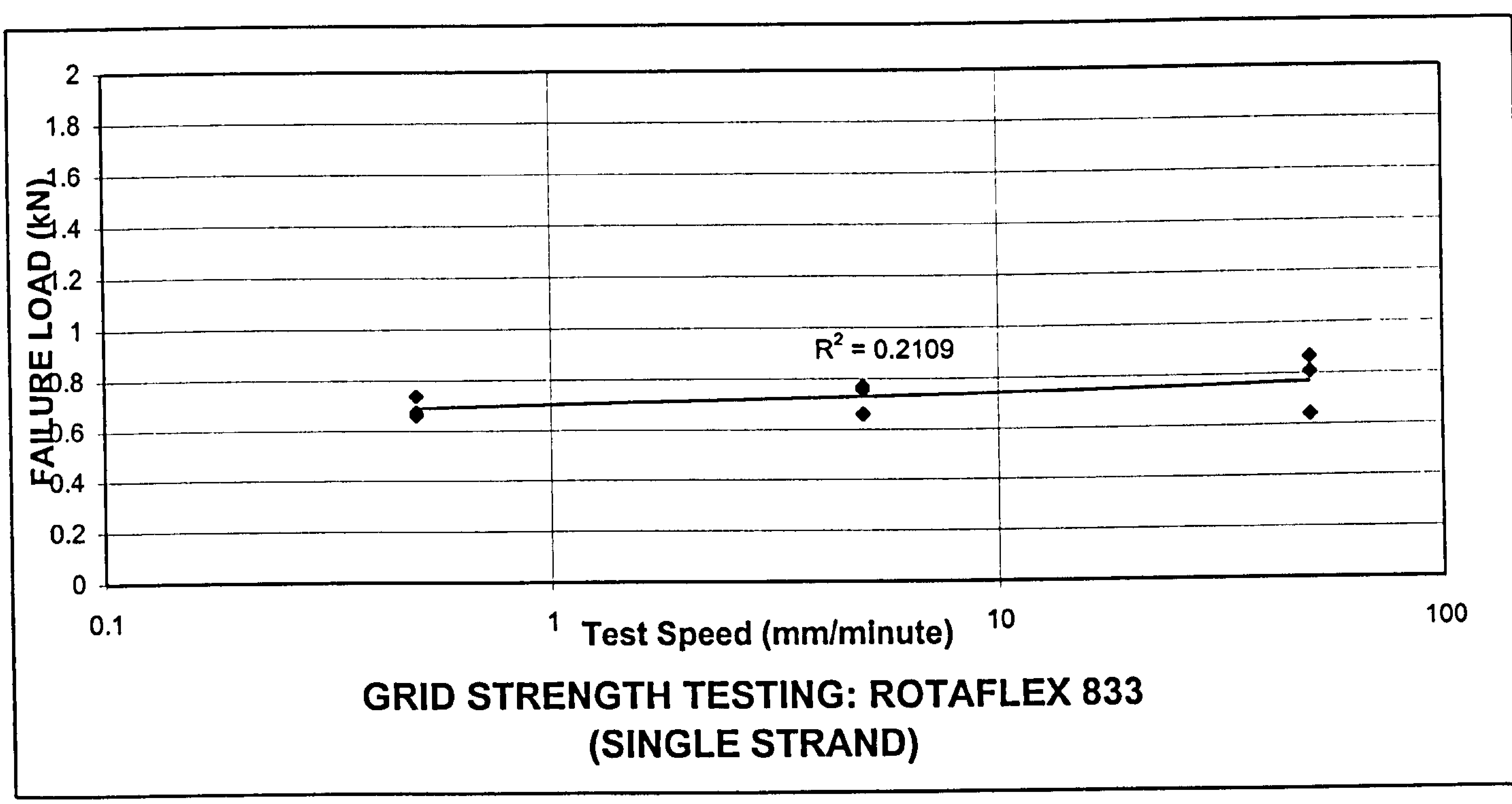
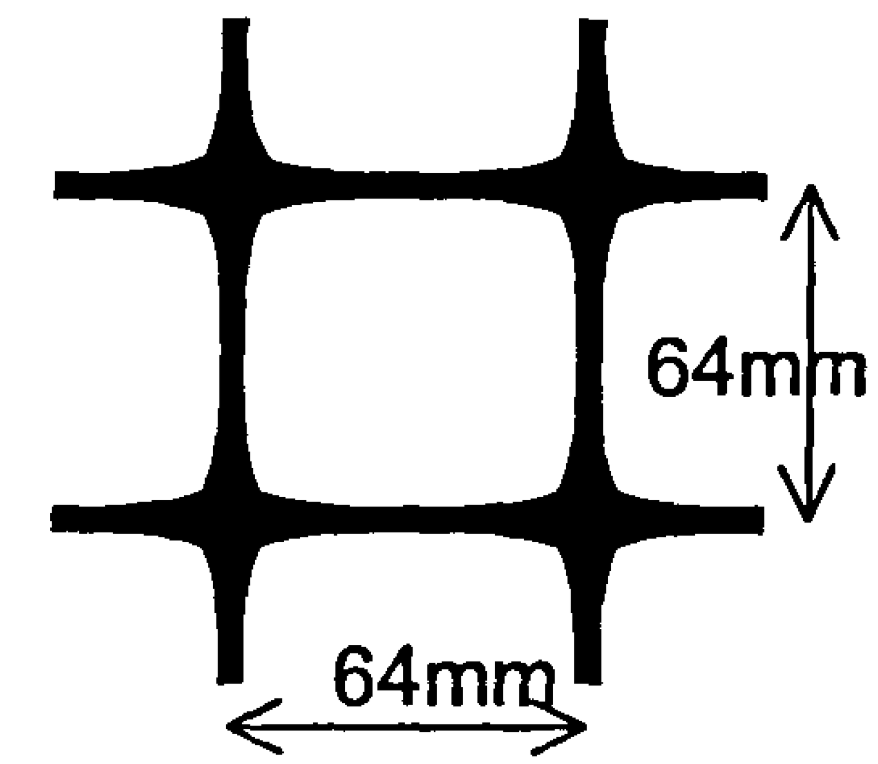
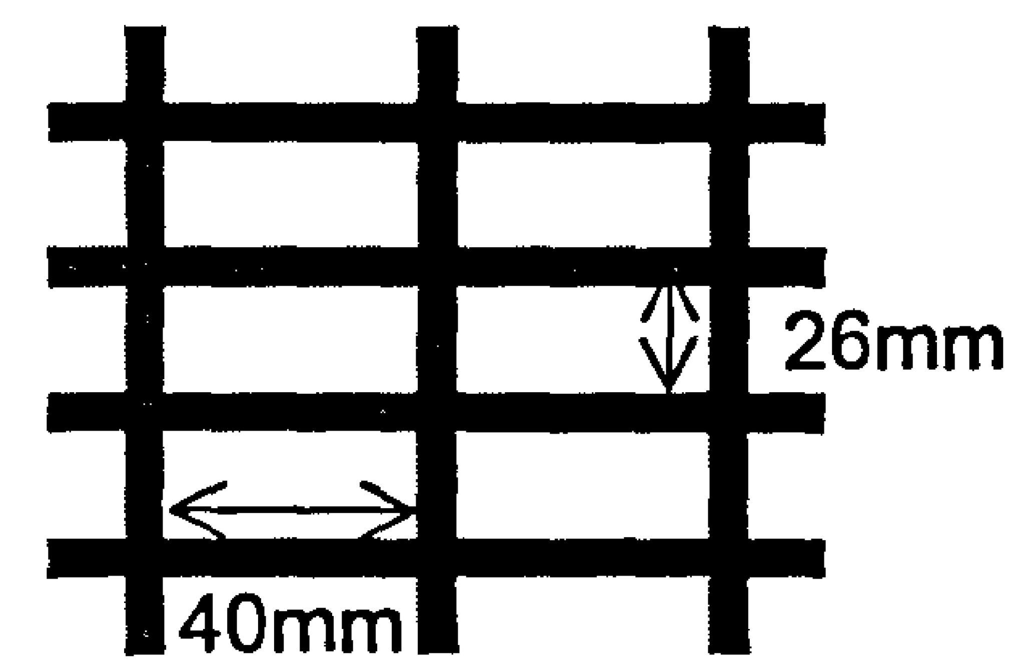


FIGURE 5.11

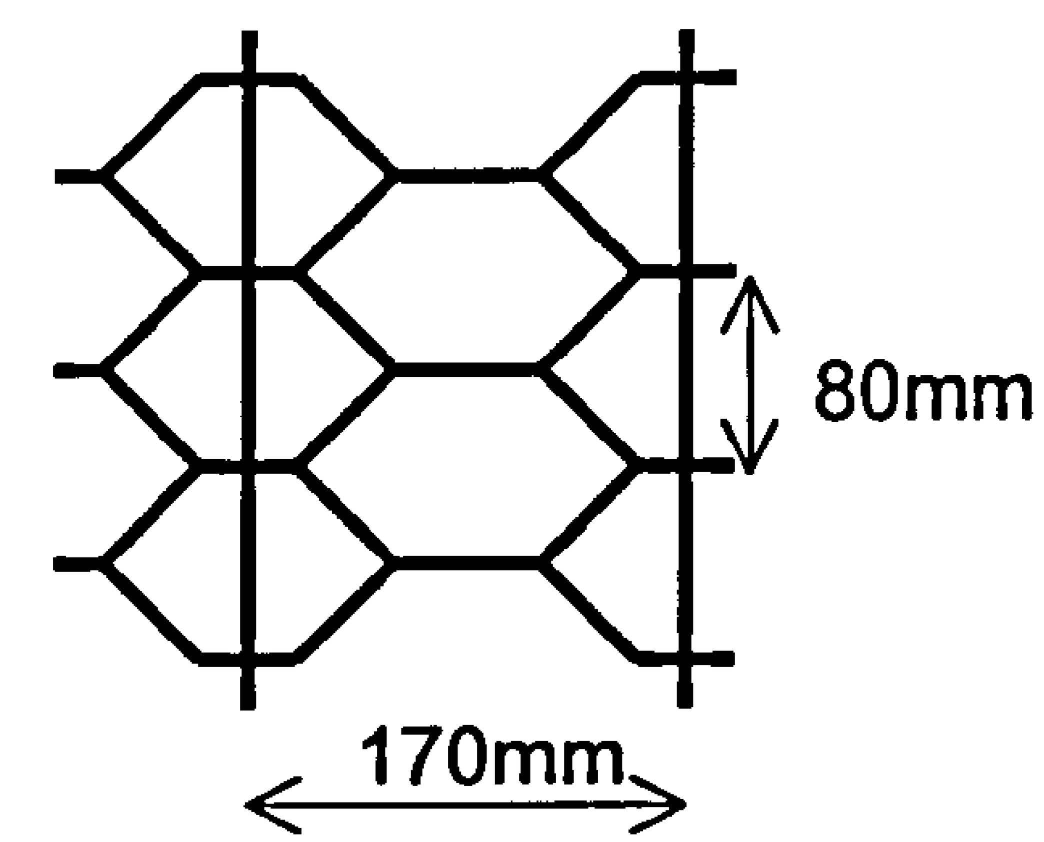
APPENDIX 5-A Details of Reinforcement Properties (Data from Manufacturers)

Table 5A-1: Summary of Reinforcement Properties

Material Type	Aperture size (mm)	Strength (kN/m)	Geometry
AR1: Polypropylene grid	64 x 64	20	
AR-G: Polypropylene grid and polypropylene/polyester geotextile	64 x 64	20	
Rotatex WG2303: Multifilament glass fibre in PVC coating	26 x 40	35^1 $(70)^2$	
Rotatex 833: Multifilament glass glass fibre in PVC coating plus polyester geotextile. Strand dimensions: 4mm by 1.5mm	26 x 40	35^1 $(70)^2$	

Notes: 1. Warp 2. Weft

Table 5A-1: Summary of Reinforcement Properties (continued)

Material Type	Aperture size (mm)	Strength (kN/m)	Geometry
Road-Mesh: twisted galvanised steel mesh Mesh strands 2.7mm diameter Transverse reinforcement 3.4mm diameter	80[3]	43[4] 45-185[5]	

Notes:
3 Minimum dimension across hexagon aperture
4 Longitudinal strength
5 Transverse strength

APPENDIX 5B

GRID MANUFACTURERS QUALITY CONTROL RESULTS:-AR1

Results from tests carried out on material from the same batch as supplied for the test programme are given:

NETLON LTD
TENSAR DIVISION
BLACKBURN LANCS

*3.8M AR1 BS EN ISO 10319 LD
SAMPLE SIZE 5 x 4 RIBS
TEST SPEED 40mm/min

Test type:	Tensile	Instron Corporation	
Operator name:	D.S.MARSDEN	Series IX Automated Materials Testing System	7.27.01
Sample Identification:	NUARLD	Test Date: Monday, September 02, 1996	
Interface Type:	4200		

Sample Rate (pts/secs): 9.1000
Crosshead Speed: 40.0000 mm/min
2nd Crosshead Speed: 0.0000 mm/min

Humidity (%): 50
Temperature: 20 C

Full Scale Load Range: 150000000.0000

BATCH NUMBER : NOTTINGHAM UNIVERSITY

ROLL NUMBER : EPSRC RESEARCH

SAMPLE WIDTH : 3.820

SAMPLE WEIGHT : 0.879

MIN RIBS/MTR T.D : 14.9

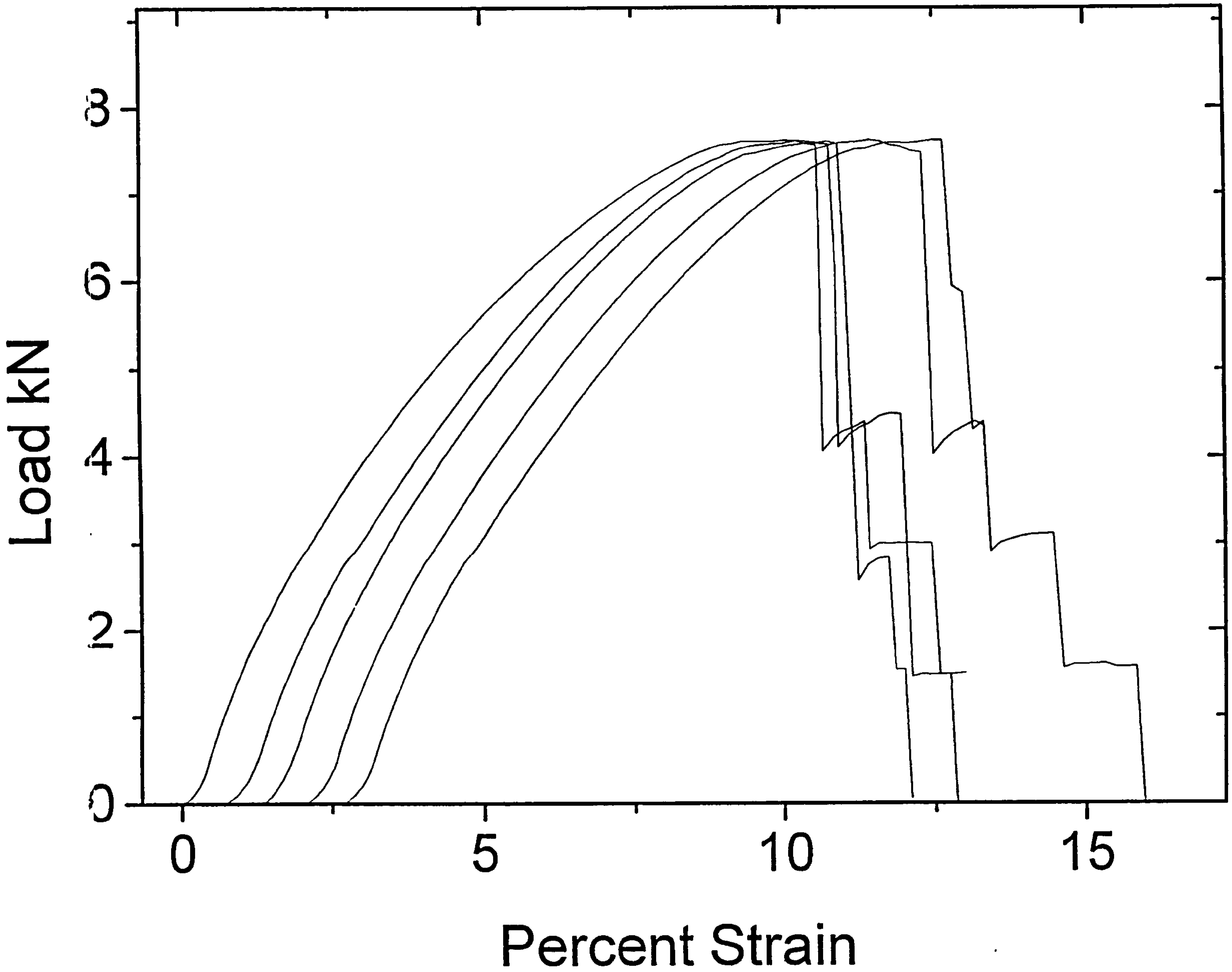
Sample comments:

Custom Test Label:

	Displcment at Max.Load (mm)	Load at Max.Load (KN)	% Strain at Max.Load (%)	MEAN STRENGTH (KN/M)	LOAD AT 2% (KN/M)	LOAD AT 5% (KN/M)
1	19.594	7.623	10.143	22.717	8.743	16.904
2	18.548	7.627	9.603	22.728	8.293	16.714
3	17.783	7.590	9.213	22.618	8.528	17.169
4	18.158	7.627	9.400	22.728	8.421	16.937
5	19.283	7.631	9.989	22.740	8.284	16.603
Mean	18.673	7.620	9.670	22.706	8.454	16.865
S.D.	0.757	0.017	0.391	0.050	0.191	0.218
Mean +0.58 SD	19.112	7.629	9.897	22.735	8.564	16.992
Mean -0.58 SD	18.234	7.610	9.443	22.677	8.343	16.739

Sample ID: NUARLD

All specimens.



3.8M AR1LD (STANDARD QC TEST)
SINGLE RIB QC TEST
TEST SPEED 100 mm/min

Test type: Tensile
Operator name: F KENYON
Sample Identification: NUILD
Interface Type: 1120

Instron Corporation
Series IX Automated Materials Testing System 7.27.01
Test Date: 27 August 1996

Sample Rate (pts/secs): 9.1032
Crosshead Speed: 100.0000 mm/min
2nd Crosshead Speed: 0.0000 mm/min

Humidity (%): 50
Temperature: 20 C

Full Scale Load Range: 2000000.0000N

BATCH NUMBER : NOTTINGHAM UNIVERSITY

ROLL NUMBER : EPSRC RESEARCH

SAMPLE WIDTH : 3.820 MTRS

SAMPLE WEIGHT : 0.879 KGS

MIN RIBS/MTR T.D : 14.9

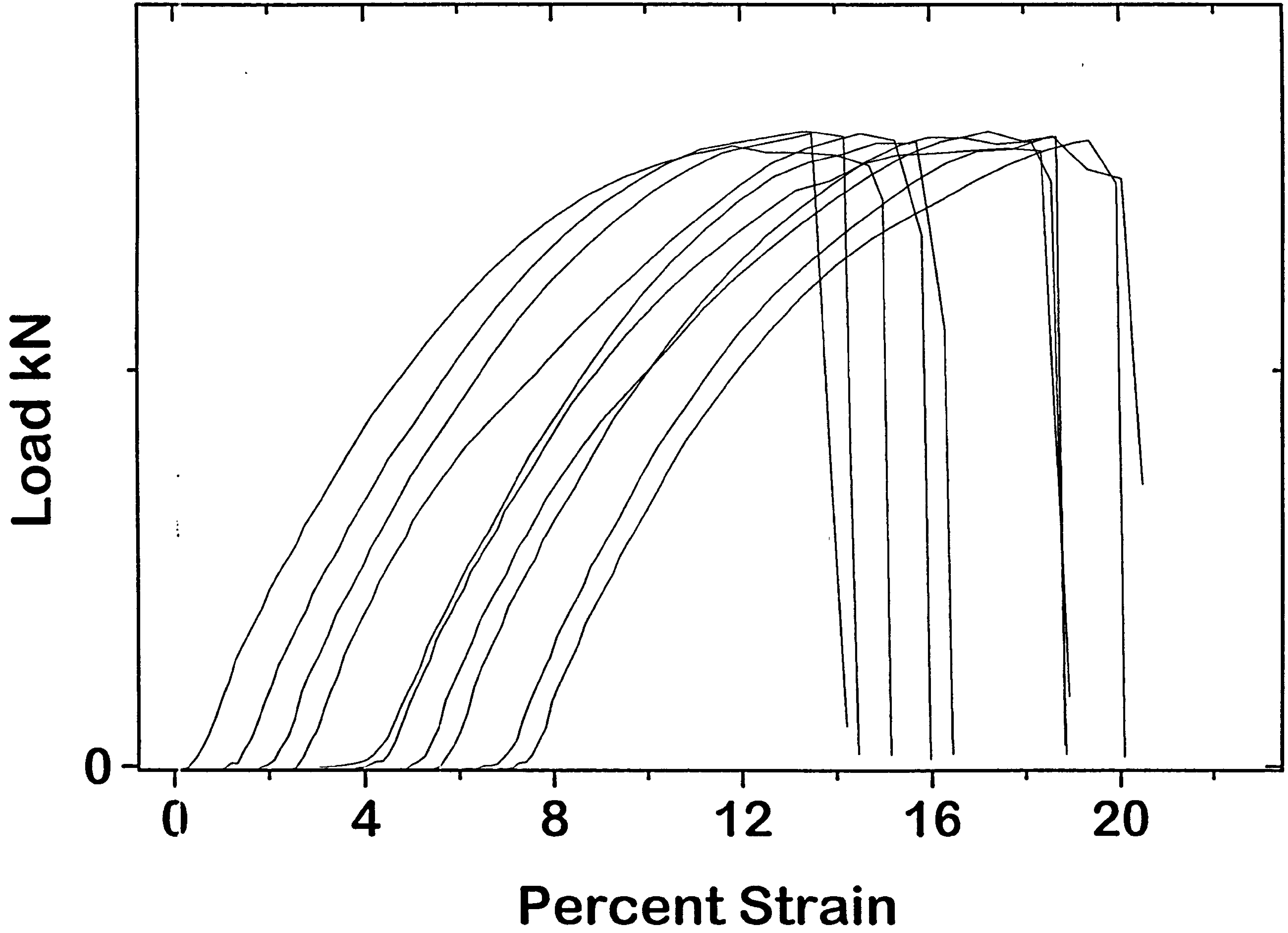
Sample comments:

Custom Test Label:

	Displcment at Max.Load (mm)	Load at Max.Load (KN)	MEAN STRENGTH (KN/M)	% Strain at Max.Load (%)	2% MODULUS (KN/M)	5% MODULUS (KN/M)
1	14.782	1.562	23.281	11.618	7.215	15.613
2	15.645	1.601	23.848	12.274	7.226	15.714
3	14.758	1.596	23.773	11.581	7.234	15.860
4	15.399	1.595	23.758	12.109	7.276	14.371
5	14.825	1.574	23.460	11.556	7.130	15.830
6	17.011	1.556	23.192	13.335	7.332	15.540
7	15.584	1.601	23.848	12.220	7.180	14.806
8	13.402	1.586	23.624	10.541	7.243	15.778
9	14.998	1.588	23.654	11.724	7.556	15.996
10	15.407	1.576	23.490	12.082	7.136	15.604
Mean	15.181	1.583	23.593	11.904	7.253	15.511
S.D.	0.912	0.016	0.231	0.712	0.123	0.515
Mean +0.58 SD	15.710	1.592	23.727	12.317	7.324	15.810
Mean -0.58 SD	14.652	1.574	23.459	11.491	7.182	15.212

Sample ID: NULD

All specimens.



NETLON LTD.
TENSAR DIVISION
BLACKBURN

3.8M AR1 LD (JUNCTION TEST)
SINGLE RIB JUNCTION TEST
TEST SPEED 100mm/min

Test type: Tensile
Operator name: P.EDDLESTON
Sample Identification: NUAR1LDJ
Interface Type: 1120

Instron Corporation
Series IX Automated Materials Testing System 7.27.01
Test Date: 10 September 1996

Sample Rate (pts/secs): 9.1032
Crosshead Speed: 100.0000 mm/min
2nd Crosshead Speed: 0.0000 mm/min

Humidity (%): 50
Temperature: 20 C

Full Scale Load Range: 2000000.0000N

BATCH NUMBER : NOTTINGHAM UNIVERSITY SAMPLE

ROLL NUMBER : EPSRC RESEARCH

SAMPLE WIDTH : 3.820MTRS

SAMPLE WEIGHT : 0.879KGS

MIN RIBS/MTR T.D : 14.9

Sample comments:

Custom Test Label:

	Displcment at Max.Load (mm)	Load at Max.Load (KN)	% Strain at Max.Load (%)	MEAN STRENGTH (KN/M)	2% MODULUS (KN/M)	5% MODULUS (KN/M)
1	11.566	1.241	8.987	18.498	6.494	14.814
2	11.620	1.241	9.007	18.498	7.350	15.238
3	11.394	1.235	8.773	18.409	7.459	15.358
4	11.144	1.211	8.528	18.037	7.703	15.466
5	12.483	1.253	9.638	18.677	6.804	14.691
6	12.032	1.247	9.370	18.588	6.851	14.813
7	12.509	1.292	9.749	19.257	6.836	14.930
8	11.347	1.246	8.865	18.573	6.975	14.963
9	12.098	1.265	9.372	18.855	6.583	14.894
10	11.966	1.239	9.216	18.468	6.823	14.874
Mean	11.816	1.247	9.151	18.586	6.988	15.004
S.D.	0.474	0.021	0.388	0.315	0.391	0.258
Mean +0.58 SD	12.091	1.260	9.376	18.769	7.215	15.154
Mean -0.58 SD	11.541	1.235	8.925	18.403	6.761	14.854

3.8 M AR1 TD (STANDARD QC TEST)
SINGLE RIB QC TEST
TEST SPEED 100mm/min

Test type: Tensile
Operator name: F KENYON
Sample Identification: NUTD
Interface Type: 1120

Instron Corporation
Series IX Automated Materials Testing System 7.27.01
Test Date: 27 August 1996

Sample Rate (pts/secs): 9.1032
Crosshead Speed: 100.0000 mm/min
2nd Crosshead Speed: 0.0000 mm/min

Full Scale Load Range: 2000000.0000N

Humidity (%): 50
Temperature: 20 C

BATCH NUMBER : NOTTINGHAM UNIVERSITY

ROLL NUMBER : EPSRC RESEARCH

SAMPLE WIDTH : 3.820 MTRS

SAMPLE WEIGHT : 0.879 KGS

MIN RIBS/MTR L.D : 15.5

Sample comments:

Custom Test Label:

	Displcment at Max.Load (mm)	Load at Max.Load (KN)	MEAN STRENGTH (KN/M)	% Strain at Max.Load (%)	2% MODULUS (KN/M)	5% MODULUS (KN/M)
1	13.537	1.626	25.205	10.318	8.504	17.720
2	14.794	1.664	25.794	11.259	8.009	17.367
3	13.204	1.622	25.144	10.145	7.844	17.317
4	13.757	1.667	25.840	10.395	8.047	17.810
5	14.941	1.655	25.655	11.470	8.086	17.752
6	15.260	1.662	25.763	11.728	7.913	17.630
7	13.385	1.622	25.144	9.482	8.411	18.255
8	12.140	1.640	25.422	9.316	8.864	18.212
9	13.114	1.635	25.345	10.161	7.935	17.528
10	14.020	1.649	25.562	10.846	8.500	17.741
Mean	13.815	1.644	25.487	10.512	8.211	17.733
S.D.	0.960	0.018	0.273	0.807	0.336	0.311
Mean +0.58 SD	14.372	1.655	25.645	10.980	8.406	17.913
Mean -0.58 SD	13.259	1.634	25.329	10.044	8.016	17.553

NETLON LTD.
TENSAR DIVISION
BLACKBURN

3.8M AR1 TD (JUNCTION TEST)
SINGLE RIB JUNCTION TEST
TEST SPEED 100mm/min

Test type: Tensile
Operator name: P.EDDLESTON
Sample Identification: NUAR1TDJ
Interface Type: 1120

Instron Corporation
Series IX Automated Materials Testing System 7.27.01
Test Date: 10 September 1996

Sample Rate (pts/secs): 9.1032
Crosshead Speed: 100.0000 mm/min
2nd Crosshead Speed: 0.0000 mm/min

Humidity (%): 50
Temperature: 20 C

Full Scale Load Range: 2000000.0000N

BATCH NUMBER : NOTTINGHAM UNIVERSITY SAMPLE

ROLL NUMBER : EPSRC RESEARCH

SAMPLE WIDTH : 3.820MTRS

SAMPLE WEIGHT : 0.879KGS

MIN RIBS/MTR L.D : 15.5

Sample comments:

Custom Test Label:

	Displcment at Max.Load (mm)	Load at Max.Load (KN)	% Strain at Max.Load (%)	MEAN STRENGTH (KN/M)	2% MODULUS (KN/M)	5% MODULUS (KN/M)
1	8.767	1.197	6.613	18.560	7.706	16.116
2	8.953	1.197	6.805	18.560	7.214	15.628
3	13.212	1.175	10.122	18.219	7.369	16.024
4	9.203	1.232	7.062	19.103	7.496	15.900
5	10.775	1.244	8.200	19.289	6.865	15.529
6	9.706	1.256	7.471	19.475	8.004	16.201
7	12.647	1.207	9.688	18.715	7.313	15.607
8	9.267	1.258	7.139	19.506	7.318	15.667
9	8.835	1.230	6.770	19.072	7.622	16.263
10	8.303	1.232	6.412	19.103	7.865	16.626
Mean	9.967	1.223	7.628	18.960	7.477	15.956
S.D.	1.700	0.028	1.304	0.429	0.335	0.355
Mean +0.58 SD	10.953	1.239	8.385	19.209	7.672	16.162
Mean -0.58 SD	8.980	1.207	6.872	18.712	7.283	15.750

CHAPTER 6

CYCLIC SHEAR BOX TESTING

CONTENTS

6.1 Introduction

In general, pavements are designed and built with as few layers as possible to keep costs to a minimum whilst still achieving the function of the pavement. By keeping the number of pavement layers to a minimum, works-induced problems (which may occur due to plant movements and exposure to inclement weather) are also minimised. Damage to exposed layers during construction can lead to variable and/or poor bond between layers, which in turn may cause poor performance under load. The effect of a poor interlayer bond is illustrated below.

It follows that if reinforcement is built into the structure an additional interface is created, which unfortunately makes the likelihood of construction-induced problems higher. Also, the additional layer interface may have quite different properties to other more typical pavement interfaces, if the stiffness and strength of the reinforcement is significantly different to that of asphalt. This further increases the possibilities of construction- and load-induced problems.

The bond between asphalt and reinforcement can be fundamentally different, depending on the type of reinforcement, i.e. grid interlock versus fabric-bitumen adhesion. The manner in which these different bonds are affected by loading also needs to be considered, as cyclic (wheel) loads may have a different effect on the two bond types.

As wheels move over a pavement, stresses are reversed, as portrayed in Figure 6.1. The effect of this stress reversal on the deterioration and failure of the bond between pavement layers, and specifically the bond between reinforcement and asphalt, is difficult to assess, but it may influence pavement behaviour.

It follows therefore that there is a need to define the properties of interface bond under load, and, to be compatible with real pavement situations, under repeated loads.

To illustrate the effect of bond strength on performance De Bondt [6.1] uses a simple example of a point-loaded beam:

The deflection of a simply supported beam of depth 'd', breadth 'b', length 'l' subject to a point load 'F' in the centre of a beam is given by

$$\delta = \frac{Fl^3}{48EI}$$

Where

$$I = \frac{bd^3}{12}$$

If the beam is decoupled into two separate beams each with a depth d/2, the central deflection now becomes

$$\delta = \frac{Fl^3}{12EI}$$

i.e. 4 times greater.

Comparing stresses at extreme fibres for the beam described above:

$$\sigma = \frac{My}{I}$$

Where y is the distance from the neutral axis to the edge of the beam.

For the decoupled beam, the maximum tensile stresses in the two half-thickness beams are greater than for the single (thicker) beam by a factor of 2 (and, in addition, tension is found at the underside of both beams). This additional tensile strain can, in turn, lead to a decrease in pavement life for bituminous materials [6.2], which is to be avoided.

Whereas it is understood that the above situation is an extreme case, i.e. it is assumed that there is no friction between the upper and lower layers, it does illustrate the importance of bond between layers. This further suggests that to analyse or design pavements which always include interlayers, the magnitude and possible deterioration of interlayer bond is one of the factors that need to be assessed.

For some potential users of reinforced asphalt, knowledge that an additional interface is created with a layer of reinforcement has lead to a perception that slippage between layers could be induced, especially where they are relatively close to the surface of the pavement [6.3]. This concern is accentuated where higher-than-normal horizontal stresses are imparted to the pavement, such as where traffic accelerates or brakes quickly, or on sharp corners. It should be noted however, that although this concern would appear to be sensible, documented evidence of this problem is difficult to find.

It is generally known that under repeated loading the strength and stiffness of bituminous materials deteriorate gradually (i.e. it fatigues [6.2]), the speed of which, largely determined by the level of stress applied, and the speed and number of load applications. The two main unknowns concerning the shear properties of the interface bond are therefore (1) the level of stress applied to the interface, and (2) the fatigue characteristics of the interface bond.

An investigation to address some of the points raised above was carried out by Hughes [6.4] using a large-scale shearbox in the direct shear mode (see Figure 6.2). Both unreinforced and reinforced samples were sheared to failure using monotonic loads, giving a set of results that are summarised in Figure 6.2. As samples were subjected to different normal stresses whilst being sheared, results are compared by using a ratio of the shear and normal stresses at failure. Figure 6.2 shows that the highest ratio of shear-to-normal stresses was found with an asphalt sample constructed in one layer.

Next were two-layer constructions including a polypropylene grid-reinforced interface, a polypropylene grid with a chipseal and unreinforced interfaces with and without a chipseal. Although a limited number of samples were tested, the results indicate that the inclusion of a reinforced interface does not significantly alter the shear resistance of the interface, and may actually increase it. The continuous (1 layer) construction had the highest shear resistance and the lowest values were obtained with the chip seal interface, which may indicate that there was little interlock between the chip-seal dressing and the lower layer of the sample. The high shear resistance of the continuous sample on the other hand reflects the better interlock of aggregate at the shear plane.

Cyclic shearing tests were not however carried out to characterise interface properties, but was thought that these would show differences to properties from monotonic loading. A programme of cyclic shearing tests and four preliminary fatigue tests was therefore carried out as described in the following sections.

Although the direct shear test is a well-used and popular means of obtaining shear strength parameters for soils, the limitations of this test mode are well-known [6.5,6.6,6.7] for (*inter alia*) the following reasons:

- shear parameters are derived from a test that is carried out on a predetermined shear plane (see Figure 6.2). This plane may not be representative of the overall mass of material or be the most critical plane for testing.
- the distribution of stress on the shear plane is not uniform, and the directions of the planes of principal stresses rotate as shear strain increases (see Figure 6.3).
- stress concentrations at the front and rear of the sample cause progressive failure along the shear plane.

Although testing on a predetermined plane is considered a limitation when determining the shear strength of a soil, for instance, for obtaining parameters describing the attributes of a reinforced interface it is an advantage. This assumes that the sample is suitably positioned in the test mould, i.e. with the interface at the junction of the two halves of the test mould. Therefore, notwithstanding the above limitations, the shearbox test was considered to be useful for providing data to aid in the understanding of reinforced asphalt behaviour, even if only as comparative data.

Due to the nature of pavement behaviour under traffic loading, the ultimate failure strength of the interface (the parameter usually measured by the shearbox) is not normally required. However, the direct shearbox test can also be used to measure the relationship between applied loading and elastic displacement across the interface. This measure of interface stiffness is required as input into the CAPA-2D Finite Element model. Similarly, to quantify interlayer slip in a pavement, the Shell Pavement Design Method [6.8] uses a measure of the 'shear spring compliance' of the interface using the principles of Goodman's Law [6.9]:

$$\text{Shear Spring Compliance} = \frac{\delta_h}{\tau}$$

Where δ_h is the relative deflection measured between the layers due to the applied stress τ.

The interface shear stiffness is thus the reciprocal of the shear spring compliance, i.e.

$$\text{Shear Spring Stiffness} = \frac{\tau}{\delta_h}$$

Values of this measure of stiffness are expressed as kN/mm/mm^2 in this document i.e. the applied shear stress required to cause a given deflection measured over the area of the sample interface.

6.2 Test Apparatus and Method

The shear box mentioned above was originally constructed to investigate the magnitude of different interface bonds [6.4] for specimens 320mm x 200mm x 120mm deep under monotonic loading. The principal reason for using shear boxes of these dimensions was to test specimens large enough to include representative samples of interlayer reinforcement. As the apparatus had already been successfully used, and considering the time required to design and construct a new apparatus for cyclic loading, the 'original' apparatus was adapted to the configuration shown in Figure 6.4. Alterations to the load cell connections and the steel frame restraining the shearbox were made to allow the application of both tension and compression loads. Also, after pilot trials, where dynamic loading caused specimens to compress and become loose within the apparatus, specimens were fixed to top, bottom and side plates with epoxy. This prevented the irregular, non-sinusoidal loading patterns previously found from reoccurring.

A full fatigue test programme was beyond the resources of the project, but a preliminary investigation was carried out to investigate the effect of repeated shear using four samples. The results of these tests are given in Section 6.5.

The test specimens were made in a roller compactor apparatus and not compacted in the test apparatus as was the case in the earlier work [6.4]. The more controllable roller-compactor compaction technique was used to improve quality control. Measures of density (and hence air voids) are given in Appendix 6.1 and show air voids of around 4 to 8 %. Samples were constructed in two layers and the reinforcement and the second layer of asphalt were placed after the temperature of the first layer had reduced to 50°C (an approximation to conditions that might be experienced on a pavement in summer). Reinforcement was placed according to manufacturers' recommendations.

To obtain an approximation of the stresses that might be experienced in the field, the multi-layer linear elastic programme ELSYM5 was used, the results of which are given in Appendix 6.2.

The results of this modelling suggest that a normal stress of 200kPa could be appropriate for testing, which was also consistent with previous shearbox tests [6.4].

To test samples to failure in a 'cyclic' (reversed shear) mode, 1000 repetitions of shear stresses were applied in increments of 100kPa at a frequency of 2 Hz at 20°C until the samples failed, i.e. showed significant reduction in bond strength.

6.3 Instrumentation and Data Logging.

To measure deflections across the interface, LVDTs were mounted on one side and a target on the other (see Figure 6.5). Deflection and load cell data was collected simultaneously using an Autoscan Ranger datalogger (using the 'Turboview' software) at a rate of 100Hz.

Shear loads were applied at 2Hz following trials that showed quicker loading rates to give erratic load waveforms due to 'play' in mechanical connections between the sample and the servohydraulic system.

Data was processed in three stages: raw data was first converted from binary to ASCII format using the data logging software. This was then digitally filtered and converted to deflections and stresses from which maximum and minimum values of stress and deflection were extracted. Examples of 'raw' and filtered data are given in Figure 6.6.

6.4 Cyclic Shear Tests: Results

Due to physical limitations of the size of instrumentation, deflection measurements across the interface included a component of asphalt strain from above and below the interface, (see Figure 6.7). To correct deflection readings, therefore, a relationship between asphalt stress and strain was developed by loading an asphalt specimen in compression and measuring resulting strains. The test configuration is shown in Figure 6.8, and test results are given in Figure 6.9.

The relationship obtained between applied stress and resultant strain in the asphalt was

$$\mu\varepsilon = 0.1615 \times \text{(applied stress in kPa)}$$

Measured deflections were corrected for asphalt compression using the non-uniform stress distribution given in Reference 6.1 and shown in Figure 6.10. Corrected readings are plotted against applied stress in Figure 6.11.

Two distinct types of behaviour are noted in Figure 6.11, and correspond to grid reinforcement and samples with interlayers comprising grid-fabric composite or fabric interlayers. This is consistent with the general understanding that the bond between grids and asphalt is due to interlock of aggregate and the grid 'ribs', and the bond between asphalt and fabrics is due to bitumen adhesion.

The AR-G, CG50 and ROTAFLEX-reinforced interfaces failed before the grid-reinforced samples. This is probably due to the 100 Penetration bitumen being relatively soft at 20°C. The grids rely on interlock 'bonding' between the grid and aggregate and this mechanism is less dependent on temperature than is the purely bituminous adhesion bond.

It is also noted that even with corrected deflections, some values seem unusual, particularly at higher stresses where deflections for the GlasGrid and Roadmesh samples appear to decrease. Provided the correction applied to deflections is appropriate, this may suggest that with higher (cyclic) loads, better interlock is developed, which with stiff materials (glass and steel), results in smaller deflections. A well-interlocked grid may help dissipate stresses into the asphalt, by reducing the effect of the discontinuity. This would in turn reduce deflections measured across the interface over the gauge length. If this is the case, it would appear that the stiffness of the grid should have an effect on test results if the bonding between asphalt and reinforcement is adequate. In this regard it is noted that deflections for the AR1-reinforced sample appear to lie between the composite-reinforced samples and the steel- and glass-reinforced samples, possibly supporting this supposition.

Secant stiffnesses calculated from data shown in Figure 6.11 are given in Figure 6.12. These values are compatible with values published by Scarpas et al [6.10], although they are generally higher than the values suggested for 'typical' conditions (i.e. only up to 5N/mm/mm^2).

Inspection of the failed specimens showed that in each case, the interface material was fixed to the upper layer of the asphalt, i.e. the bond between the interface material and the lower layer of asphalt was weaker than the bond between the top layer of asphalt and the interface material. This was probably due to the hot asphalt top layer being more fluid than the cooler lower layer, thus moulding itself better in and around the grids. With composites, it seems the fabrics formed a better bond with the hot overlay than the lower asphalt layer, even though a layer of tack coat was provided between the fabrics and the lower layer of asphalt.

Samples that failed at higher stress levels were subjected to more load repetitions, and therefore more degradation due to fatigue than those that failed at lower stresses. How significant this effect has been on results is not known, but, as all samples were subject to similar loading regimes, a comparison of behaviour is still considered valid.

Values of failure stress from earlier testing using monotonic loading [6.4] are given in Table 6.1, and a summary of both cyclic and monotonic stresses at sample failure is given in Table 6.2

Table 6.1 Monotonic Shearbox Test Failure Stresses [6.4]

Sample	**Interface Characteristics**	**Normal Stress (kPa)**	**Failure Stress (kPa)**
Control 1	One-layer compaction	195	380
Control 3		435	513
Control 7	Two-layer compaction	193	329
Control 9		427	625
AR1-1	AR1	166	313
AR1-3		425	431
AR1-5	AR1 +Chip seal	175	225
AR1-7	.. ..	433	441

It is noted that shear 'failure' in the cyclic shear tests (i.e. where displacements increase noticeably) occurs at approximately 40% of the failure stress of the monotonic tests for unreinforced samples, and around 45% for reinforced samples. The reason for this large reduction in applied stress was not established but is thought to be due to weakening of the bond through fatigue.

Table 6.2 Comparison of Monotonic and Cyclic Shear Stresses at Failure

Sample	**Normal Stress** (kPa)	**Shear Stress Amplitude** (kPa)	**Ratio:** (τ/σ)
Monotonic loading			
Control 1	195	760	3.9
Control 2	186	580	3.12
Control 7	193	658	3.41
AR1-1	166	626	3.77
AR1-2	180	574	3.19
Cyclic loading			
Control	200	250	1.25
AR1	200	280	1.4

Although there are a limited number of samples for comparison, results show that there is a significant reduction in shear stress at failure (for a given normal stress) in cyclic tests, as compared to monotonic tests. More testing is required to determine reasons for these apparent differences in failure loads, as there are important implications for reinforced pavement design. In particular, comparisons of monotonic and cyclic shear failure stresses are required for other types of reinforcement, especially fabrics and composite reinforcement, to see if similar reductions in loads required for failure exist.

6.5 Fatigue Testing

In-service pavements normally 'fail' due to repeated loading (fatigue), and not a single load application. The fatigue properties of the bitumen emulsion bond typically used between sample (and pavement) layers was therefore assessed by testing four unreinforced samples at a frequency of 2 Hz, with the applied stresses given in Table 6.3.

Table 6.3 Loading used for Shear Box Fatigue Tests

Sample	Shear Stress (kPa)	Normal Stress (kPa)
CNTa	364	200
CNTb	295	200
CNTc	280	200
CNTd	314	200

The value of applied shear stress for the first fatigue test was chosen on consideration of the failure stresses measured in earlier monotonic tests, (typically around 320 to 330kPa [6.4]). The philosophy was to first test samples with relatively high interface stresses and to fail the sample with few repetitions. Then, by reducing load, tests were to be carried out with longer test durations.

To monitor relative movements of the two halves of the sample, and thus define interface failure, horizontal displacement across the interface was measured using LVDTs in the centre of the specimen. Failure of the interface bond was defined as being where the slope of the displacement-load applications graph noticeably changed, as shown in Figure 6.13. To obtain representative measurements, samples were instrumented on both sides.

Figure 6.14 gives the results of the fatigue testing, which shows some obvious differences in results between the LVDTs. This could be due to the sample not failing evenly across the interface. Considering the possible variations in tack coat thickness, eccentric loading in the horizontal plane, or variation in the roughness of the interface surfaces of the asphalt and their adhesive properties with bitumen, it seems quite likely that 'failure' would be measured at different times on both sides of the samples. To be more certain of when and how interface failure occurs, additional instrumentation placed across the interface at intervals along the entire length of specimen would be useful.

The test results in Figure 6.14 show that (as expected) the number of applied loads to interface failure increases as shear stress reduces. Results also indicate that the best relationship was obtained from LVDT P data. For the relationship shown in Figure 6.14, the value of shear stress for failure to occur at one load application, (i.e. a monotonic test mode) is between 908 and 1102kPa, which is higher than failure stresses measured by Hughes [6.4]. This could be due to natural variation in materials, effects of load reversal (possibly causing more interlock to occur), or test rate effects, which can alter the stiffness of bitumen.

Monotonic tests were carried out with a rate of shear of 5mm per minute, which meant that the cyclic loading was approximately 7.5 times quicker. Using the Van der Poel nomograph, [6.11] it can be shown that for a test speed ratio of 7.5, the ratio of Bitumen stiffness is approximately 3 times (i.e. stiffer in the cyclic test).

Janssen and Molenaar [6.12] report the findings of an investigation where bitumen emulsion was sheared under repeated loading, and gave the fatigue relationship shown in Figure 6.15. Shearbox test results (factored by 3.0 in calculating stress ratios to take into account test rate effects) are also plotted in Figure 6.15. Although the gradient of the lines are reasonably similar, results from the shearbox lie above the bitumen emulsion fatigue line, i.e. apparently showing that the shearbox specimens were more resistant to fatigue deterioration than a layer of bitumen emulsion.

The cause of the increased fatigue resistance is probably aggregate interlock across the interface, which, if true, requires further investigation. Understanding this feature of reinforced asphalt interfaces could lead to development of better combinations of asphalt and reinforcement to improve interface fatigue properties. In this respect, additional types of reinforcement need to be tested.

Notwithstanding the small number of samples tested and the scatter of points, results suggest that (as expected) the number of load applications to failure is affected by load magnitude and the presence of reinforcement. This may have important implications in design and will need further investigation.

6.6 Concluding Comments

From the above, the following points are noted

(a) Consideration of interface bonding properties is important. Under load a well-bonded layered structure will develop smaller deflections and tensile stresses than a poorly-bonded structure.

(b) The shear box can be used to measure interface shear bond by applying monotonic or cyclic shear stresses.

(c) There is a clear difference in stress-displacement behaviour for samples reinforced with grids and those reinforced with fabrics or composites when tested at 20°C. Where cyclic shear stresses are applied with amplitudes greater than 450kPa, the cyclic displacement of samples with fabric and composite interlayer materials is greater than for samples with grid interfaces. The variations are attributed to the differences between grid-aggregate interlock and adhesion of fabrics to asphalt.

(d) Failure loads for cyclic tests were between 40% and 45% of monotonic test results. More testing is required to confirm this provisional finding and to investigate further – especially with other reinforcement types.

(e) Failure occurred on the interface between the reinforcement and the lower layer of asphalt. Bonds between the freshly-applied asphalt and the reinforcement were better than between the reinforcement and the 'older' asphalt.

(f) Fatigue tests show that interface bonds on unreinforced (2-layer) asphalt specimens deteriorate more slowly than reinforced interfaces bonded with a layer of bitumen emulsion, i.e. with composite reinforcement. It is considered that this is due to the interlock between the upper and lower layers of the unreinforced samples. This needs confirmation with more testing.

(g) Values of shear stiffness in general lay between 5 and 30N/mm/mm^2 with values for unreinforced and grid-reinforced samples tending to increase with applied shear stress. Shear stiffnesses for composite and fabric-reinforced samples, on the other hand tended to decrease with increased stress.

(h) Dynamic shearbox tests have shown clear differences between grid-reinforced interfaces and interfaces comprising fabrics. The stiffnesses of grid-reinforced interfaces tend to be higher than interfaces comprising fabrics particularly at higher stresses. Table 6.4 gives a summary of interface shear stiffnesses derived from measurements of applied stresses and resultant deflections.

Table 6.4 Summary of Interface Shear Stiffnesses (kN/mm/mm^2)

Interface Reinforcement Type		Applied Stress (kPa)			
		50	100	200	250
AR1	Grids	-	15	20	13
Glas Grid		5	8	22	>50
ROAD-MESH		8	20	>50	>50
None (Control)		13	-	17	-
AR-G	Fabric -	6	14	10	4
CG50	backed	10	11	12	8

6.7 References

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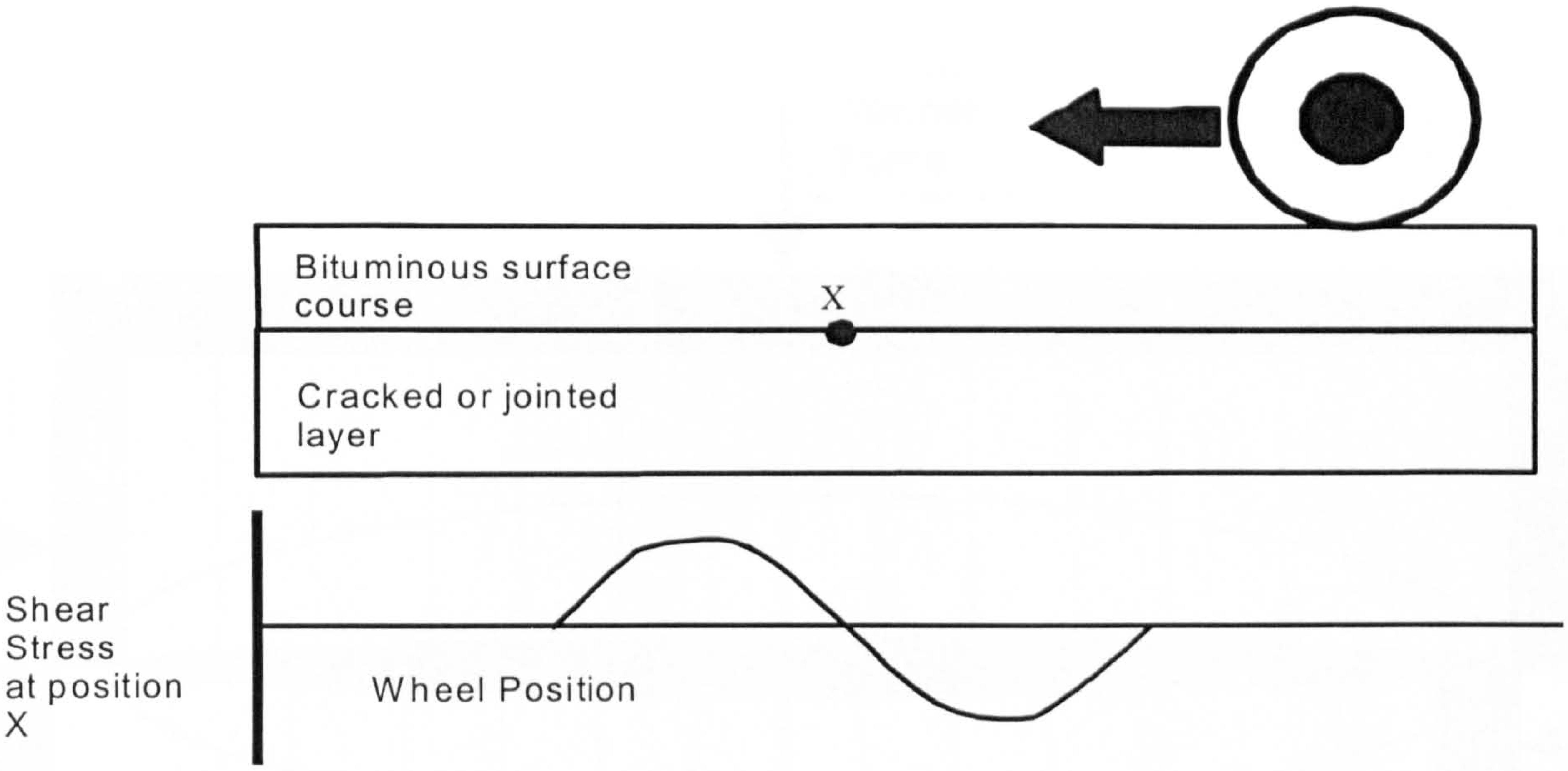

FIGURE 6.1
STRESS REVERSAL UNDER A MOVING WHEEL LOAD

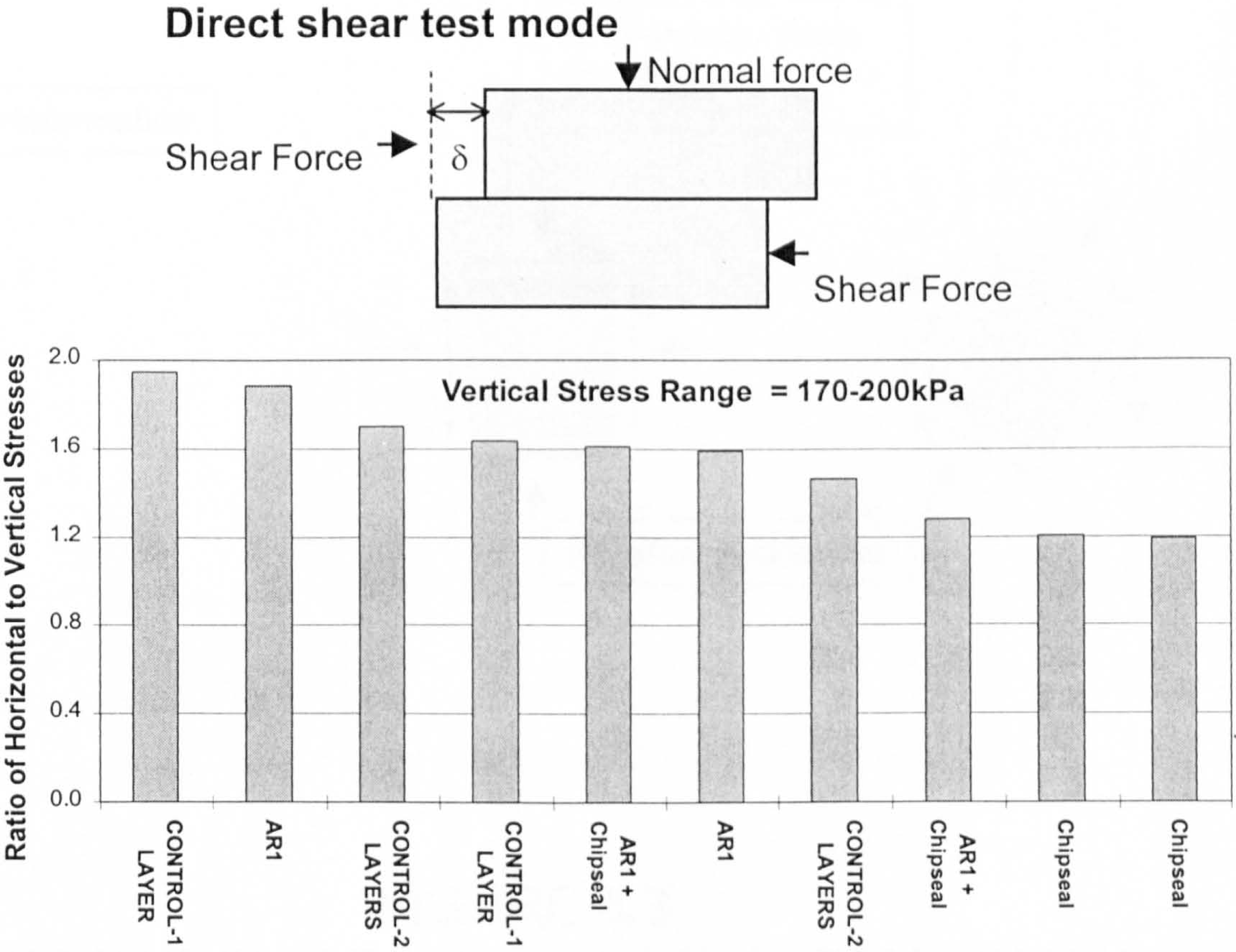

FIGURE 6.2
DIRECT SHEAR TEST:MONOTONIC LOADING [6.4]

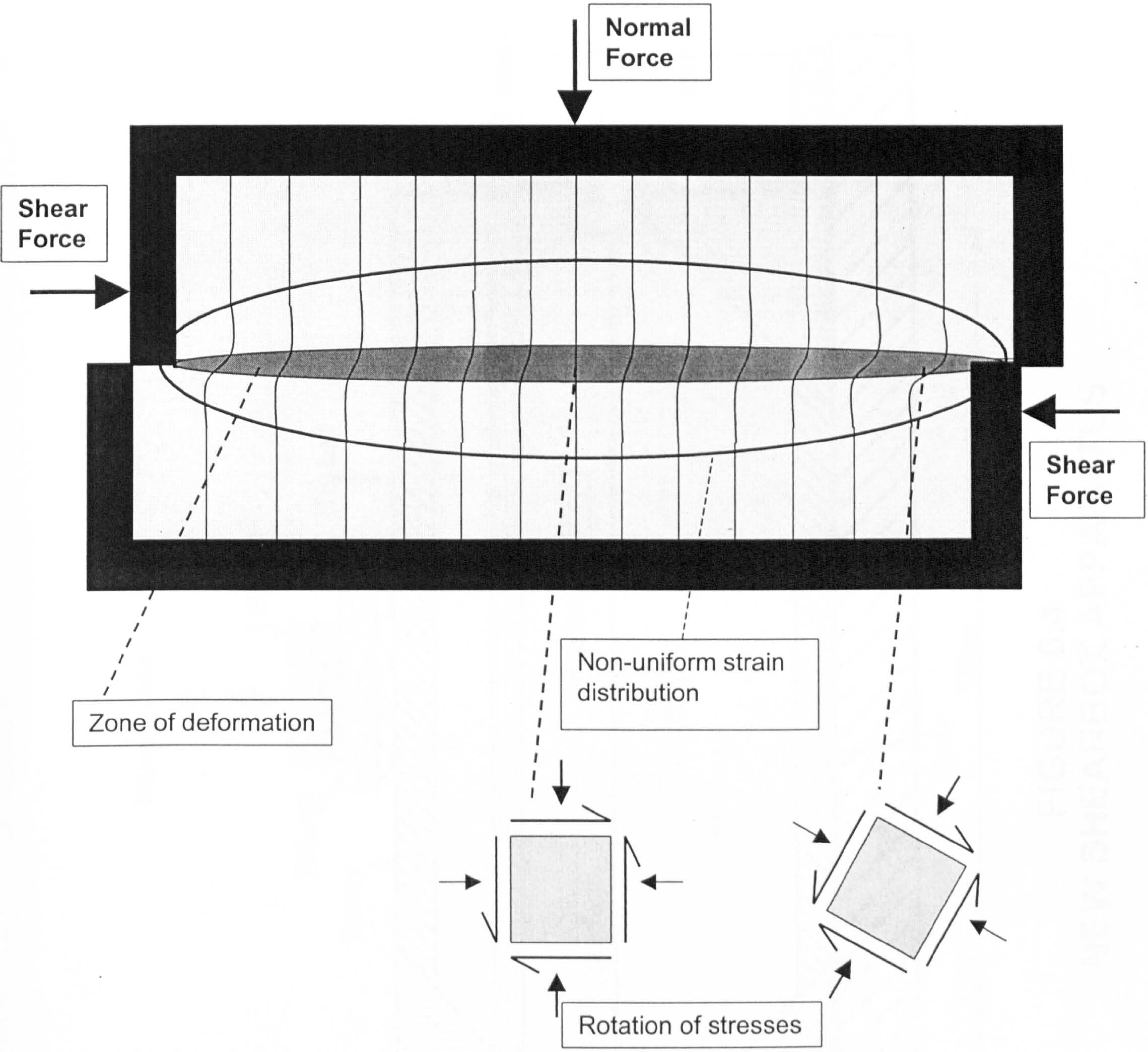

FIGURE 6.3
ILLUSTRATION OF NON-UNIFORM CONDITIONS
WITHIN THE DIRECT SHEAR TEST

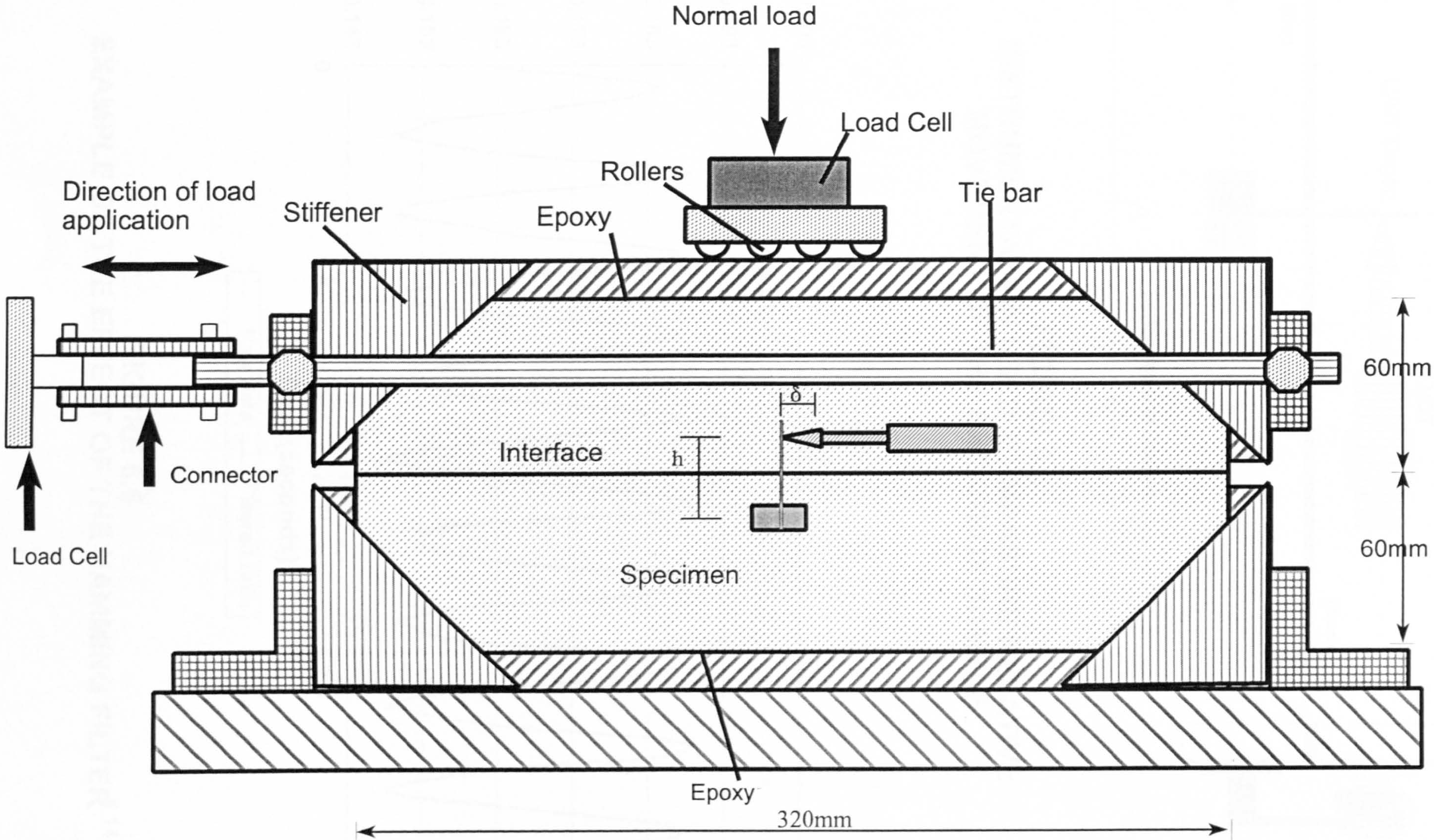

FIGURE 6.4
NEW SHEARBOX APPARATUS

SH2.wpg

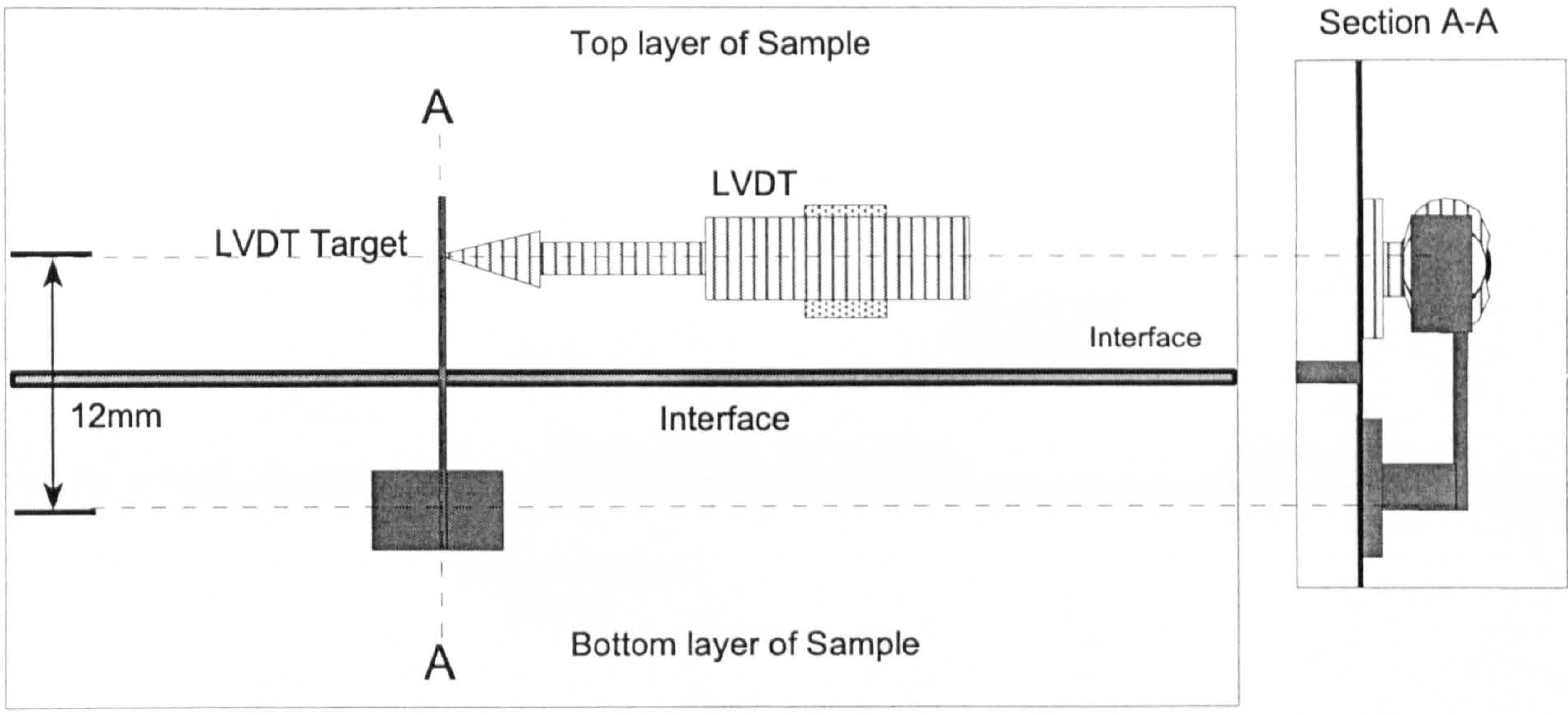

FIGURE 6.5
INSTRUMENTATION USED TO MEASURE RELATIVE MOVEMENT OF THE SHEARBOX SAMPLE

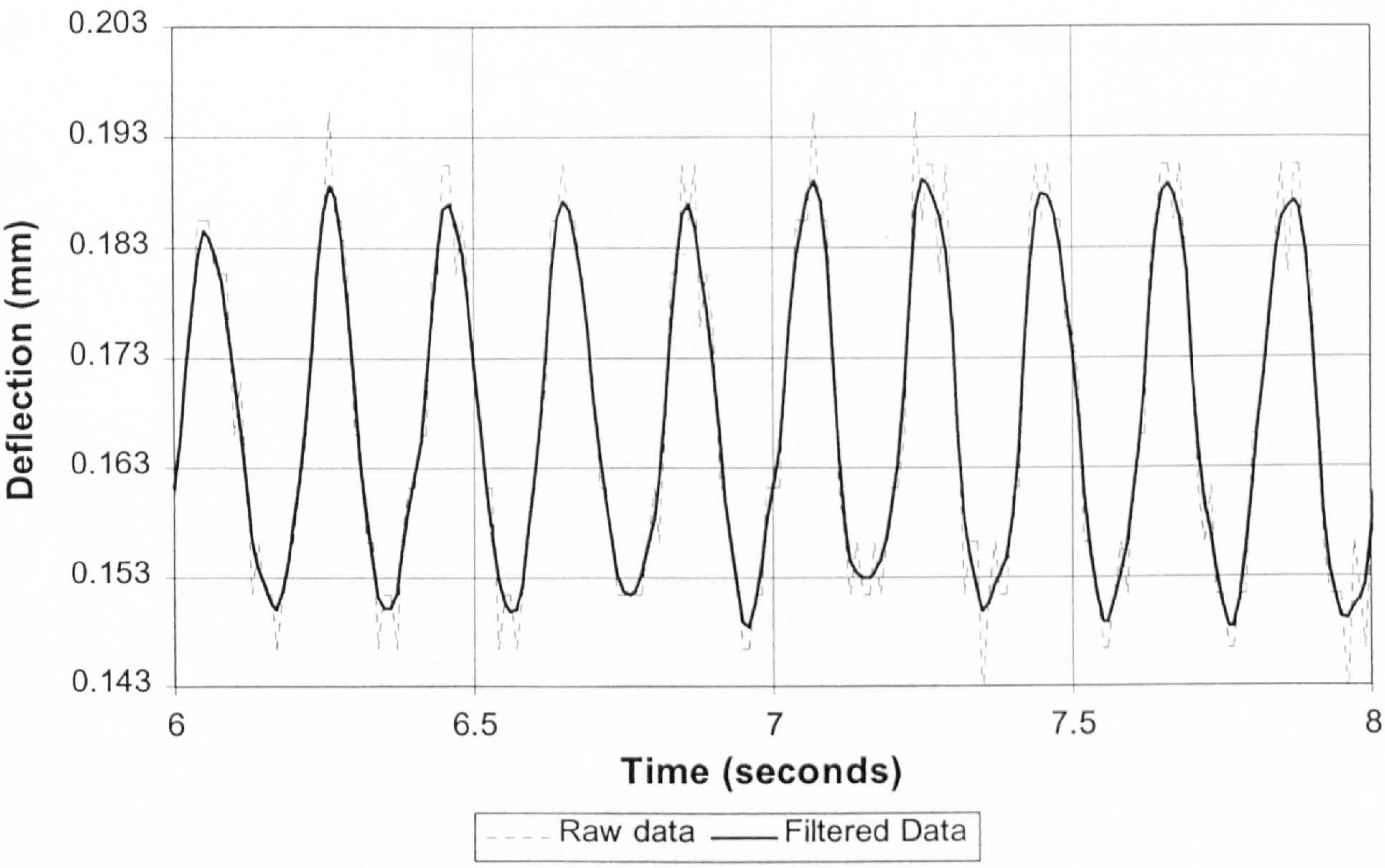

FIGURE 6.6
EXAMPLE OF THE EFFECT OF THE HAMMING FILTER

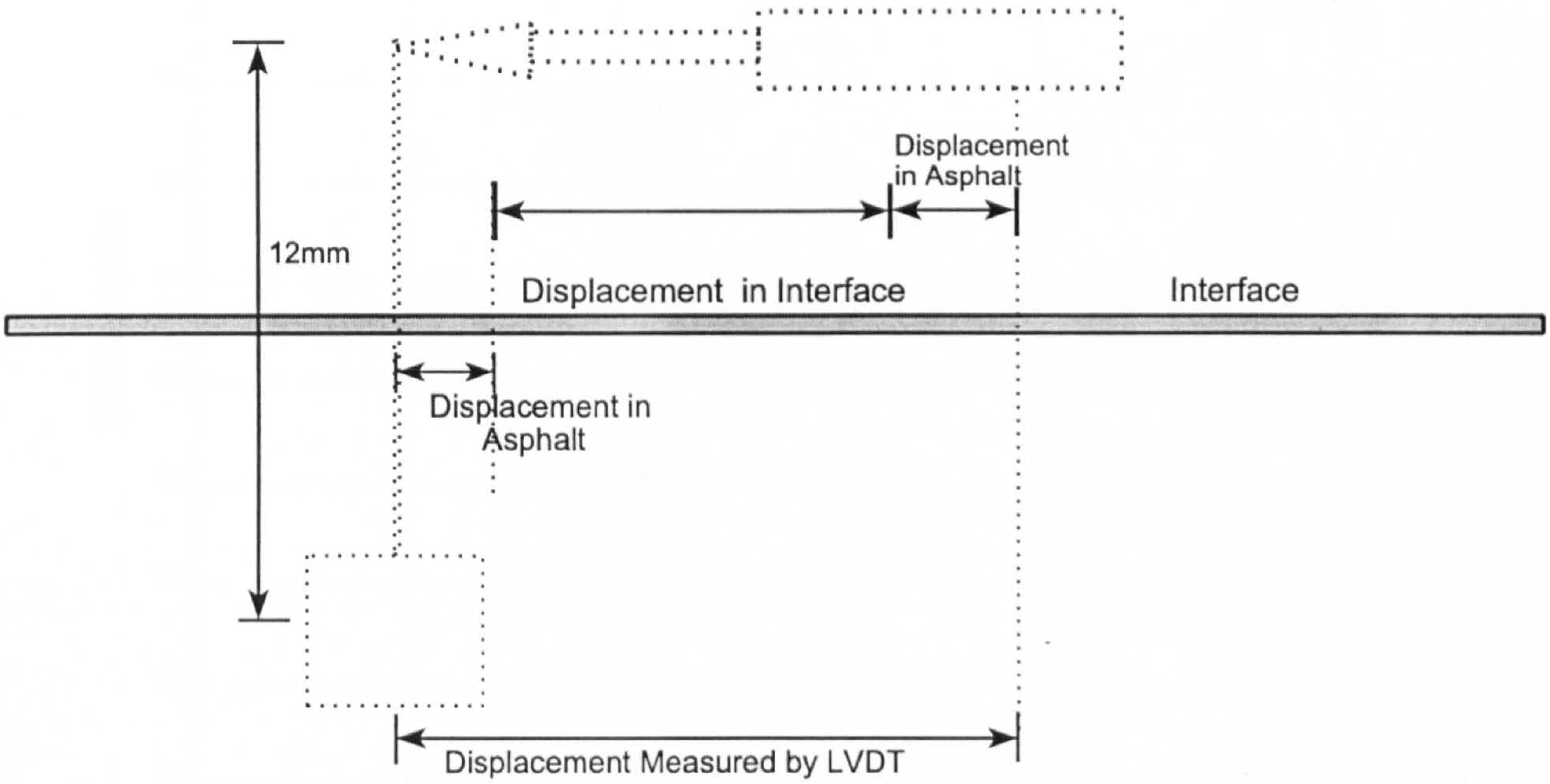

FIGURE 6.7
COMPONENTS OF MEASURED DISPLACEMENTS

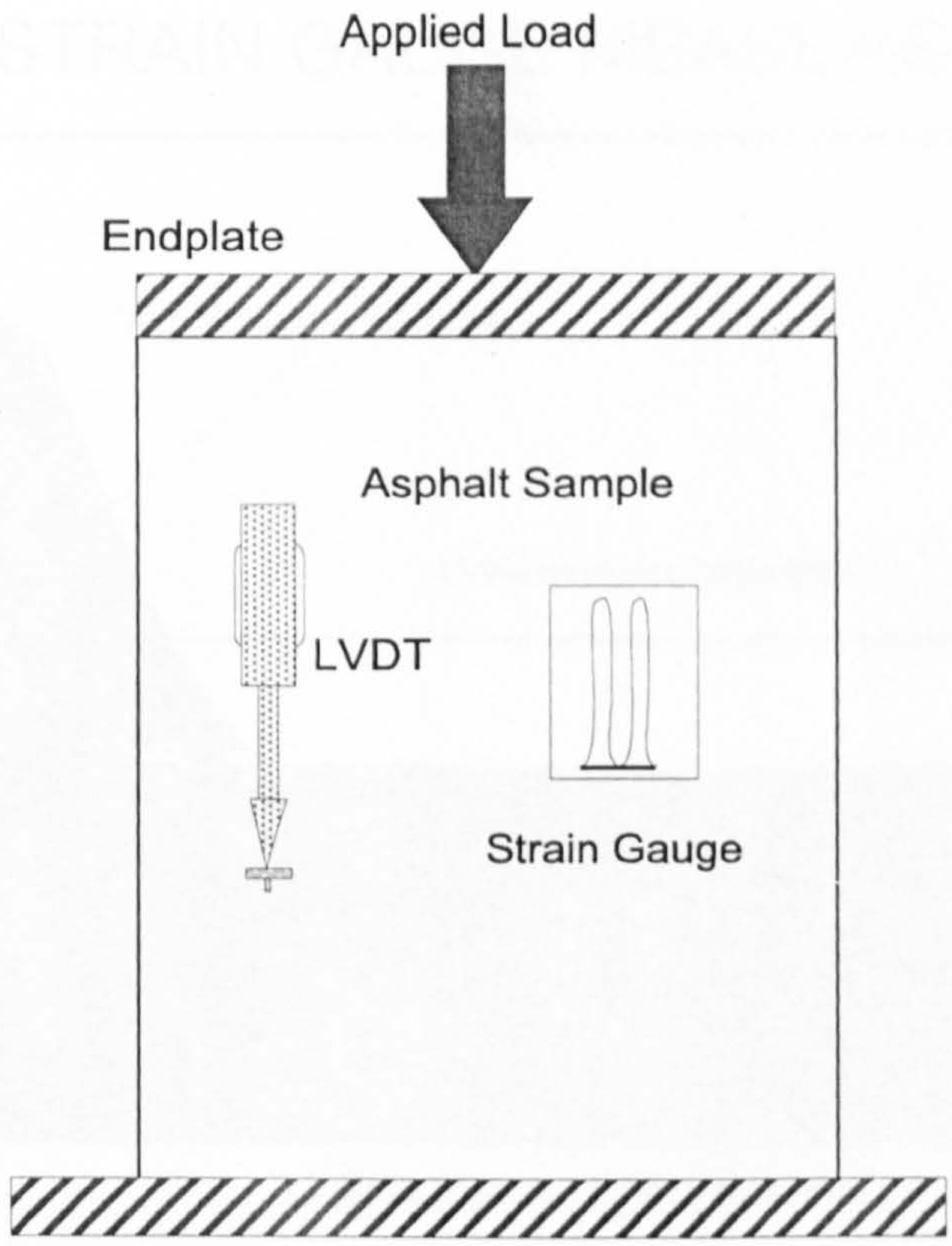

FIGURE 6.8
TEST CONFIGURATION USED TO MEASURE ASPHALT COMPRESSION

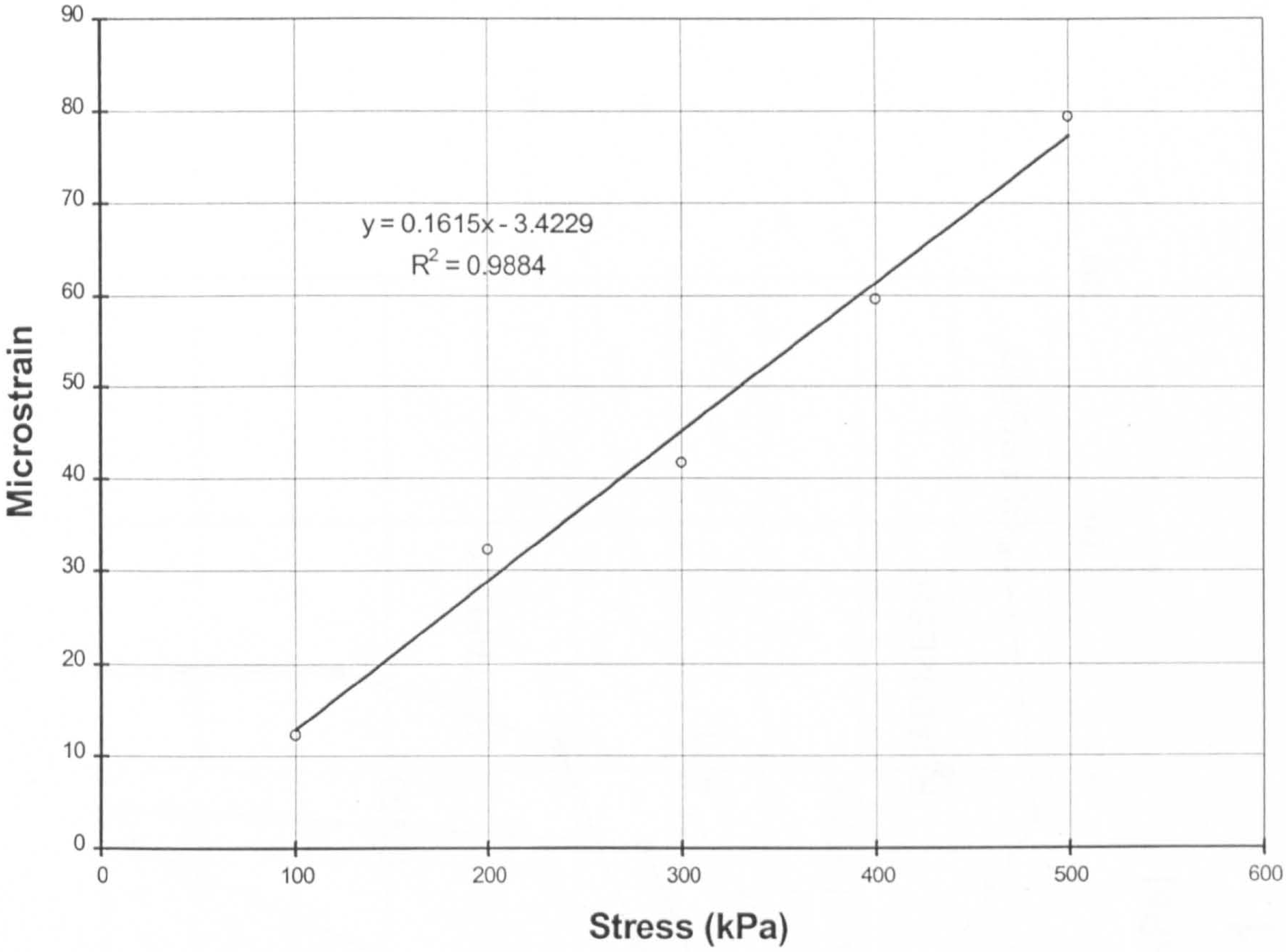

FIGURE 6.9
STRAIN GAUGE MEASUREMENTS

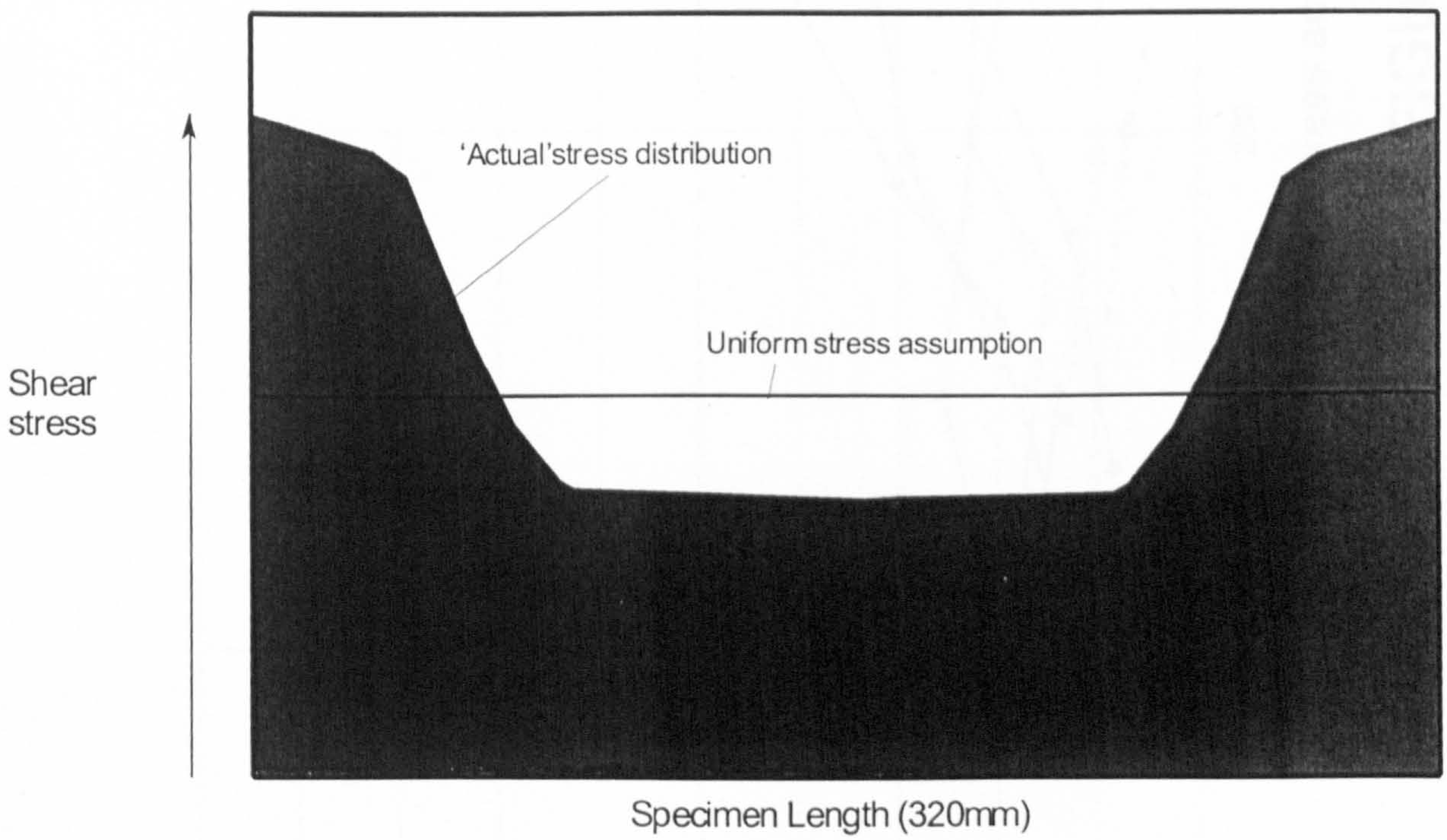

FIGURE 6.10
SHEAR STRESS DISTRIBUTION [6.1]

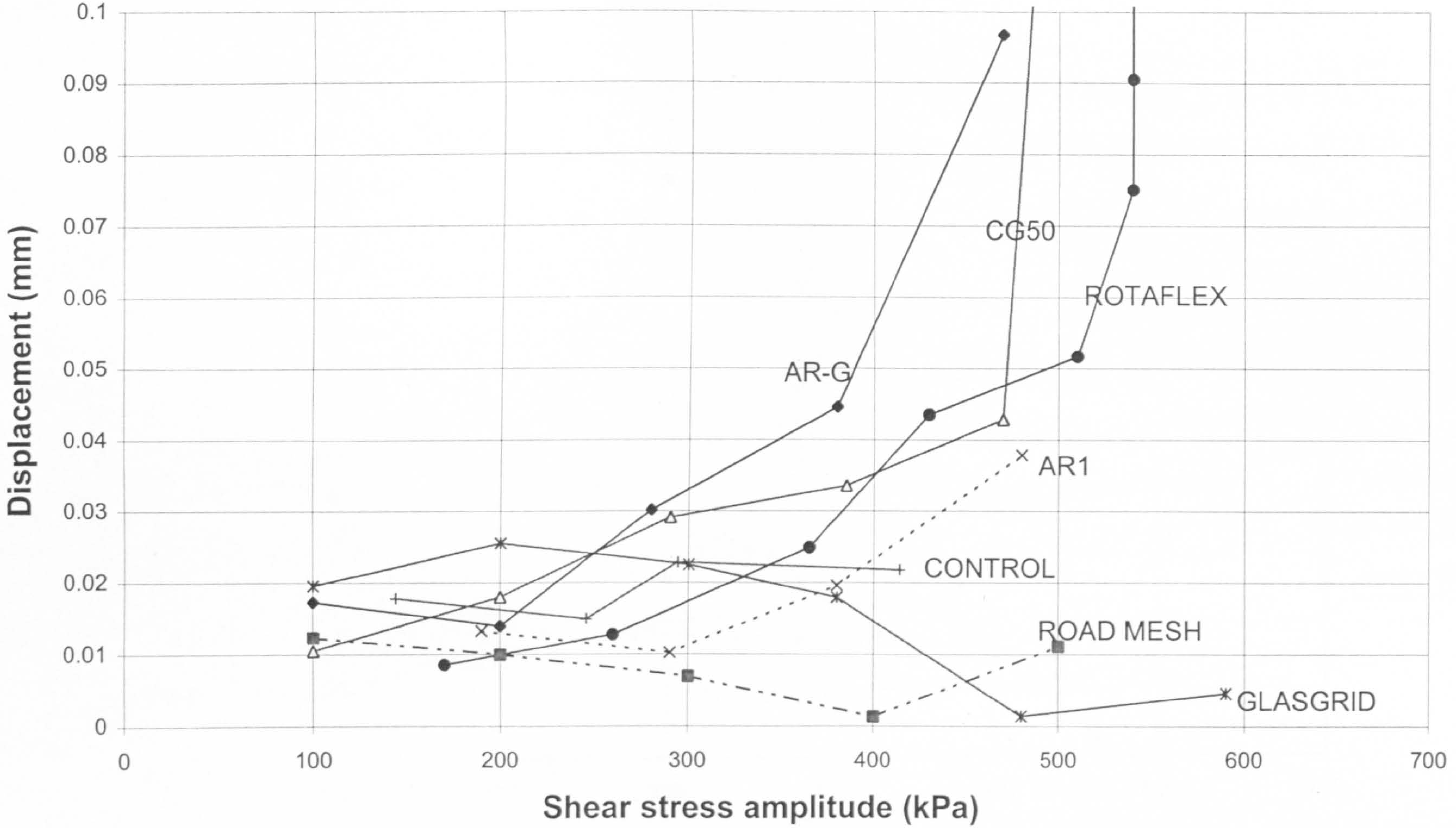

FIGURE 6.11

SUMMARY OF INTERFACE DISPLACEMENTS FROM CYCLIC SHEARBOX TESTS

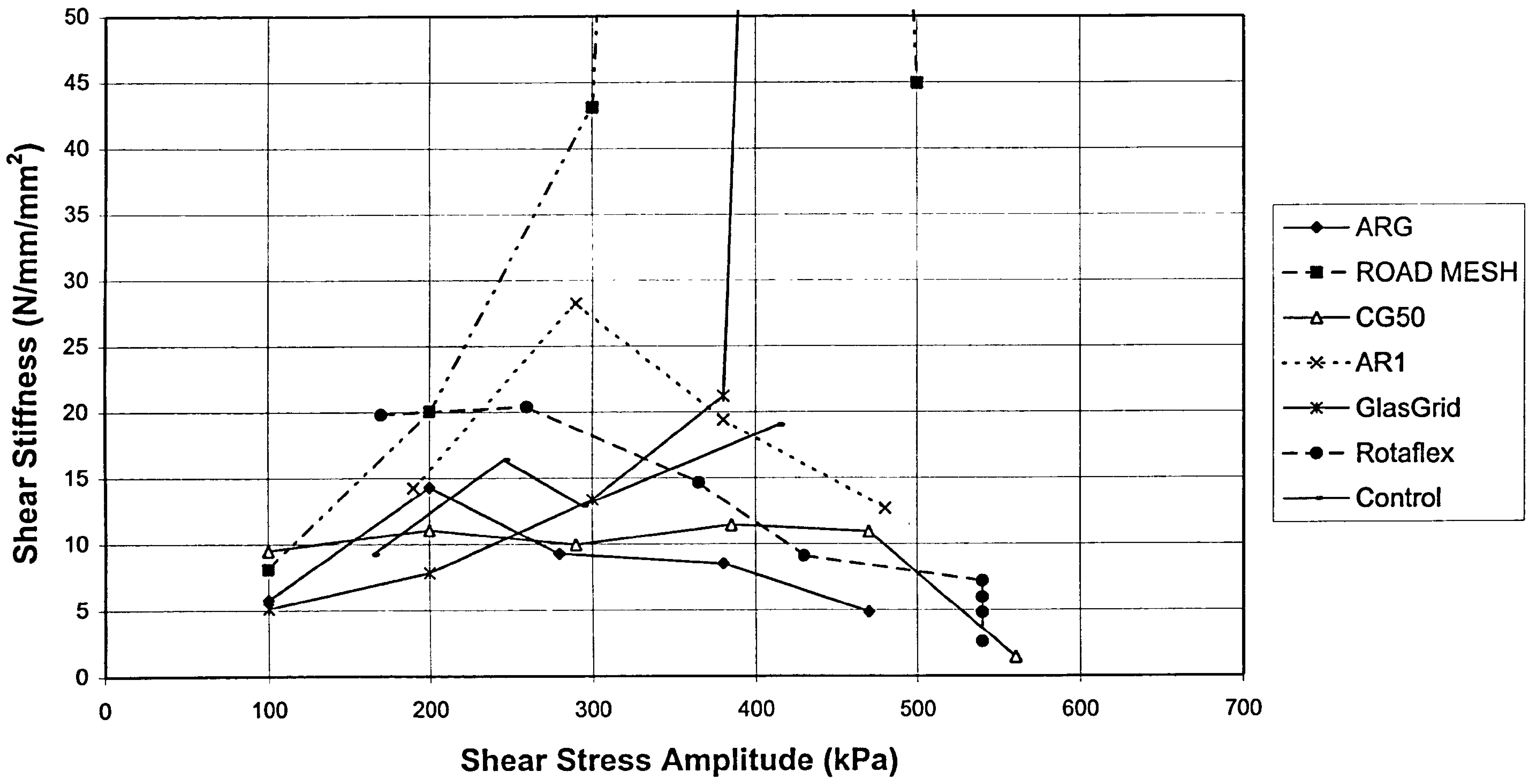

FIGURE 6.12
SHEARBOX INTERFACE STIFFNESS MEASUREMENTS

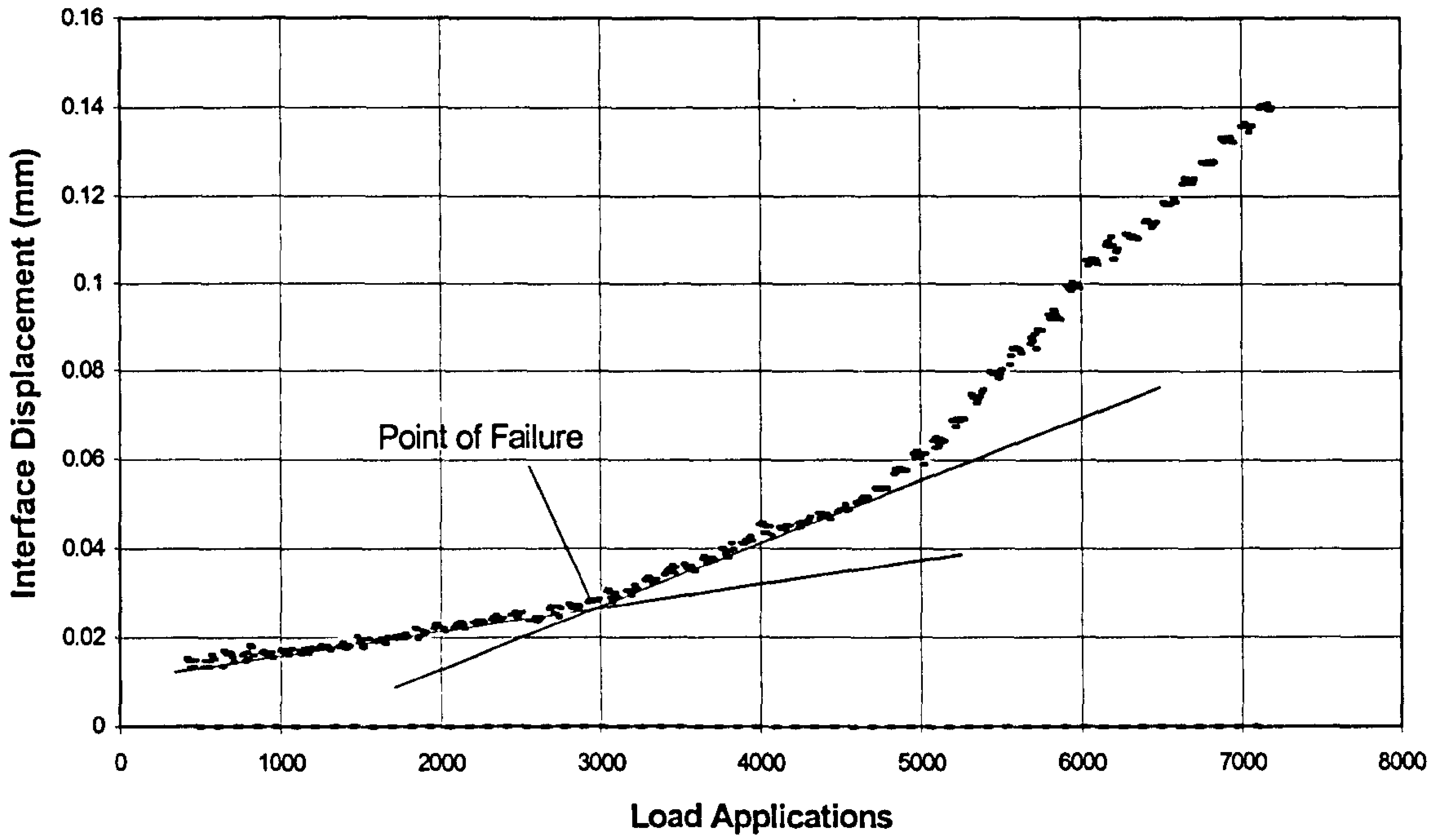

FIGURE 6.13
DATA INTERPRETATION – DEFINITION OF INITIATION OF INTERFACE FAILURE

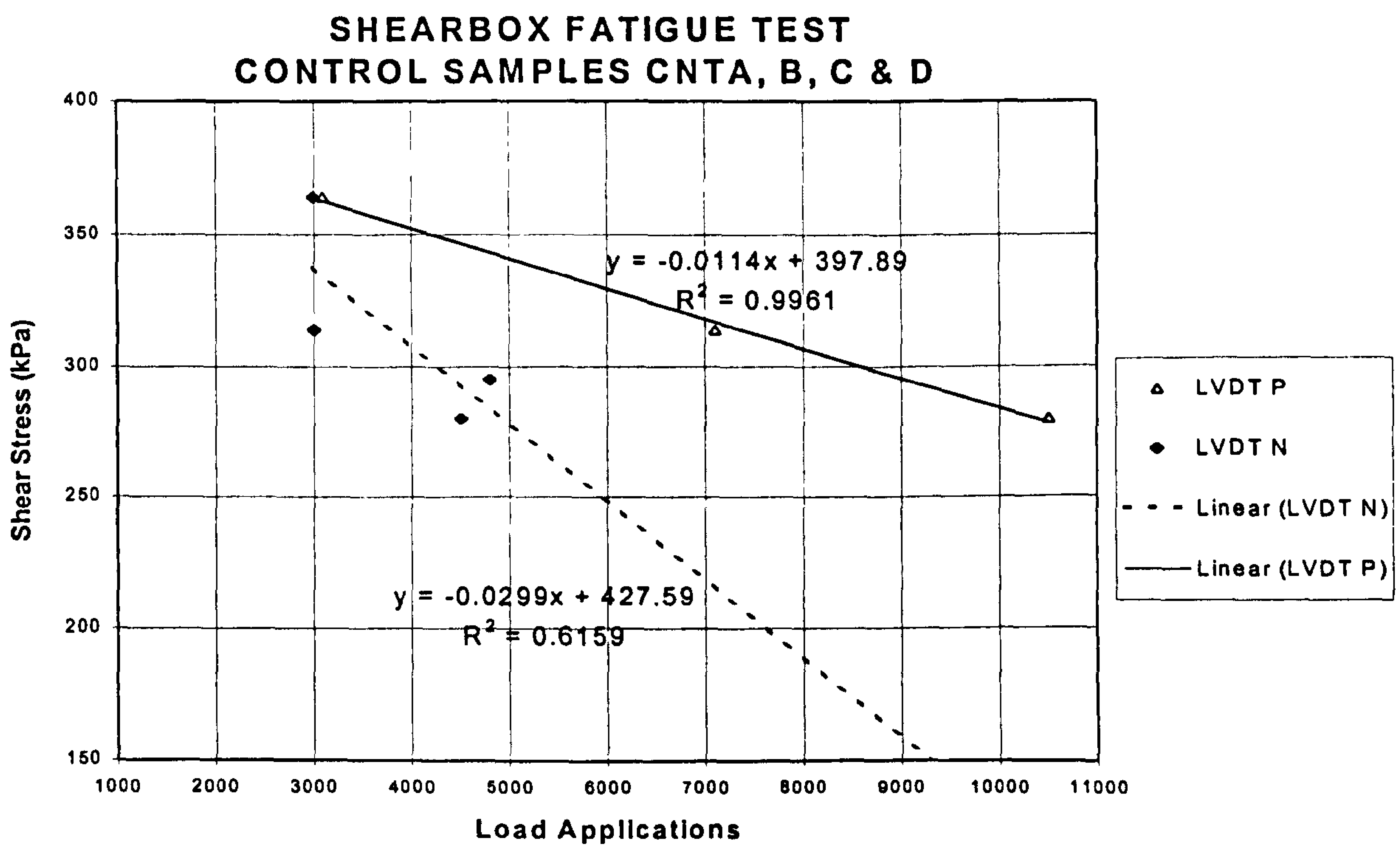

FIGURE 6.14
SHEARBOX FATIGUE TEST CONTROL SAMPLES CNTA, B, C & D

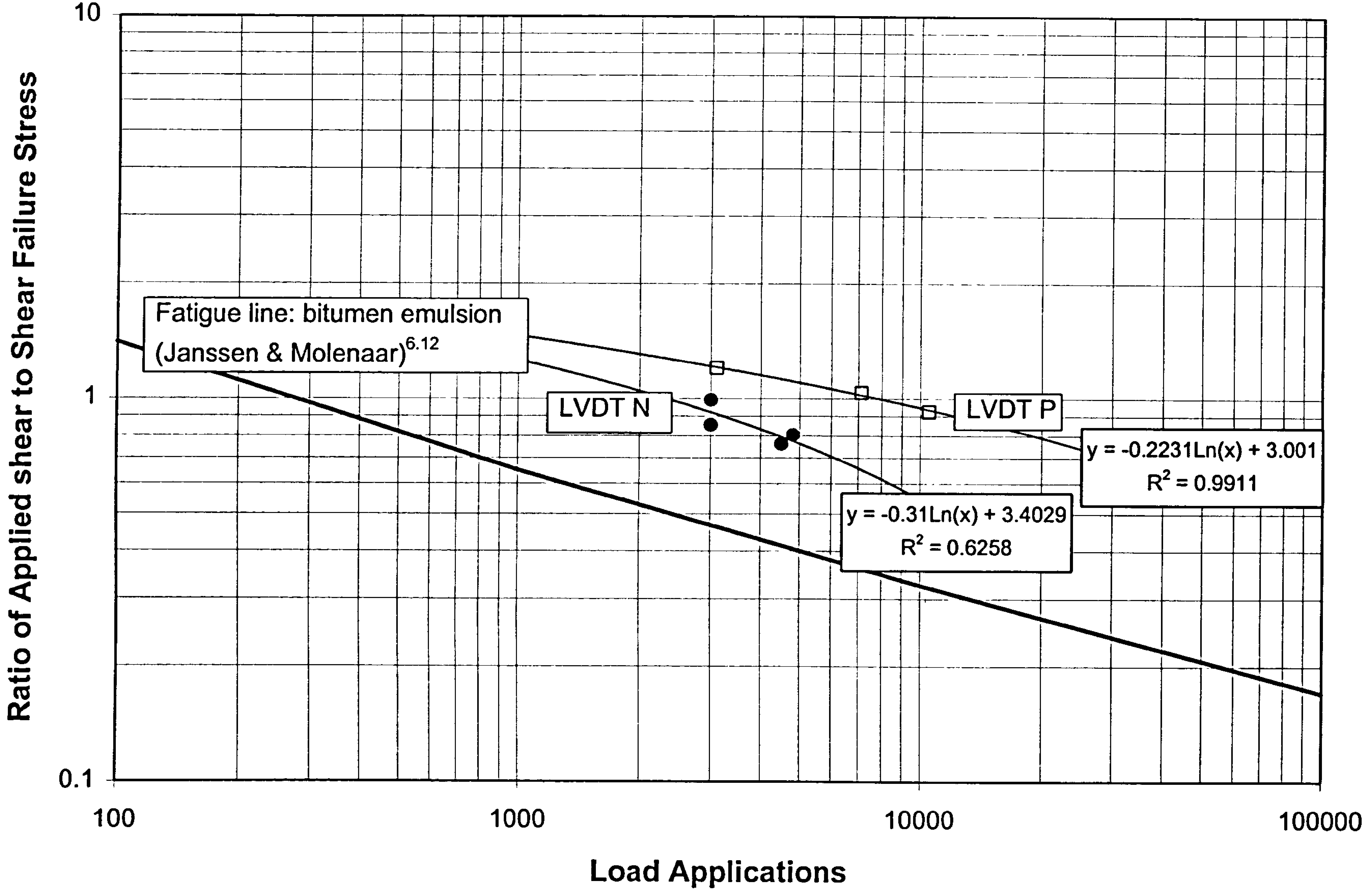

FIGURE 6.15

COMPARISON OF SHEAR-FATIGUE RELATIONSHIPS

APPENDIX 6.1 Void contents and densities of material tested with the shearbox

Sample	Top Layer		Bottom Layer	
	Density (Mg/m³)	Void Content (%)	Density (Mg/m³)	Void Content (%)
ROTAFLEX	2.47	3.09	2.52	5.06
GLAS GRID	2.44	5.15	2.37	8.01
CG50	2.47	4.0	2.45	4.86
ROAD MESH	2.37	7.9	2.37	7.97
CONTROL	2.43	5.5	2.38	7.0
AR-G	2.48	3.71	2.47	3.9
AR1	2.43	5.3	2.37	7.75
CNTA	2.49	3.32	2.49	3.3
CNTB	2.46	4.34	2.49	3.4
CNTC	2.44	5.28	2.47	4.17
CNTD	2.49	3.41	2.46	4.46

APPENDIX 6.2 Estimation of in-situ stresses

To estimate stresses applied by wheel loads on a pavement layer interface, the multi-layer linear elastic programme ELSYM5 [6.13] was used. Although it appreciated that there are considerable differences between a real pavement situation and the model used, such as materials that are not homogeneous and not having linear elastic properties, this approach has been used to good effect for pavement design in various countries over a number of years [6.14].

The model incorporated a 20 kN wheel load on a 4-layer pavement structure as shown in Figure A6.2. The material properties used in the model are given in Table A6.2.

Table A6.2 Material Properties used in the Multi-Layer Linear Elastic Model.

Layer No.	Layer Material	Thickness (mm)	Stiffness (MPa)	Poisson's Ratio
1	Surfacing Course	60	3500	0.4
2	Binder Course	100	5000	0.4
3	Subbase	200	150	0.35
4	Subgrade	Semi-infinite	50	0.35

Calculation gives a maximum shear stress of around 140kPa at the edge of the load (i.e. 113mm). However, in view of likely variations in material stiffnesses, loading and pavement thickness, it is considered that the maximum shear stress applied will be considerably higher than this. For testing therefore, a range of stresses should be used to cover most eventualities.

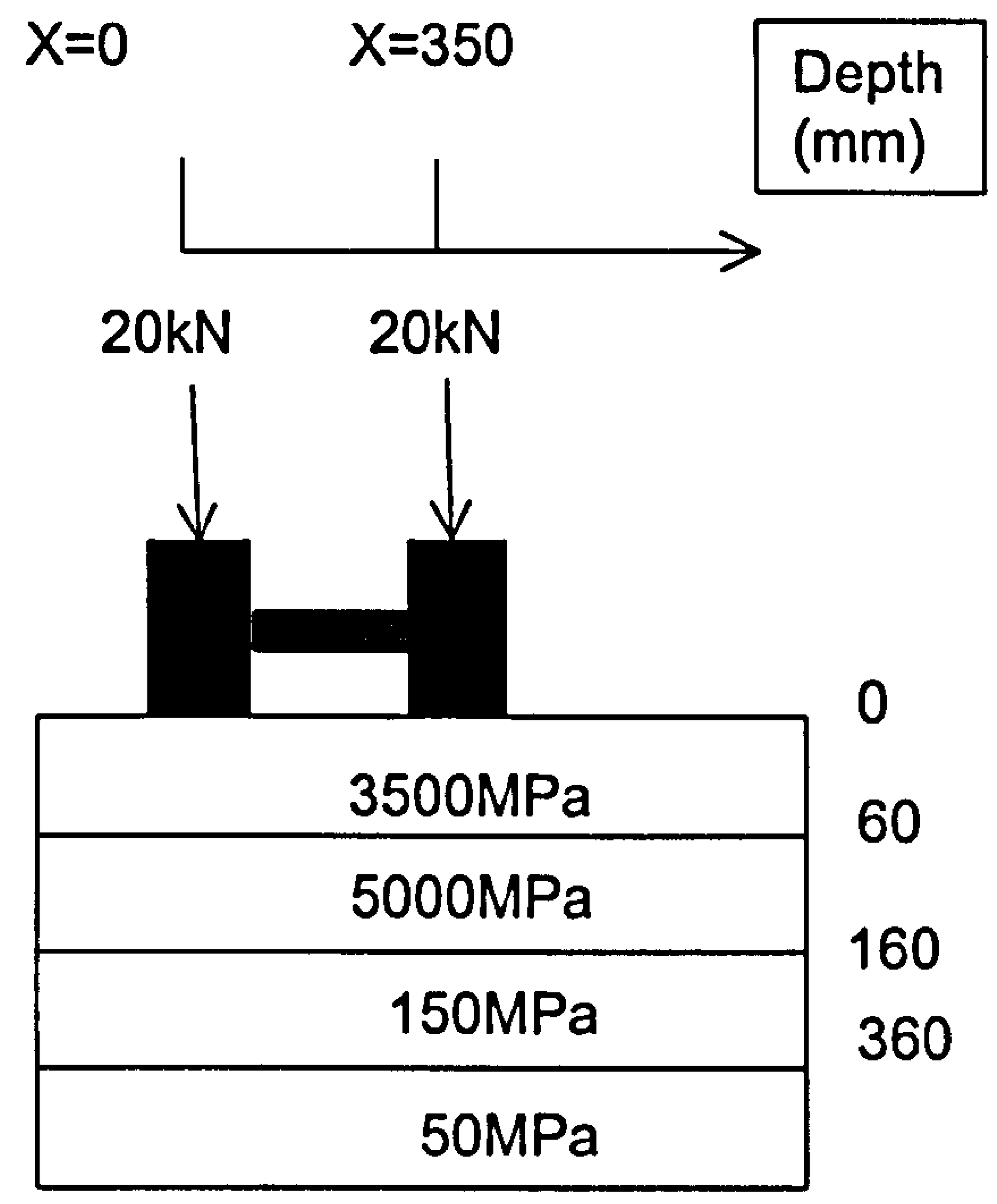

Figure A6.2
Pavement Structure and Load Configuration

APPENDIX 6.3 Correction of measured deflections for asphalt compression.

Table A6.3 Correction of measured deflections for asphalt compression.

Sample	Shear Stress (kPa)	Displacement			
		Measured (mm)	Correction (m*10^{-6})		Corrected (mm)
			38mm[1] sample	320mm sample	
	100	0.02	0.39	2.66	0.017
	200	0.02	0.89	6.03	0.014
ARG	280	0.03889	1.29	8.73	0.030
	380	0.05672	1.78	12.10	0.044
	470	0.1119	2.23	15.13	0.096
	100	0.015	0.39	2.66	0.012
	200	0.016	0.89	6.03	0.010
ROAD MESH	300	0.01636	1.39	9.40	0.007
	400	0.01412	1.88	12.77	0.001
	500	0.02727	2.38	16.15	0.011
	100	0.01316	0.39	2.66	0.011
	200	0.0241	0.89	6.03	0.018
CG50	290	0.03816	1.34	9.06	0.029
	385	0.04583	1.81	12.27	0.034
	470	0.05802	2.23	15.13	0.043
	560	0.4	2.68	18.17	0.382
	190	0.019	0.84	5.69	0.013
AR1	290	0.01933	1.34	9.06	0.010
	380	0.03167	1.78	12.10	0.019
	480	0.05333	2.28	15.47	0.038
	100	0.02222	0.39	2.66	0.020
	200	0.03158	0.89	6.03	0.026
GlasGrid	300	0.03186	1.39	9.40	0.023
	380	0.03	1.78	12.10	0.018
	480	0.01689	2.28	15.47	0.001
	590	0.02376	2.83	19.18	0.005

Note 1 This sample was tested to measure asphalt compression under load.

CHAPTER 7

BEAM TESTING

CONTENTS

CHAPTER 7 BEAM TESTING

7.1 Introduction

To investigate the effectiveness of interlayer reinforcement against cracking, a test was required where crack development could be quantified with different interface materials and bond conditions. Ideally, in-situ tests using measurements under 'real' traffic flows and axle loads would have been preferred, but, as the initiation and propagation of cracking is difficult to measure under these conditions, a laboratory test was considered necessary. The alternatives considered were two relatively large-scale wheel tracking test modes (the Pavement Test Facility (PTF) and the Slab Testing Facility (STF)) and beam testing. The decision to use beam testing for crack growth measurements was taken following a literature appraisal and carrying out trial tests with the STF. These tests showed that the test was more suited to rut development than cracking, and that making slabs of consistent quality was not simple, largely due to problems with compaction with a hand roller. Also, moving slabs from the location where they were constructed to the test apparatus led to premature cracking and, together with the difficulties of monitoring crack development, it was apparent that the test mode was not suitable. The difficulties of time and cost associated with preparing and carrying out PTF tests was the main factor that led to the PTF not being used for 'routine' testing. A brief summary of the literature review carried out to investigate previous reinforced asphalt work is now given.

7.2 Desk Study

Historically two main test modes have been used to appraise the performance of reinforced asphalt beams: one simulating the effect of traffic loading, i.e. relatively quick loading, and one modelling the effect of relatively slow movements of a pavement layer. Slow relative displacements of pavements may occur due to differential thermal gradients through the pavement, especially where different layer types overlie each other.

Traffic-induced loading was selected as the mode for investigation and modelling, as it appears to be the dominant mode in temperate climates such as the UK, (although environmental effects may still play a significant role in crack formation). The imposition of traffic stresses on a pavement is different to thermal stressing and is represented in Figure 7.1, which shows how shear stresses on an element reverse whereas vertical and horizontal stresses peak in one direction only.

A summary of information on the testing of reinforced asphalt beams obtained from literature is given in Table 7.1. For practical reasons and for brevity, only references relating to products generally available on the market have been referenced.

A summary of test results taken from references quoted in Table 7.1 is given in Table 7.2, and shows how the effects of different interlayers on crack propagation were quite varied. However, considering the range of products, test configurations and loading, this is not unexpected. An important observation is that all test results showed the reinforced asphalt to be more resistant to cracking than unreinforced material.

Table 7.1 Summary of selected beam tests

Reinforcing material	Test type	Loading Type	Grid/fabric position	Beam size/layer thicknesses	Organisation/Source
Polyfelt PGM14	Beam in a pivoted frame (combination of thermal and traffic loading)	Cyclic-vertical	20mm from base	Top layer = 40-60mm thick Bottom layer = 20mm thick	LRPC Autun-France.[7.1]
Tensar	Beam on split foundation[A]	Cyclic-Vertical	Base of beam	525 long x 150wide x 100 thick	University of Nottingham[7.2]
Glasgrid	Thermal and traffic simulation	Cyclic-Vertical & horizontal	19mm from base	325mm long, Top layer = 75-100mm thick Bottom layer = 19mm thick	Texas A&M University[7.3]
Heat-bonded Paving Fabric	Wheel tracking on sawn asphalt support[B]	Cyclic-vertical	20mm from base	300mm long, 80mm wide Top layer = 20-40mm thick Bottom layer = 20mm thick	Kansai University, Japan[7.4]

Table 7.1(Continued) Summary of selected beam tests

Reinforcing material	Test type	Loading Type	Grid/fabric position	Beam size/layer thickness (mm)	Organisation/Source
REHAU ARMAPAL	Beam on 'split foundation[1]	Cyclic-vertical	Base of beam	525 long x 150wide x 100 high	SWK Pavement Engineering (Nottingham)[7.5]
Polyester (grids)	3-Point Bending	Monotonic (vertical)	30mm from base	600 long x 140mm wide x 80mmhigh	Netherlands Pavement Consultants (NPC)[7.6]
Glasphalt	4-Point Bending	Monotonic (vertical)	30mm from base	600 long x 180mm wide x 90mmhigh	NPC, Schere & Partner Laboratory, Belgium Road Research Centre[7.7]
Glassfibre reinforced grids	4-Point Bending	Cyclic	30mm from base	600 long x 180mm wide x 90mmhigh	NPC and Chomorat (France)[7.8]
Steel (grids)	3-Point Bending	Monotonic (vertical)	30mm from base	700 long x 470-500mm wide x 90mmhigh	Netherlands Pavement Consultants (NPC) and Maccaferri[7.9]

Note A: 'Split foundation' refers to beams cast onto a support with two discrete halves.

Note B: Sawn asphalt support refers to a layer of reinforced asphalt on a layer of asphalt sawn through as a measure to induce reflection cracks.

Table 7.2 Summary of Beam Test Results

Reinforcing material	Test Mode	Benefit (Approximate)
Polyfelt	Fatigue Low temperature interface shear	Life increased by x 2 Asphalt tensile stress reduced by 50%
Tensar	Fatigue	Life increased by x 3
Glasgrid	Fatigue	Life increase of x1.5 to x 4.5
Heat-bonded Paving Fabric	Permeability Crack initiation	Large reduction in permeability Life increased by 1.5 to 3.0 times
Polyester Grids	Monotonic loading	Life increased by around x 7
Glasphalt	Fatigue	Life increased by around x 7
Glass-fibre-reinforced grids	Fatigue	Life increased by around x 10
AM6030 (Polyester) REHAU ARMAPAL ARM G (Glass-fibre)	Fatigue	Life increased by x 2.0 Life increased by x 10.0
Road Mesh	Monotonic Loading	Comparison with unreinforced beams was not carried out.

It is noted that asphalt reinforced with fabrics and polyester or polypropylene materials tends to have an average increase in life (as defined by cracking) of around a factor of 3, whereas the glass-reinforced products (except Glasgrid) have values up to 10. Taking this evidence alone, it would appear that the stiff glass-reinforced products could be recommended for reinforced asphalt applications. As will be shown, however, this is not always the case, especially when larger-scale samples (e.g. the Pavement Test Facility test sections) are tested. There are several possible reasons for this, the most likely being a combination of test mode differences and construction control. In particular the preparation (and especially compaction), of reinforced asphalt beam samples is easier to control than the construction of a pilot-scale pavement. Also, fundamental differences in the stiffness of glass reinforcement and asphalt is believed to play a part, as to mobilise the high stiffness of the glass the asphalt must be properly bonded to the reinforcement. Conversely, If the reinforcement is not properly bonded to the asphalt, then it may act like a separation layer and promote higher tensile strain in the asphalt. As beam construction is easier to control than the much larger pilot-scale pavements, this may be an important factor influencing test results, and leading to differences in performance between laboratory and field situations.

7.2.1 Implications of the Desk Study

The desk study has helped determine a suitable test configuration for investigation of the properties of reinforced asphalt by identifying the beam test to be (potentially) useful for determining reinforced asphalt behaviour. Even though large test specimens are probably more representative of in-situ behaviour, the relatively small-scale asphalt beams have been shown to provide noticeable and measureable differences in reinforced and unreinforced behaviour. Practical reasons also exist for the use of asphalt beams in an investigation rather than larger accelerated pavement tests, for instance. Small-scale beams are relatively straightforward to construct, affordable, and thus can be easily replicated, allowing systematic changes to be made or repeat tests to be carried out without incurring large costs or lengthy durations.

From the literature, it seems that laboratory investigations to determine crack propagation and fatigue properties are normally carried out using beams with crack initiators such as split foundations or notches. Also, typically, test specimens have been constructed in two layers, often between 70 and 100mm high, with a ratio of layer thicknesses of between 1:2 and 1:3 (lower layer:top layer). Test apparatuses that apply both horizontal and vertical loading appear useful in providing realistic (in-situ) combinations, but appear complicated to construct and difficult to analyse. With the assumption that test behaviour needs to be understood well before findings can be applied to field situations, test configurations that are as straightforward as possible are preferred.

Assessment of the points raised above, and practical issues such as use and modification of existing equipment, led to the 4 Point Bending (4PB) test being selected as a suitable test mode. The test is well-understood and thus suitable for analysis. Relative to the 3 Point Bending Test (3PB), the 4PB mode was preferred as it provides a configuration where cracks can propagate (in the central zone of the beam) under constant moment. The central load used in the 3PB test on the other hand, is expected to influence crack propagation, especially when cracks move into the compressive zone in the upper part of the specimen. A 4PB test was also preferred to a continuously supported test as the analysis of a 4PB test is relatively straightforward.

Although initially it appeared that the 4PB test mode was the most appropriate test mode, as testing progressed it was found necessary to alter the test configuration to obtain meaningful results. The development of the test equipment and test method finally used in beam testing is described in the next section.

7.3 Beam Apparatus Development

In developing the beam apparatus the first consideration was the scale of beams to be tested, and in this respect, two main factors were taken into account:

(1) Compatibility of the asphalt mixture with materials currently used in practice, and
(2) The suitability of material for use with the laboratory test apparatus, i.e. especially maximum aggregate size.

Point number one was considered important for the acceptance and application of any test findings, and the second point is very much a practical issue to ensure realistic test findings are obtained. This refers mainly to the need to construct layered samples that can be properly compacted, and that are not likely to lead to unrealistic failure modes. For instance, samples with large aggregate sizes may lead to compaction problems in relatively thin layers, leading in turn to premature failure.

Taking the above points in mind, and in the knowledge of previous test investigations, a 14mm Dense Bitumen macadam (DBM) mixture, commonly used in surface courses in the UK, was selected. The constituents of this mixture are specified in BS1497 [7.10] and are summarized, with information on preparation of the samples in Appendix 7A.

For compaction purposes the minimum layer thickness is normally taken as being two to two-and-a-half times the nominal aggregate size. Following this rule-of-thumb, sample layer thicknesses of approximately 30mm and greater, were considered suitable.

In addition to considerations of aggregate size for compaction, practical constraints of the roller-compactor apparatus were also taken into consideration, which meant that the maximum sample height was restricted to 120mm. Accordingly, the initial samples were constructed using a 40mm, and an 80mm layer.

The first beams used to develop the test apparatus were unreinforced and of dimensions L=400mm, H=120mm and W=200mm. A width of 200mm was used to accommodate sufficient stands of reinforcement for testing. Samples were cut from 400mm long and 280mm wide slabs which were compacted with the roller compactor apparatus. The 4-Point Bending (4PB) apparatus, shown in Figure 7.2, was then used to test the samples.

To initiate cracking in the central portion of the beam, a 4mm wide, 10mm deep notch was sawn in the samples across the centre of the beam. This notch was used to represent the effects of an existing crack in a pavement and helped to locate the LVDTs on the side of the beam that were used to monitor crack growth. The configuration of these LVDTs is shown in Figure 7.3. In addition, to measure the vertical deflection of the beam, an LVDT was placed beneath the centre of the beam.

Sinusoidal loading was applied at 5Hz at 20°C, and LVDT deflection and loadcell outputs were recorded with a data logger at 100 Hz. To smooth data and remove electronic noise, a digital (Hamming) filter was incorporated into the Visual Basic programme written to process data. A test frequency of 5Hz was eventually used as it gave good loading control (minimal noise on load wave forms) and reliable output response.

As testing progressed, a technique of monitoring crack growth was developed. Initially, a 'high quality' video camera was used but this was discontinued due to inadequate image resolution and the problem of requiring excessively long playing tapes. The solution adopted after trials with crack foils and additional LVDTs placed on the side of the beam, was visual inspection and manual recording of crack lengths on a paper replica of the grid pattern drawn on the side of the painted beam. Where appropriate, output from the LVDTs was used to help identify crack movement (see below). Crack inspection under magnification was carried out at half-hourly intervals. An example of a grid used for 'crack mapping' is shown in Figure 7.4.

The LVDTs were found to have limited usefulness in detecting crack growth due mainly to difficulties in positioning LVDTs to coincide with crack locations, and, also, in interpreting their output. To illustrate the problem, Figures 7.5 and 7.6 show plots of LVDT deflection versus loading repetitions for two different beams. The movement of a crack past the LVDT position is quite obvious in Figure 7.5, whereas Figure 7.6 shows a more gradual change of deflection. The reason for the more gradual LVDT movement seen in Figure 7.6, was probably the micro-cracking of the general area around the LVDTs as described by (*inter alia*) Jacobs [7.11], and Read [7.12]. Jacobs noted that a micro-crack zone precedes the macro-crack tip, the growth of which, (depending on the characteristics of the mixture and aggregate type), is often discontinuous. This, coupled with the manner in which micro-cracks are dispersed in the asphalt, can make the detection of a 'recognisable crack' in the early stages of its growth difficult.

7.3.1 Test Procedure

Once placed in the loading frame, a sinusoidal load with an amplitude of 10kN (+4.5 to –5.5kN) was applied. The results of the initial 3 specimens showed that, even though a notch had been cut in the bottom of the beam, cracks formed quite readily in the top of the beam. To remedy this and develop a satisfactory mode of beam testing, 23 beams were tested in various configurations as summarised in Table 7.3, which charts the development of the beam test configuration. The final test configuration consisted of a 90mm thick beam placed on a non-continuous 13mm rubber foundation, as seen in Figure 7.8.

Table 7.3 Summary of test mode development

Test Mode	**Problem**	**Solution**
4-point bending through zero loading on 120mm thick beams in stress-control mode (see Figure 7.6).	Top-down cracking	Apply permanent downward load
	Short test duration	Reduce load amplitude
	Excessive beam bending permanent deformation	Use continuous support conditions
Continuous support (stress control)	Large loads are required to deflect the beam, and therefore cause permanent deformation under loading platens. Creep deformation becomes dominant over cracking. Placing LVDTs beneath the beam to measure vertical deflection is difficult.	Reduce support under the centre of the beam and use pivoted aluminium plates (see Figure 7.7)
	Permanent Deformation under load platens	Strain control
Strain control	Inadequate servo-control system leading to excessive loading and subsequent damage to the beam	Test with stress-control
With the appropriation of the CAPA-2D programme, the modelling of 'non-standard' beam configurations became easier. Other support conditions, previously considered difficult to analyse by first principles, were therefore used (see Figure 7.9).		
Partial rubber support (Stress control)	Top-down cracking and permanent deformation under platens.	Use a Thinner (90mm thick) beam and a non-continuous rubber support. See Figure 7.8.

7.4 Test Results

Tests were carried out on 20 beams with the application of a compressive sinusoidal load between 0.5kN and 5.5kN. A list of beams tested is given in Table 7.4.

Table 7.4 Summary of Beam Tests.

Sample Type	Sample Reference	Reinforcement type
Control (unreinforced)	C1 C2 C3	Unreinforced
Steel-reinforced	S1 S2 S3	Meshtrack Road Mesh Road Mesh
Glass-reinforced Composite	CG1 CG2 CG3 CG4 CG5 CG6	CG50 CG50 PGMG2 PGMG ROTAFLEX ROTAFLEX
Polypropylene-reinforced composites	PC1 PC2	AR-G AR-G
Glass grids	GG1 GG2	RotaflexWG2303 Glas Grid
Non-woven fabrics	F1 F2	PGM14 PGM14
Polypropylene-Grids	PG1 PG2	AR1 AR1

Graphs of load repetitions versus crack growth derived from the 'Crack Maps' in Appendix 7B and load repetitions versus deflection are given in Figures 7.10 to 7.22. Initially several cracks started to propagate, before one crack became dominant and 'active', i.e. could be seen to open and close under load when viewed under magnification. It is the progression of this 'active' crack that has been used for analysis.

7.5 Analysis

7.5.1 Cracking

Two main types of cracks can be seen in the crack maps in Appendix B; one with vertical cracks dominating, and the other showing cracks growing horizontally at the interface before reorientating vertically above the interface. Cracks progressed differently on each side of the beams, with mixtures of both crack types present, making it difficult to assess the effect of the different reinforcement types. The literature suggests that effective reinforcement should either stop, or change the direction of cracks from vertical to horizontal. It was therefore considered that the length of horizontal cracking at the interface, taken together with the number of repetitions needed for the crack to propagate across the interface, would illustrate the effectiveness of the reinforcement. The use of the number of repetitions for crack propagation between crack lengths of 20 and 40mm (N40-N20) was thought appropriate as it removes the effects of crack initiation, (thought to be affected by the position of aggregate within the notch), and the effects of the compressive zone higher in the beam. The average length of horizontal cracking was therefore taken from the crack maps in Appendix B and plotted against (N40-N20) as shown in Figure 7.23.

Four main points can be derived from Figure 7.23:

(1) With the exception of steel- and fabric-reinforced beams, there is a general tendency for the length of horizontal cracking to increase with the number of loads applied.

(2) Unless the reinforcement type is taken into account, in general, the length of interface cracking is not in itself a measure of the effectiveness of reinforcement against crack progression.

(3) The length of horizontal cracking increases more gradually for polymer grids than it does for glass and fabric-reinforced beams and control beams.

(4) The crack patterns in steel- and fabric-reinforced beams develop differently to those in beams reinforced with other materials.

A common denominator of each of the two main groups of data points seen in Figure 7.23 could be interlock between reinforcement and the top layer of the beam. The polymer reinforcement and the steel grids have more pronounced profiles than the glass and fabric reinforced beams, and tend to have higher interlayer stiffnesses, as shown by shearbox test results. This may mean that the development of interface cracking depends on the shear or tensile strength of the interlayer bond. Direct tension tests on interface bonds on material cored from beams (described later in the chapter), show that the highest tensile strengths were obtained from steel- and Glas Grid-reinforced beams, and control beams. This may suggest that interface cracking is influenced more by shear bond than by tensile bond, as the control beam and the Glas Grid beam required fewer repetitions for a given length of crack.

Control beams also tend to crack vertically and thus have minimal horizontal interface cracking, but have lower values of (N40-N20) than the grid reinforced beams. The 'low' values of (N40-N20) with the glass-reinforced composites suggests that this relatively stiff interface material may encourage interface debonding to occur. Intuitively this is more likely to occur earlier in a test with a stiff interface than when materials have a comparable stiffness to the asphalt mixture, (as do polypropylene materials). This possibility has been investigated using Finite Element analysis in Chapter 9.

As a result of partial debonding at the interface, beams will have a tendency to act more as two thinner beams than as a single thicker beam. This leads to relatively high tensile strains developing on the lower interface of the top layer of the beam, which in turn encourages cracks to grow at a faster rate than would be the case if the beam acted as a single deeper beam. The relatively slow initial rate of vertical crack progression, (where cracks develop along the interface) followed by quicker vertical cracking is illustrated in Figure 7.16.

Aspects of interface bonding have been investigated and reported by Scarpas et al [7.13]. It was found that cracking tended to be vertical where pavement layers were connected with a strong interface bond, whereas with weaker interface bonding, cracking tended to horizontal. This agrees with the findings of the beam tests, and shear box interface bond measurements from the work described here, i.e. composite-asphalt and fabric-asphalt bonding tends to be weaker than grid-asphalt interlock bonds, thus helping to promote horizontal cracking at the interface. However, as is seen in the crack maps, typically a mixture of the two cracking modes occurs. This combination of crack modes was also modelled by Scarpas et al [7.13] who showed that the relationship between vertical and horizontal stresses and bond strengths defined the crack pattern in the vicinity of the interface.

Lytton and Jayawickrama [7.14] also observed both horizontal and vertical modes of cracking during beam tests during a test programme using similar sized reinforced asphalt beams to those used in the present study. Fatigue tests were carried out in a 4-Point Bending mode to determine fracture mechanics parameters, and tensile 'overlay' tests were carried out to simulate the thermal opening and closing of the joints in existing pavements beneath overlays. Failure in the fatigue test was characterised by a 'fracture process zone' where a network of microcracks developed ahead of a single macro crack that worked it's way through the sample from the zone of tension. 'Overlay tests' were carried out with beams epoxied to a split support, one side being fixed, and the other free to move horizontally. Beams were then loaded by opening and closing the moveable support plates between 0.25 and 1.0mm, thus inducing cracks through the specimen. Three modes of failure were defined from test results:

Mode A: 'Strain relief' where a crack moves from the bottom to the top of the beam, in a predominantly vertical orientation.

Mode B: 'Reinforcement' where a crack grows vertically to the interface, and then reorientates horizontally, between the underside of the reinforcement and the lower layer of asphalt.

Mode C: 'Strain relief ' where cracks propagate to the underside of the grid then cracks move down from the top of the beam.

It was reported that Mode B cracking occurred when the grid had a higher modulus than asphalt and sufficient cross-sectional area to reinforce the layer. This observation is not entirely consistent with the results of the present study where the samples reinforced with glass grids, for example, have predominantly horizontal cracks. As the test modes are different to the mode used in this study, however, more detailed analysis that takes into account bond strengths between the asphalt and reinforcement layer is required before any firm conclusions may be drawn.

Using fracture mechanics to interpret test results, Lytton and Jayawickrama estimated that Glassgrid extended the life against cracking by a factor of two to three, which, for the Texas conditions and materials, was equivalent to between 50 and 125mm asphalt.

To gauge the effect of reinforcement in the present study, measures of the number of load repetitions required for cracks to propagate to different heights in the beam were used. To estimate the effect of reinforcement on cracks across the interface, the number of repetitions required for the crack to propagate from 20mm to 40mm (denoted as (N40-N20)) was noted for each reinforcement type and compared. Likewise, for crack propagation above and below the reinforced interface, measures of (N50-N30), and (N30-N20) respectively were used. The results are given in Table 7.5.

Table 7.5. Comparison of the average effects of reinforcement on crack propagation (in load repetitions per mm propagation)

Position	Parameter	Glass	Polypropylene	Steel	Control
Below interface	N30-N20	2324	15277	20684	7220
	Ratio (with control)	0.32	2.12	2.86	1.0
Across interface	N40-N20	5346	14358	12507	5281
	Ratio (with control)	1.01	2.72	2.37	1.0
Above interface	N50-N30	5899	13024	7005	4367
	Ratio (with control)	1.35	2.98	1.6	1.0

The results show that all reinforcement has an effect on crack propagation, especially above the interface. It is interesting to note that it appears that the stiff glass-reinforced samples reduce crack resistance below the crack.

Whereas an effect was expected across and above the interface (i.e. on N40-N20), the influence of reinforcement below the interface (N30-N20) was not expected. More testing is required to establish whether this apparent phenomenon actually exists. Further investigation may then be required. The glass- and the polypropylene-reinforced beams are seen to become more effective as the crack moves above the interface, whereas the steel reinforcement makes the largest contribution when cracks are below the interface. This may mean that the combination of good asphalt-grid interlock and stiff reinforcement increases the overall beam stiffness sufficiently to reduce tensile strain and thus crack initiation. The general increase in effectiveness of polypropylene-reinforced beams as cracks move up through the beam could mean that more movement is required to mobilise the effect of this type of reinforcement.

The effect of **asphalt density** and **aperture size** on crack growth rate is illustrated in Figures 7.24-7.26. A more distinct trend between density above the interface and N50-N30, than density below the interface and N40-N20 is apparent. The reason for this is probably linked to the difficulty in compacting thin layers to a fine tolerance, i.e. a small change in thickness can make a large difference in density. Also, it is possible that by compacting the upper layer over the lower asphalt layer, aggregate is arranged differently than when compacted as a thin layer over a steel plate. In general, however, it seems that density has little influence on the rate of crack growth across the interface.

In contrast to Figures 7.24 and 7.25, however, the plot of minimum aperture size against (N40-N20) in Figure 7.26 gives an indication that crack resistance improves as aperture size increases. Also, it appears that the effect of aperture opening has more effect on crack resistance than does grid strength or stiffness, i.e. the polypropylene shows higher values of (N40-N20) than those of the glass-reinforced grids, even though this material is weaker and less stiff than the glass grids. The reason for this may be linked to the continuity of asphalt between the top and bottom layers of the beam, which should be better for the large-aperture grids, and in turn may also lead to better interlock. The effect of aperture size, however, does not seem to be dominant in interlayer shear resistance, as measured by the dynamic shearbox (see Chapter 6), where the GLASGRID sample was seen to have a relatively high shear resistance. This suggests that other factors such as the effect of tensile strength and grid properties also need to be taken into consideration when assessing crack resistance.

Tensile bond strength across the interface has been measured on cores taken from beams and Pavement Test Facility (PTF) test sections (described in Chapter 8). The results of tests on cores taken from beams is seen in Figure 7.27 and shows that tensile failure stress increases with test rate and that unreinforced material has a relatively high tensile strength. In general, it was noted that grid-reinforced materials tended to fail on the bottom interface, whereas composite materials would fail partially on both top and bottom interfaces. A summary of the failure modes is given in Table 7.6.

Table 7.6. Summary of Tension Test Failure Modes

Reinforcement type	No. of failures on top Interface	No. of failures on bottom interface	No. of 'part failures' i.e. on both top and bottom interfaces
AR1	0	6	0
Glasgrid	0	5	1
MeshTrack	Failure in Slurry interface		
Roadmesh	See note 1 below		
Rotaflex	0	3	3
WG2303	0	3	3
PGMG	0	0	5

Note 1 Failed interfaces had a rough and irregular shape, and failure occurred both above and below the grid on each sample. Grids tended to be better embedded in the top layer.

Test results for cores taken from beams show the composite-reinforced asphalt to have a lower tensile strength than the grid-reinforced beams, although the AR1- and WG2303-reinforced beams appear to have similar values to the composite-reinforced beams. This could be due to natural variability of materials and construction practices, and the fact that WG2303 material has a grid aperture of only 26mm, which, relative to the nominal 14mm DBM mixture aggregate, is still small, i.e. Hozayen et al [7.15] recommend that the aperture be between 3 and 4 times the size of the nominal aggregate size for good performance. The high values for the GLASGRID seem to be due to the adhesive covering on the material, which adhered very well to the asphalt and resulted in the GLASGRID material pulling apart in preference to failure of the bond between the asphalt and the grid.

Read [7.12] found that crack propagation was related to the initial (pre-fatigue test) asphalt stiffness, and in particular that a lower stiffness mixture is more crack resistant than a mixture of high stiffness. Accordingly, the possibility of asphalt stiffness having an influence on the beams tested was investigated using results of the Nottingham Asphalt Tester (NAT) with material cored from the beams. Figure 7.28 is a plot of (N40-N20) against the Indirect Tensile Stiffness Modulus (ITSM) of the asphalt, and shows that in general there is no consistent relationship. This confirms the essential consistency of the specimens, and the variation found is often found with these relatively small elements of material.

7.5.2 Creep deformation and cracking.

Loading in the beam tests included both dynamic and static loading components, and to investigate whether the component of dead load caused cracks to initiate and propagate, a limited investigation into the effect of the static load on cracking was carried out. This consisted simply of applying a dead load, to both reinforced and unreinforced beams.

Two beams were constructed and set-up in the same way as the dynamically-loaded beams, except for the substitution of two dial gauges for the LDVT used to measure vertical deformation, and the omission of LVDTs on the sides of the beams. The mean compressive load applied in the dynamic test (3kN), was applied to the beam was imposed using metal weights, and a crack map was used to record cracking, in a similar way to the dynamic tests.

A plot of deflection versus time is given in Figure 7.29 and shows the AR1-reinforced beam initially deforming quicker than the unreinforced sample, then from around 3mm deformation, deforming at a much slower rate. The unreinforced beam shows a more gradual increase in deformation, with deflections only becoming relatively constant at around 5.5mm. It is thought that the initial quick increase in deflection for the AR1 reinforced beam, followed by a relatively slow increase in deformation was due to a degree of slip between layers occurring before the asphalt-grid interlock became effective.

As the stiffness of polypropylene is greater than that of the DBM mixture used (at such slow strain rates), the grid reinforces the beam in a conventional manner. This is dependant on the effect of interlock between layers. In this respect, the AR1 may have provided more interlock between layers, thus effectively deepening the beam and helping it to deflect less than the unreinforced sample.

Regardless of the above comments on the deflection behaviour of the beams, the main purpose of carrying out the creep tests was to investigate whether the static load significantly influenced cracking. In this regard, no definite cracking was recorded during the testing, although there was general crazing of the painted surface as deformation increased, which was more concentrated under and adjacent to load platens.

The lack of significant cracking in either beam beyond the general faint cracking of the paint as the beam deformed suggests that cracking measured during dynamic testing was due to fatigue and not creep movement.

7.5.3 Deflection

From Figures 7.11, 7.13, 7.15, 7.17, 7.19 and 7.21 (plots of deflection versus load repetitions) there appear to be three possible deflection curve types that correspond to grid, reinforcement, composite reinforcement, and unreinforced beams. Plots of deflection for reinforced beams tend to be more elongated and have an 's'-shape (see Figure 7.21), whereas unreinforced beams have two main parts to the curves (see Figure 7.11). The behaviour of fabric and composite-reinforced beams tend to lie between these categories, and shows a large variation in behaviour.

There seems to be some general distinction between the deflection curves of the different material types. In particular, glass-reinforced beams tend to show a relatively quick increase in deflection after an initial slow increase (see Figure 7.17). The steel-reinforced beams on the other hand appear very similar over the first 200,000 repetitions before a marked, but less rapid (than glass-reinforced beams) increase in deflection occurs. The deflection of polypropylene-reinforced beams varies considerably, but with similar shapes. The plot of deflection for beam PC2 seems inconsistent to the others, having a quicker increase in deflection after 80,000 repetitions or so. Also, the initial deflection is higher than the other beams which may suggest that damage to the interface bonding had occurred. The relatively sharp increase in deflection at around 80,000 repetitions, is consistent with the crack growth measurements seen in Figure 7.14.

It is noted, that there are general similarities between crack propagation rates and deflection plots. However, prediction of crack growth using deflection data would be difficult and inaccurate.

7.5.4 Interface condition

The condition of the interface, and in particular, the adhesion or interlock of the reinforcement to the asphalt, is probably the single most important factor determining the behaviour of reinforced asphalt structures. Also, the most cited reason for poor performance of reinforced asphalt appears to be construction-induced defects. Consequently, the possibility of defects in the reinforced interface having an influence on crack patterns was of interest during beam tests. Unfortunately, during testing, the behaviour of the reinforced interface could not be directly monitored with the instrumentation used. Also, it was not known whether the construction procedures had in fact led to any defects in the reinforcement, and thus, what their effect on beam performance might be. In attempting to resolve this issue, a visual inspection of the interfaces of six beams was carried out after testing. Interfaces were exposed by splitting beams apart at the interface, after cooling them to –5°C. Beams reinforced with polypropylene, steel, fabric, and glass were selected, and images of the interface were recorded using a digital camera.

Overall, the interfaces appeared to conform to what was aimed for, i.e. reinforcement was found in the same position as laid, before placing the upper layer of asphalt. Fabric appeared well-bonded and grids had good interlock with the asphalt. However, damage to a glass-reinforced grid (WG2303) was noted in one case, as was limited voiding around a twisted wire junction for a Road-Mesh sample. Images of these 'defects' are shown in Appendix 7C, as well as examples of the fabric-reinforced interface, and a polypropylene composite-reinforced interface. A summary of observations is given in Table 7.76.

Table 7.7 Principal Visible Features of Beam Interfaces

Main Interface Component	Sample	Interlayer Adhesion to the top or bottom asphalt layer	Visible Voids	General interlayer appearance
Polypropylene grid and composite	PG2	Top	Very few and small	Regular aperture spacing
	PC2	Grid on top layer, fabric on bottom[1]	None	
	PC1	Top	None	
Fabric	F1	Top and bottom	None	Smooth
Glass-grid	GG2	Top	None	Damage to glass fibres[2]
Steel-grid	S3	Top	Yes-around wire twists	Well-embedded in top asphalt

Note 1 The bond between the grid and the top asphalt layer, and the fabric bond with the bottom asphalt layer was such, that when the sample was split, the fabric was torn from the grid.

Note 2 One transverse rib (comprising three bundles of fibres) was found to be broken. There appears to be no correlation between crack patterns and the position of the broken rib.

From the images, and observations made during the visual inspection, it appears that there were insufficient defects in the beam interfaces to have any noticeable effect on beam performance. Also, the limited defects found in the steel and glass-reinforced beams have no recognisable correlation with the cracking noted on the beam sides. The voids found in the vicinity of the twisted wire in the steel grid suggest that an asphalt mixture needs to be sufficiently workable to compact around the junctions, as well as having a maximum stone size and grading compatible with the aperture opening.

As was expected, the fabric-reinforced interlayers were well-bonded with both top and bottom asphalt layers, reflecting the ease of construction. This is probably one of the main reasons why fabrics are a popular choice for reinforced asphalt construction.

Considering the need for reliable installation of reinforcement, and definition of the degree to which it affects beam performance, further work should be carried out in this area. In particular, reinforcement 'laps' and joins and 'defects' such as broken reinforcement and areas of poor bond and/or voids could be built in to test beams or pavements, and their effect on beam performance measured.

7.6 Conclusions

- Beam testing has indicated that crack progression is affected by the presence of reinforcement, and its bond with asphalt.
- The partially supported beam test has proved suitable for evaluating the effect of different reinforcement types.
- Partially supported beam tests suggest that cracks may take up to three times longer to progress through a well-constructed reinforced interface than for an unreinforced interface.
- Plots of deflection versus load repetitions indicate that deflection behaviour tends to be different for unreinforced beams, and those reinforced with glass-reinforced composites, and grid-reinforced beams. Beams reinforced with steel and polypropylene grids and composites show more gradual increases in deflection than do the glass-reinforced and unreinforced beams. The breakdown of interlayer adhesion bond is thought to be responsible for the sudden increase in deflection, whereas the interlock between steel and polypropylene reinforcement and asphalt seems to be more durable.
- Rates of crack propagation were similar to the rates of increase of deflection, and showed broad trends of polypropylene and steel-reinforced samples propagating more gradually than cracks in unreinforced, and glass-reinforced samples. The rates of crack propagation of the control beam cracked between 2 and 3 times faster the rate of the steel and polypropylene samples, and between a third and 1.4 times the rate of glass-reinforced beams.
- Various patterns of cracking were observed that generally correlate with different interface materials, i.e. grid-reinforced, and unreinforced beams tend to have cracks that are vertically-orientated, whereas fabric and composite-reinforced beams often had a large component of horizontal cracking, typically between the lower layer of asphalt and the interlayer material.
- To achieve good interlock between grids and asphalt, apertures must be sufficiently large to promote good interlock with aggregate in the asphalt. This in turn depends on the aggregate size in the asphalt mixture. A ratio of aperture to nominal aggregate dimension of at least 3 or 4 has been suggested as being appropriate, and is compatible with test results.
- Results indicate that crack resistance is dependant on a number of factors including tensile and shear bond strengths between asphalt and reinforcement, and not just the reinforcement type, strength or stiffness.

- From observations of beam interfaces after fatigue testing, in general, reinforcement bonded better to the top layer of asphalt than to the bottom.

7.7 References

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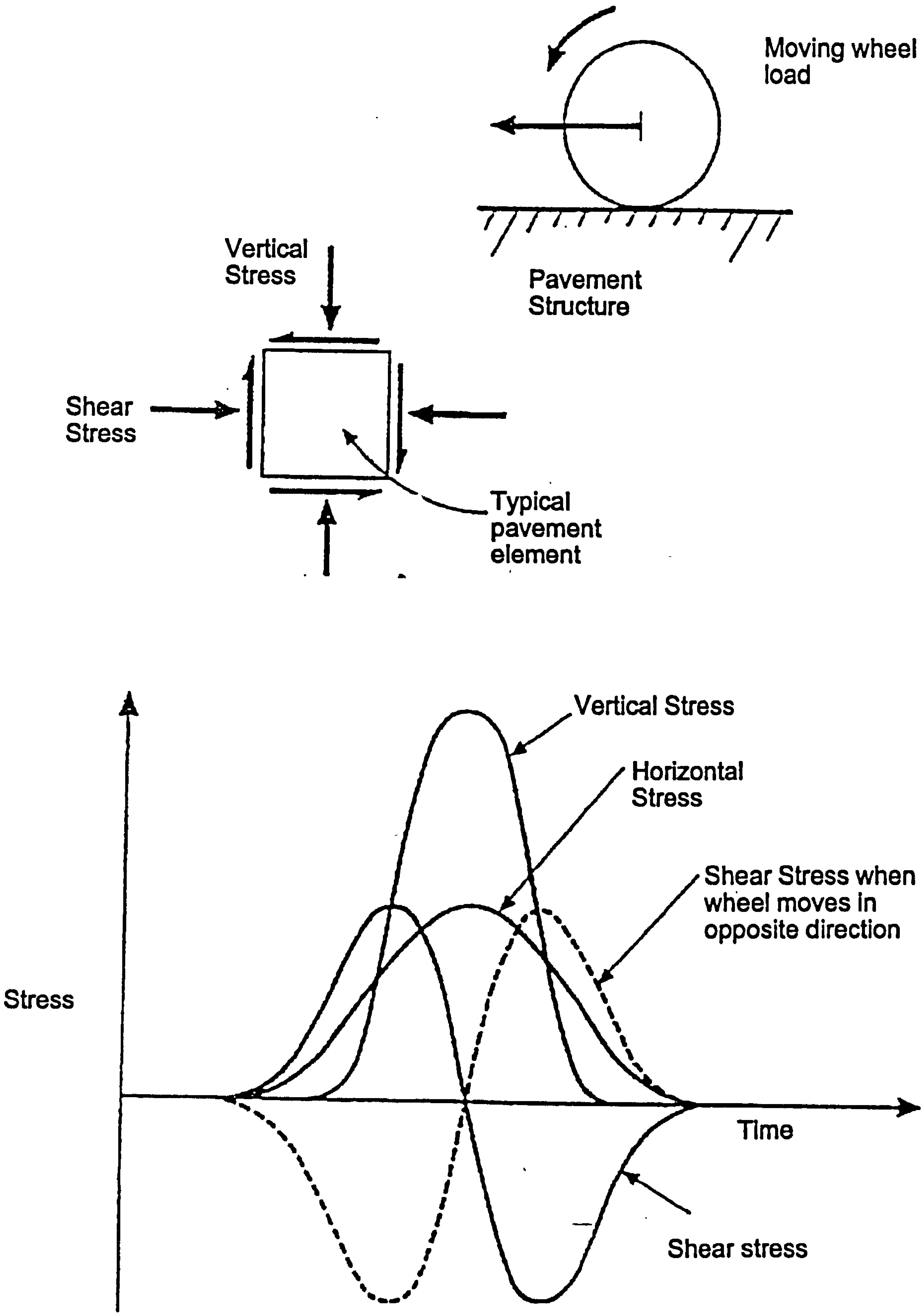


FIGURE 7.1
WHEEL-INDUCED STRESS REVERSAL

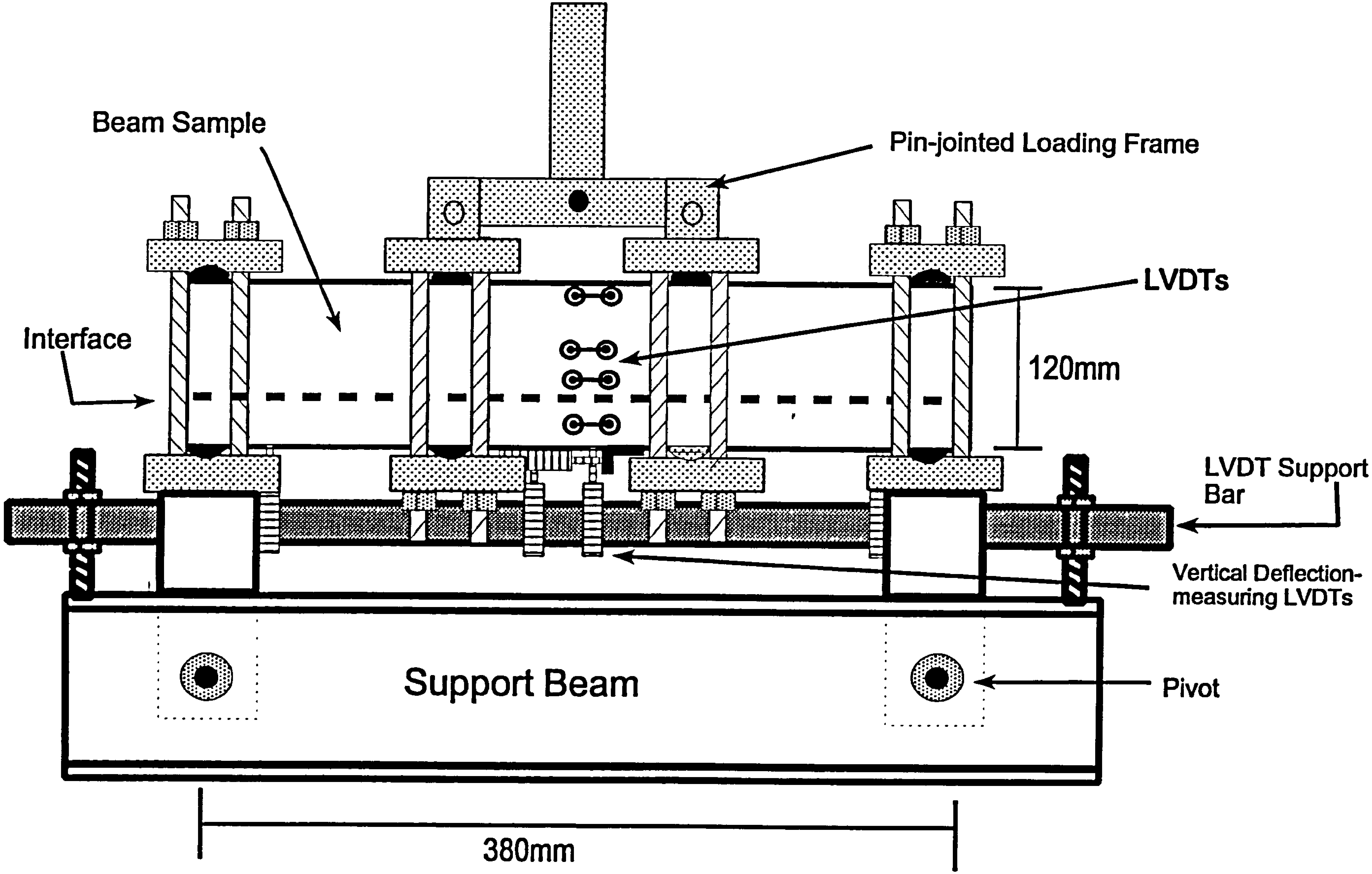
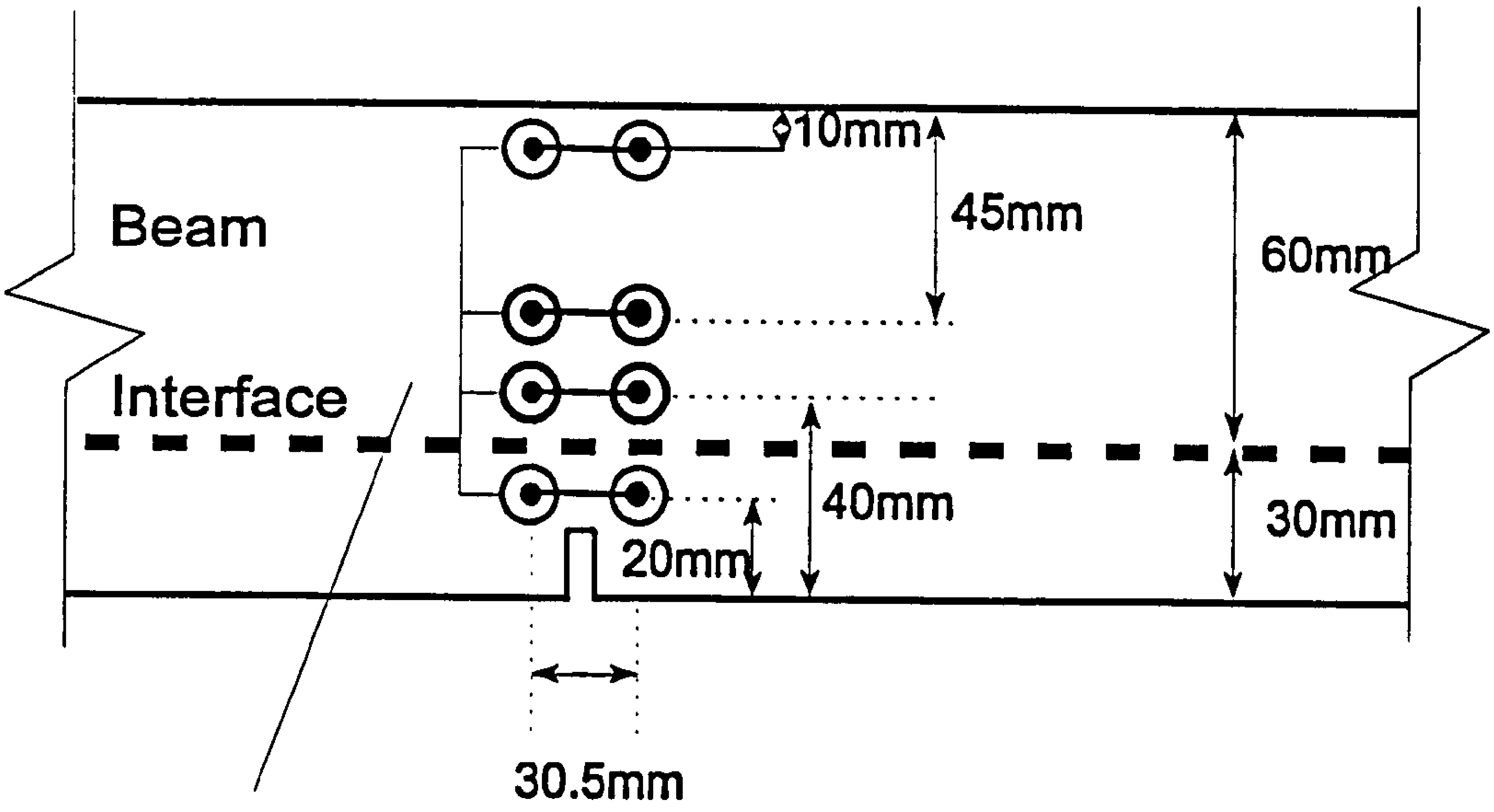
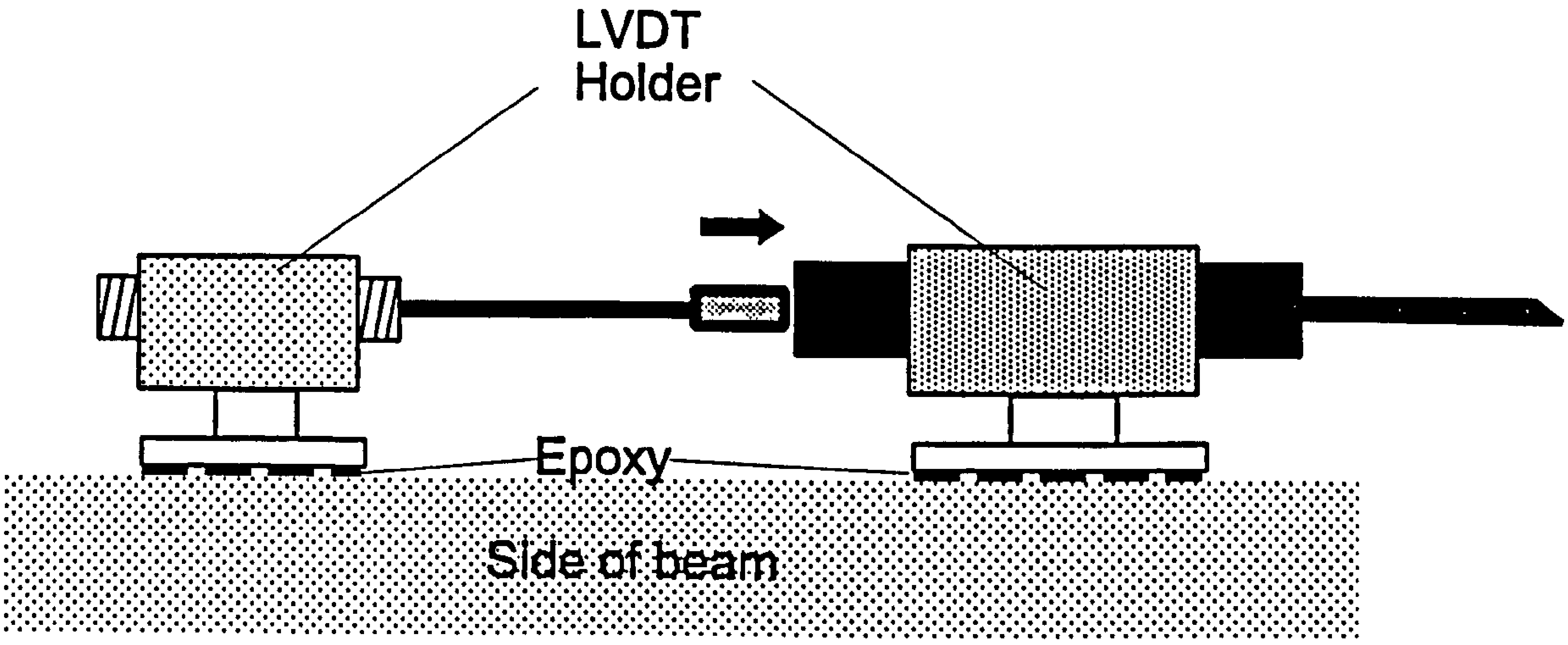


FIGURE 7.2
4-POINT BENDING APPARATUS



Side Elevation



Detail of LVDT configuration

FIGURE 7.3
CONFIGURATION OF LVDTs FOR CRACK DETECTION

SPECIMEN:-
FRONT/BACK:-
TEST MODE:-
DATE:-

MARKER:-																																
REPETITIONS:-																																

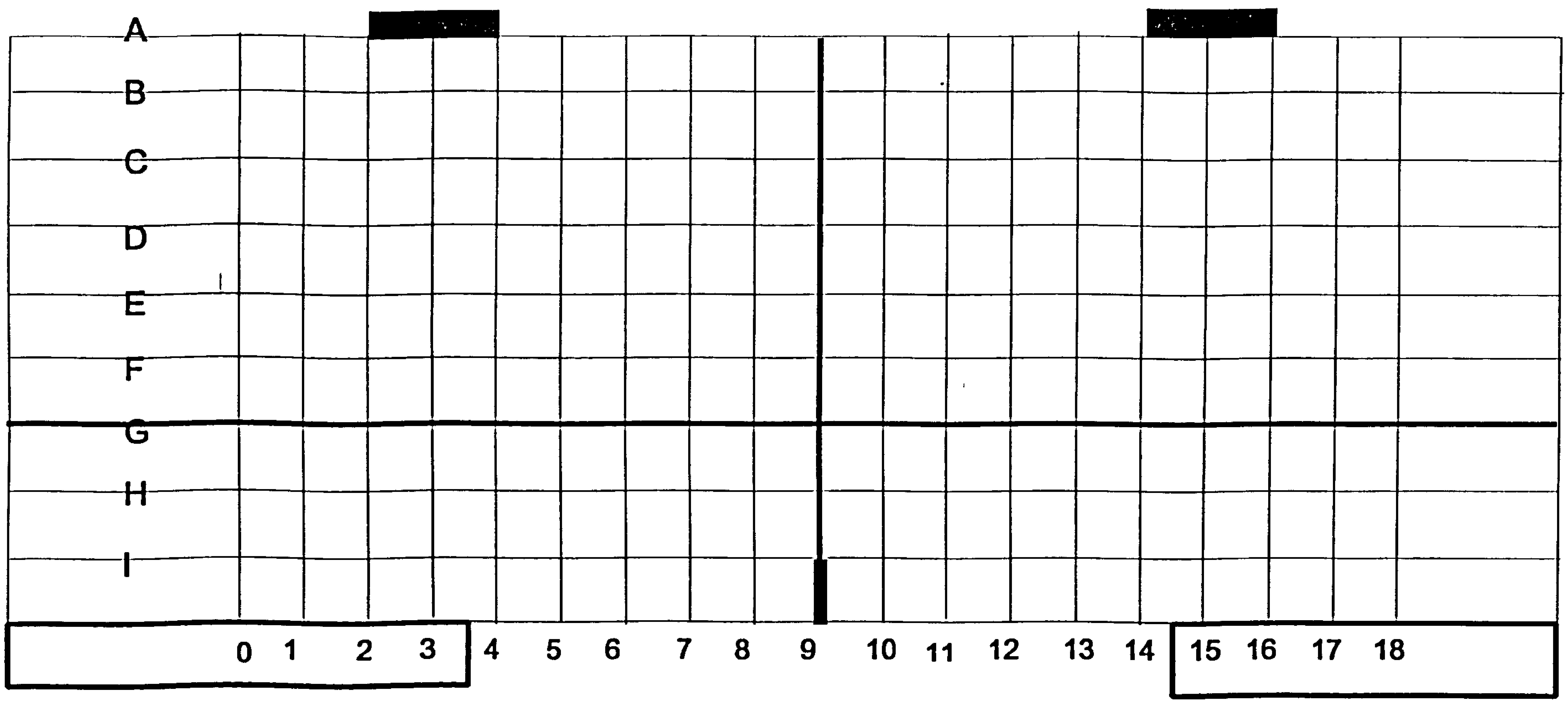


FIGURE 7.4
GRID USED FOR CRACK MAPPING

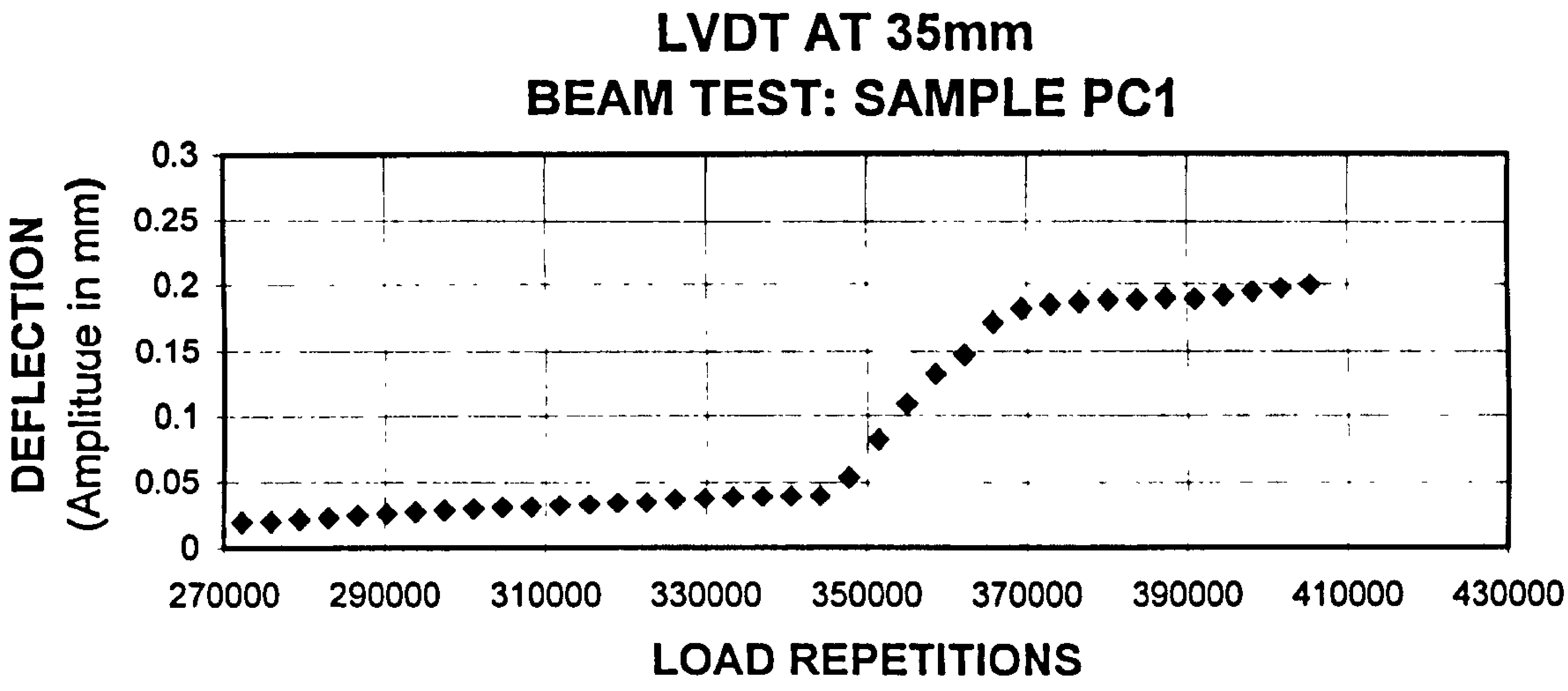


FIGURE 7.5
EXAMPLE OF WELL-DEFINED CRACK PROGRESSION

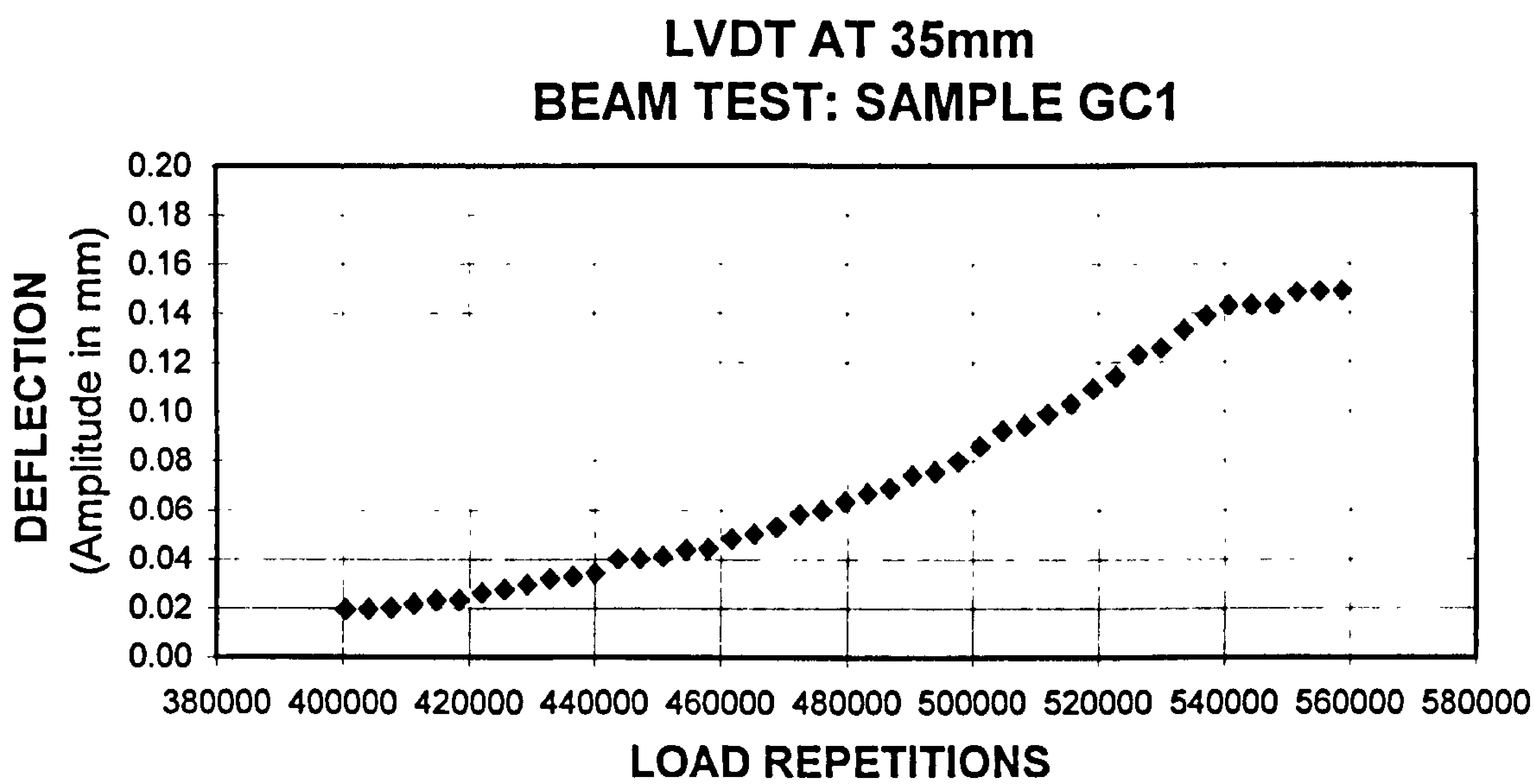


FIGURE 7.6
EXAMPLE OF POORLY DEFINED CRACK PROGRESSION

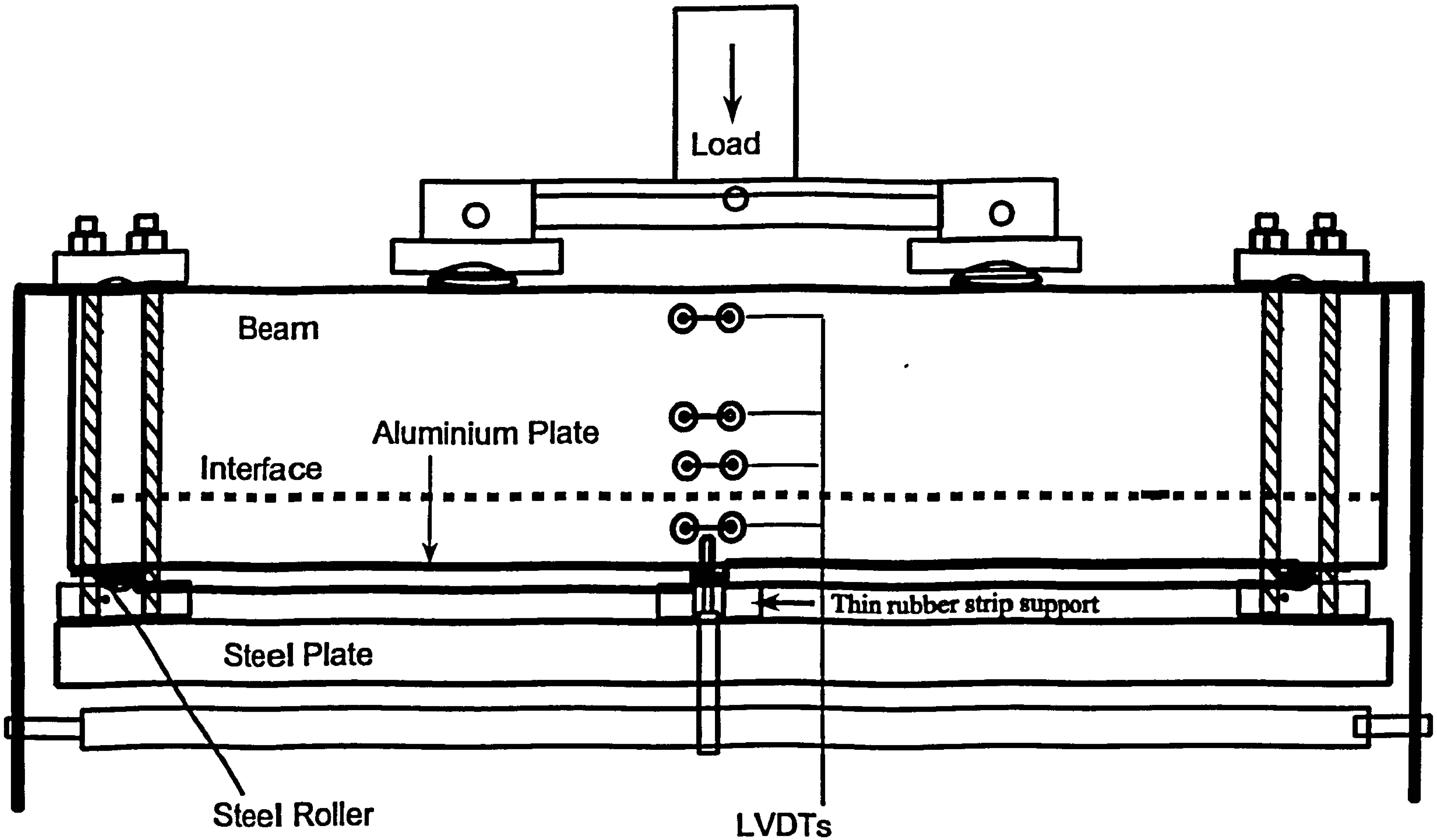


FIGURE 7.7
INTERIM BEAM TESTING CONFIGURATION

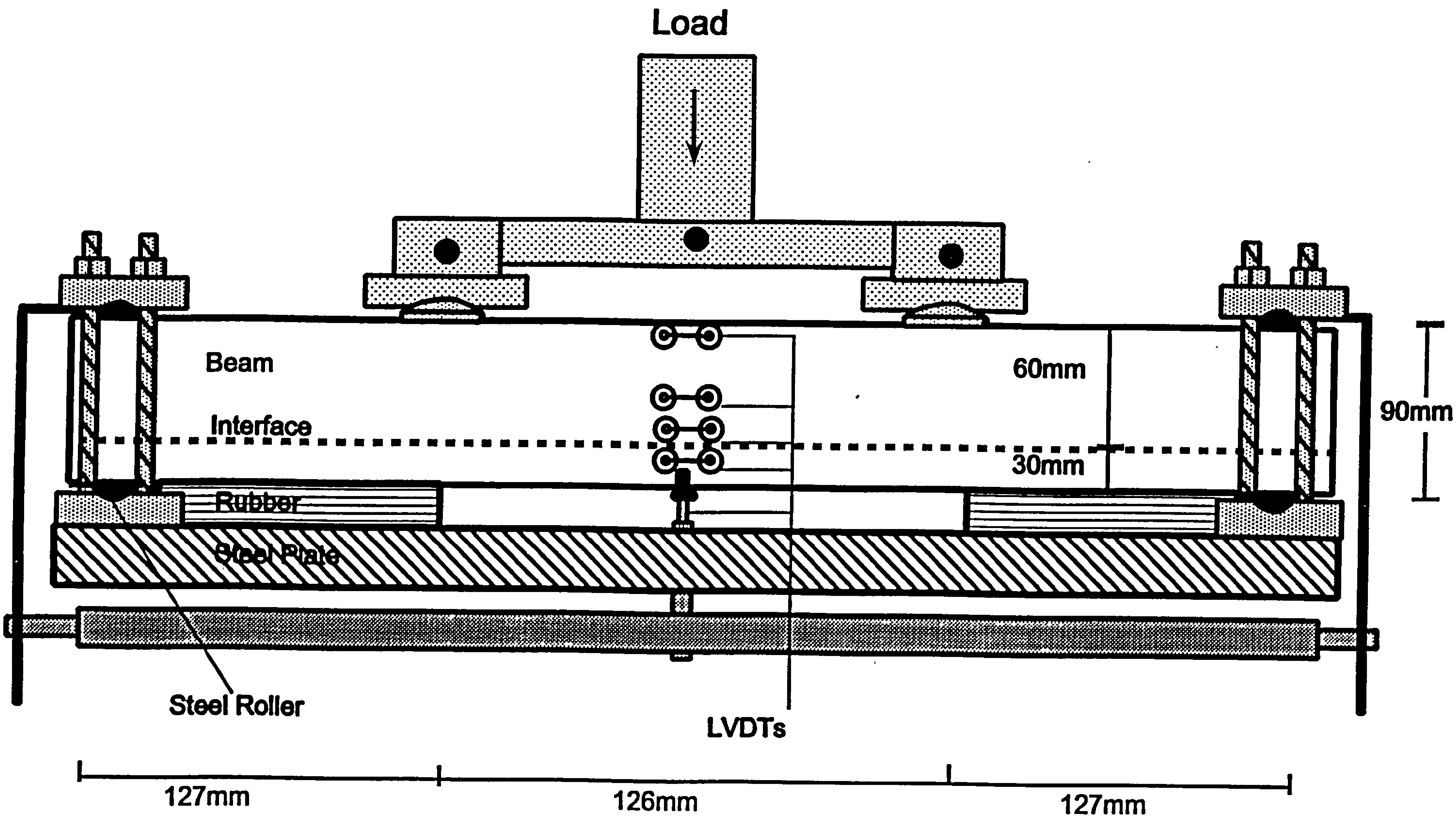


FIGURE 7.8
FINAL BEAM TESTING CONFIGURATION

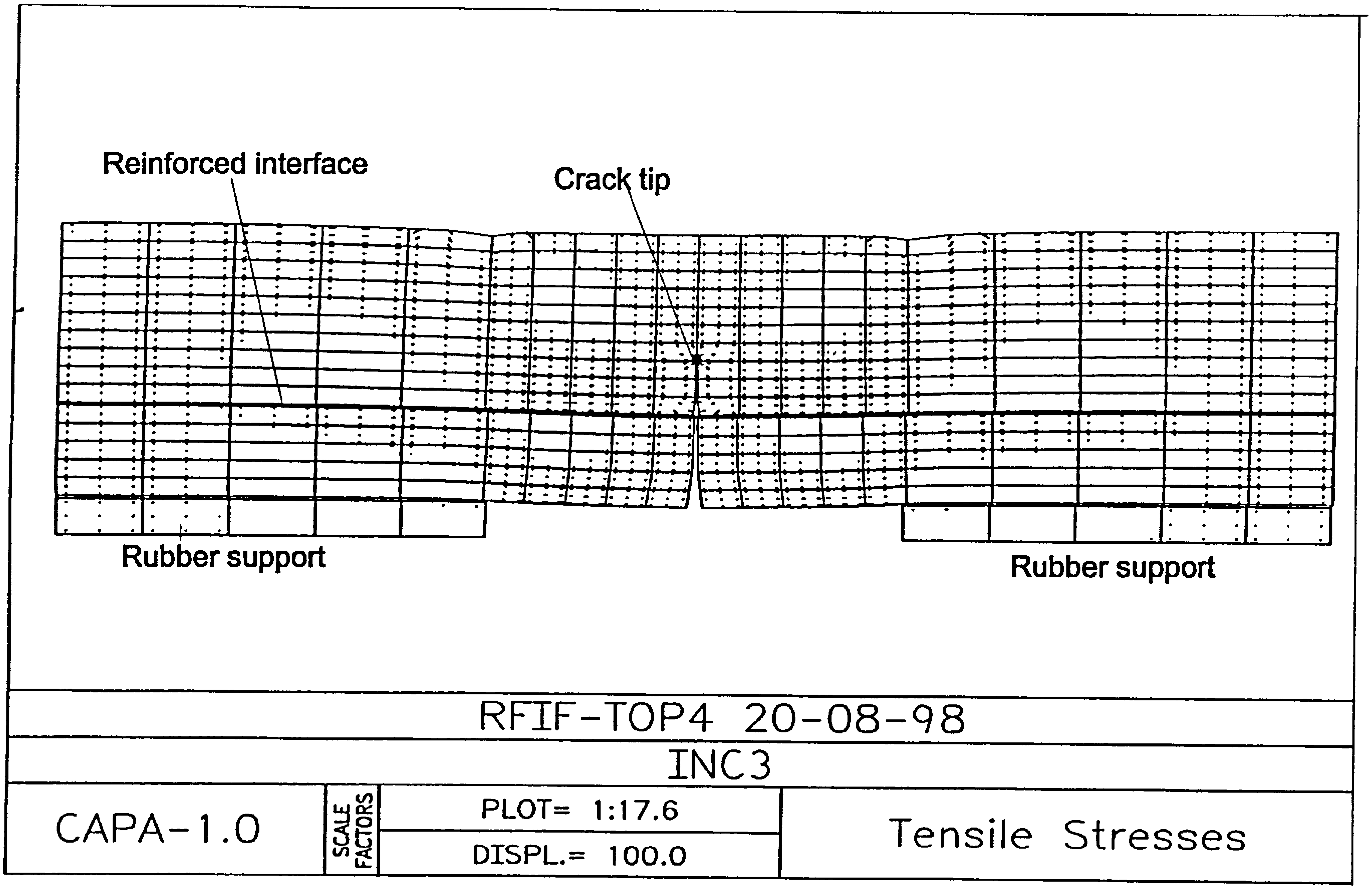


FIGURE 7.9
EXAMPLE OF THE FINITE ELEMENT BEAM MODEL

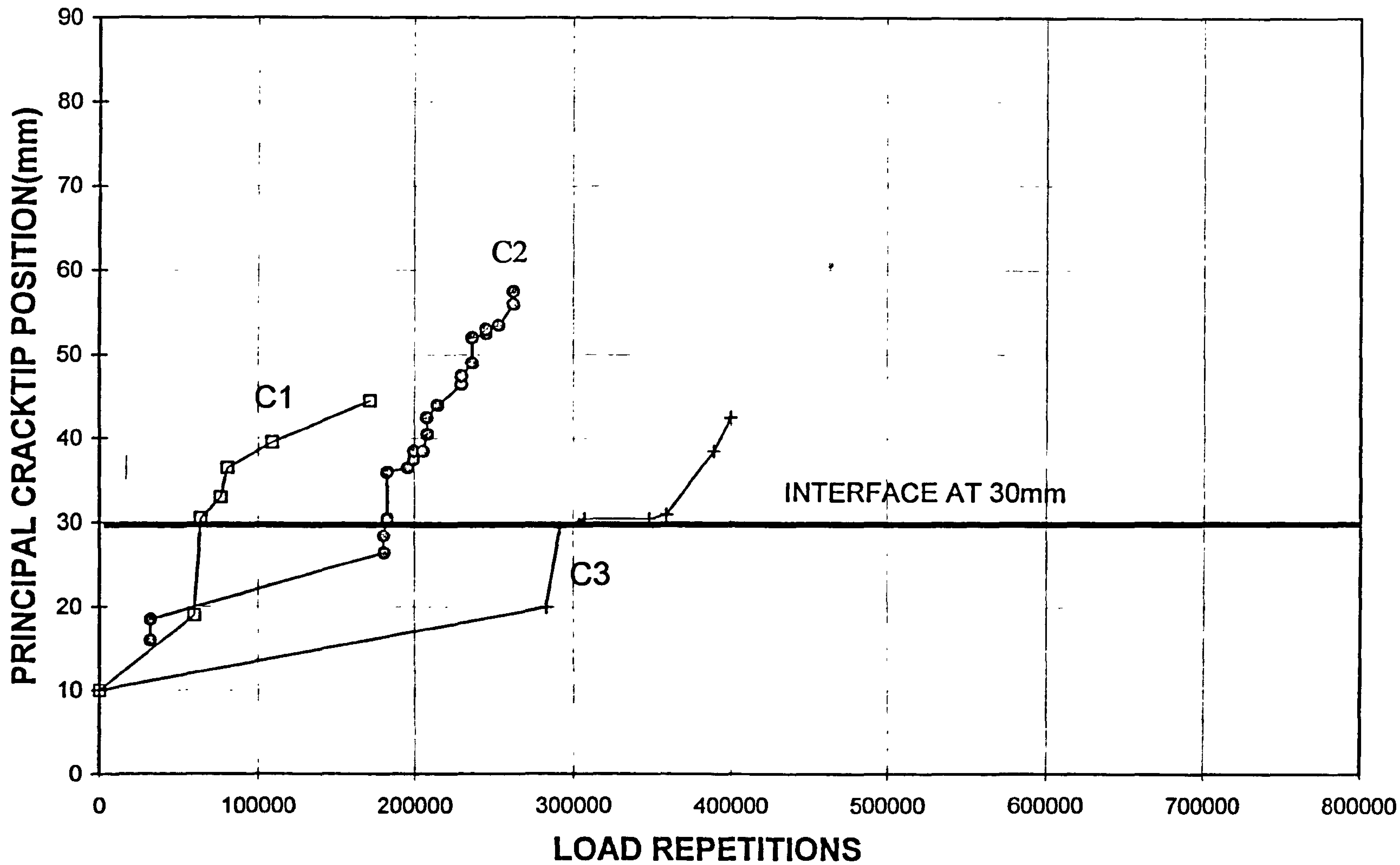


FIGURE 7.10
CRACK PROPAGATION: CONTROL BEAMS

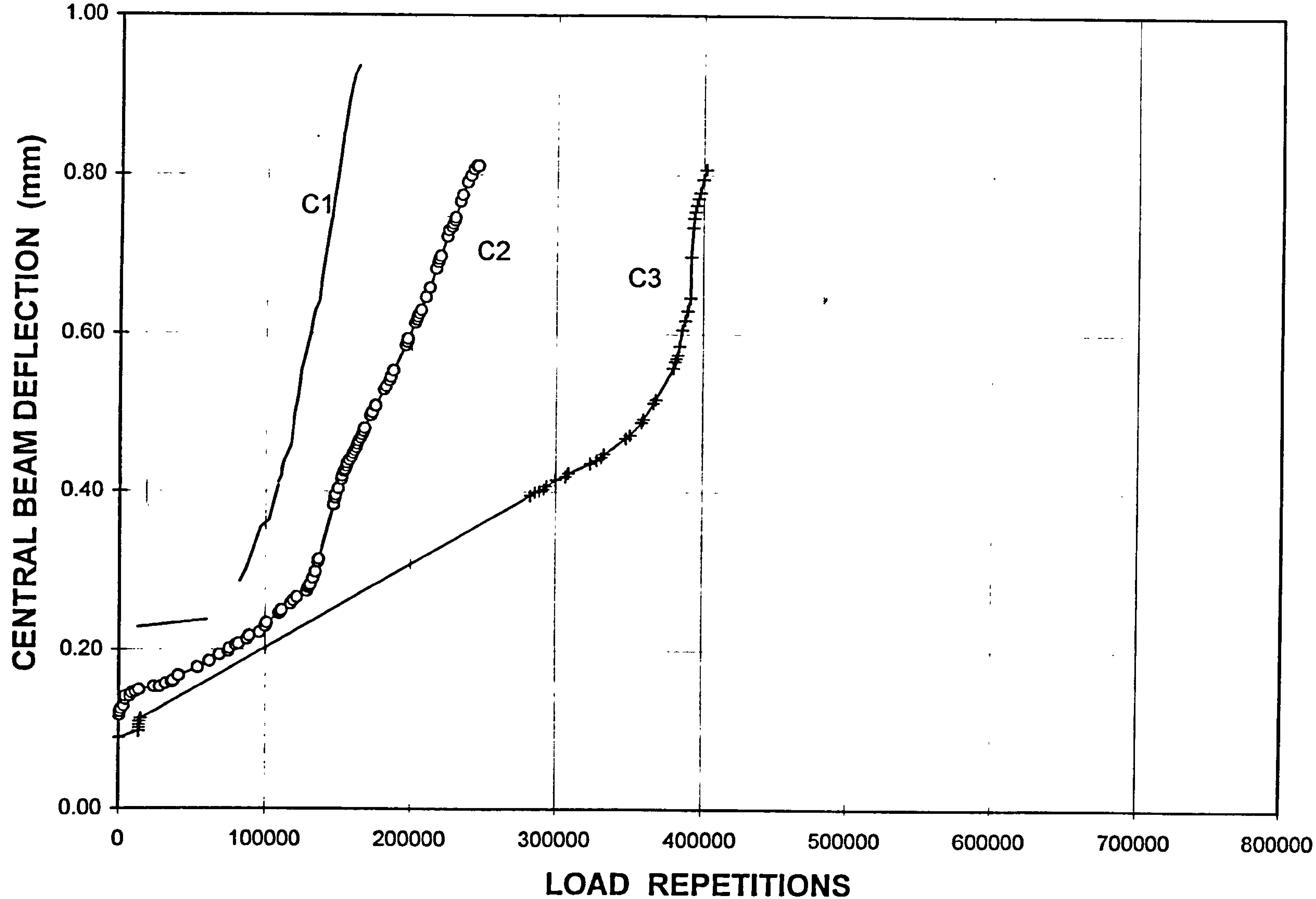


FIGURE 7.11
BEAM DEFLECTION: CONTROL BEAMS

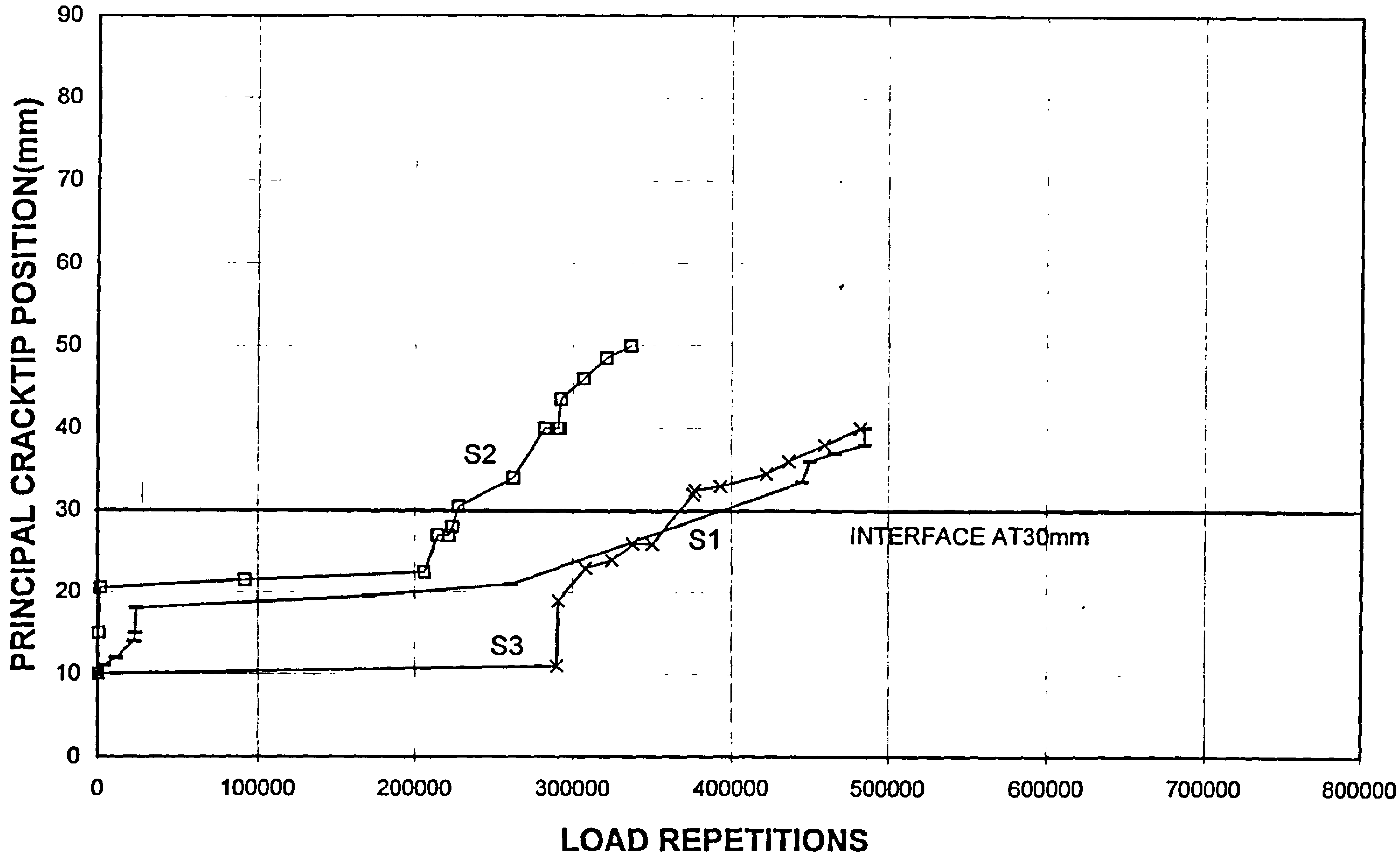


FIGURE 7.12
CRACK PROPAGATION: STEEL-REINFORCED BEAMS

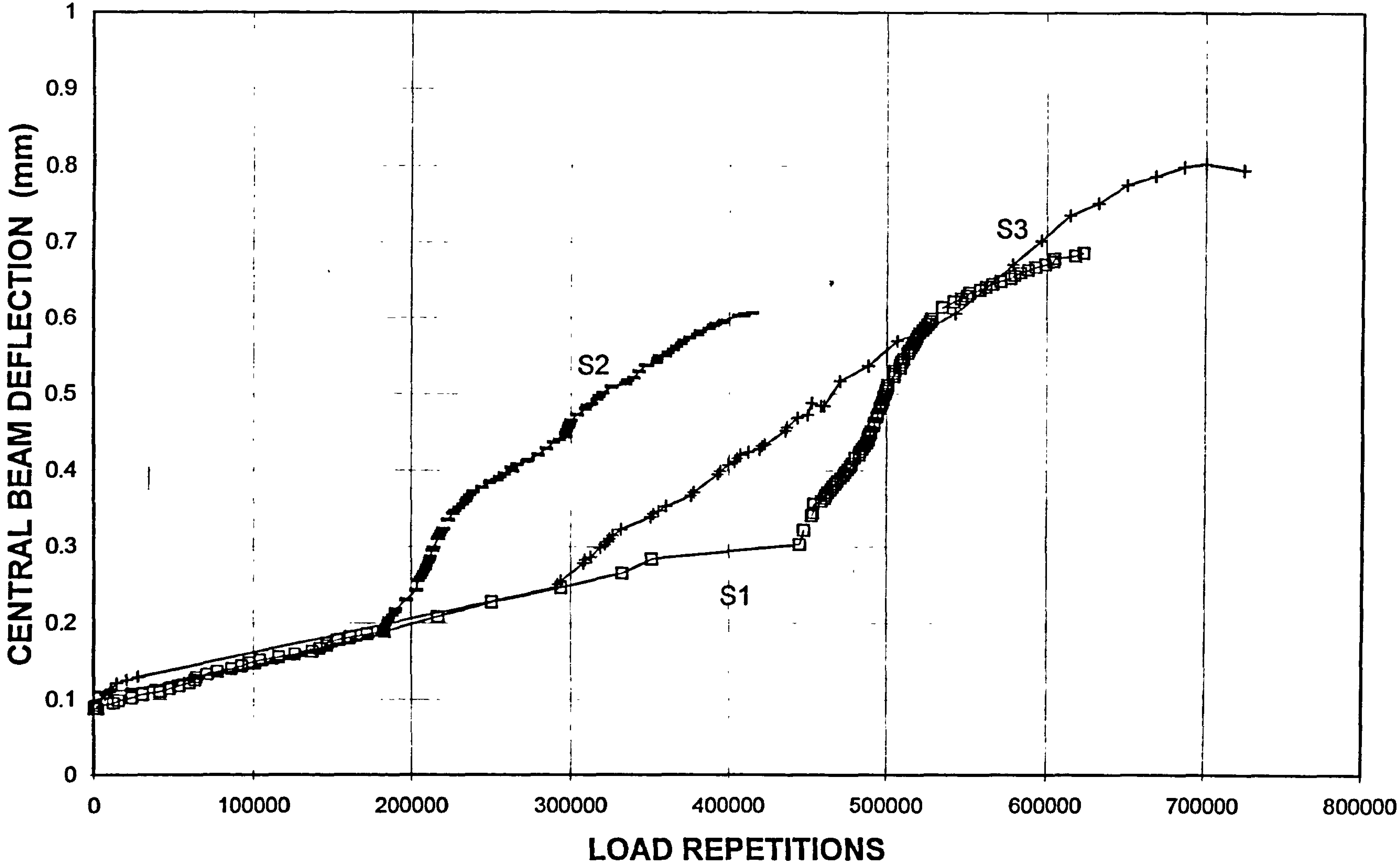


FIGURE 7.13
BEAM DEFLECTION: STEEL REINFORCED SAMPLES

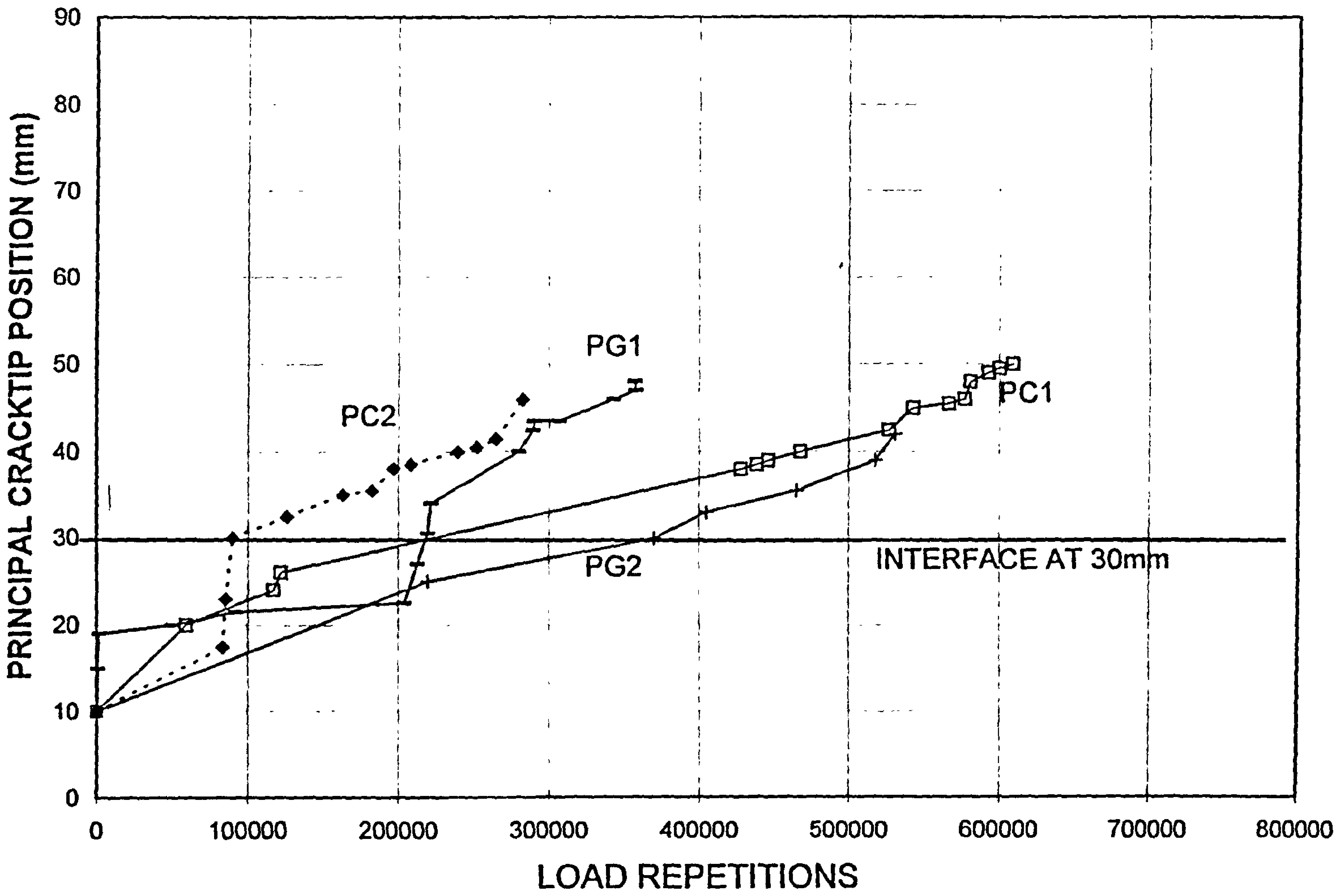


FIGURE 7.14
CRACK PROPAGATION: POLYPROPYLENE-REINFORCED BEAMS

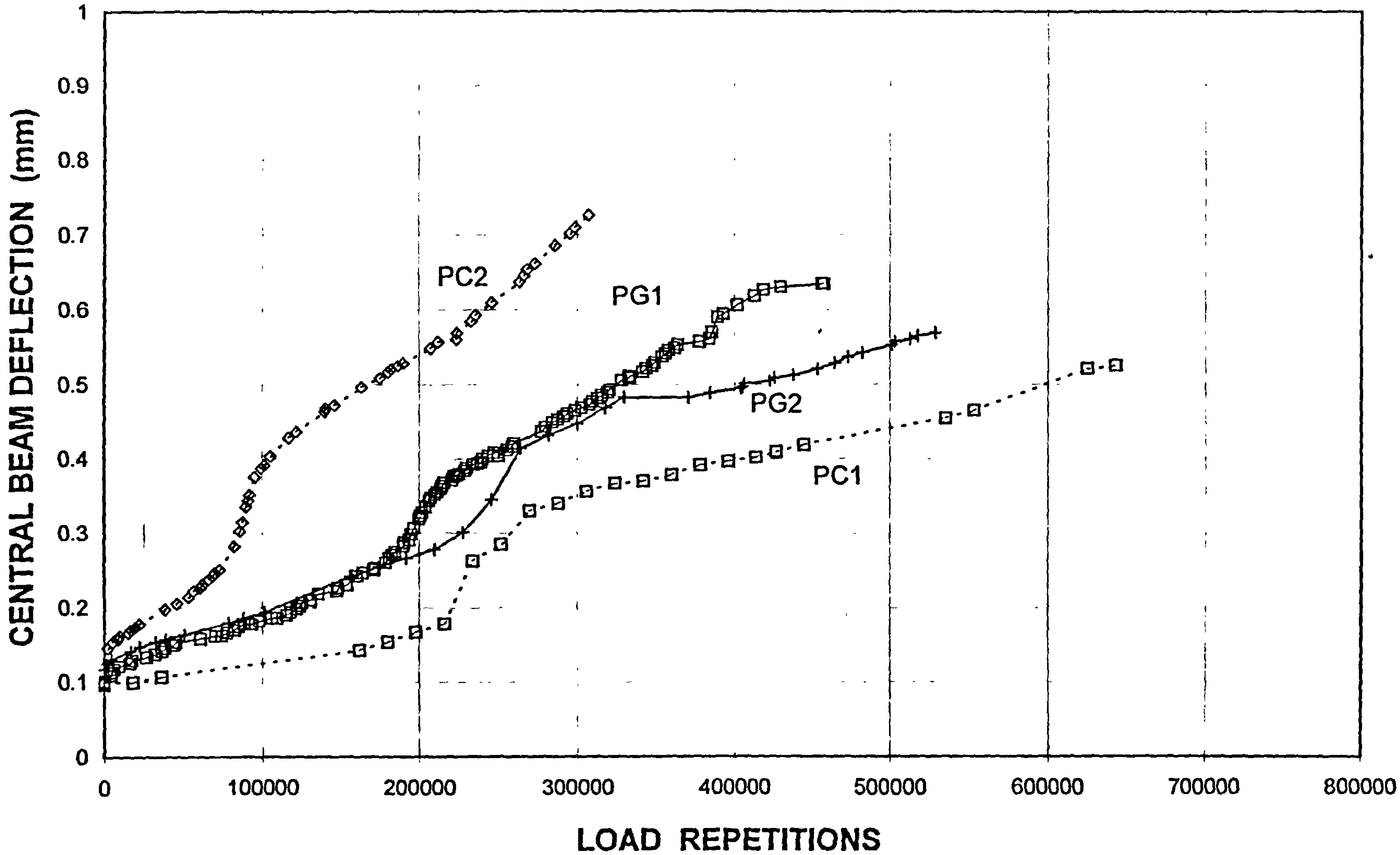


FIGURE 7.15
BEAM DEFLECTION: POLYPROPYLENE-REINFORCED BEAM

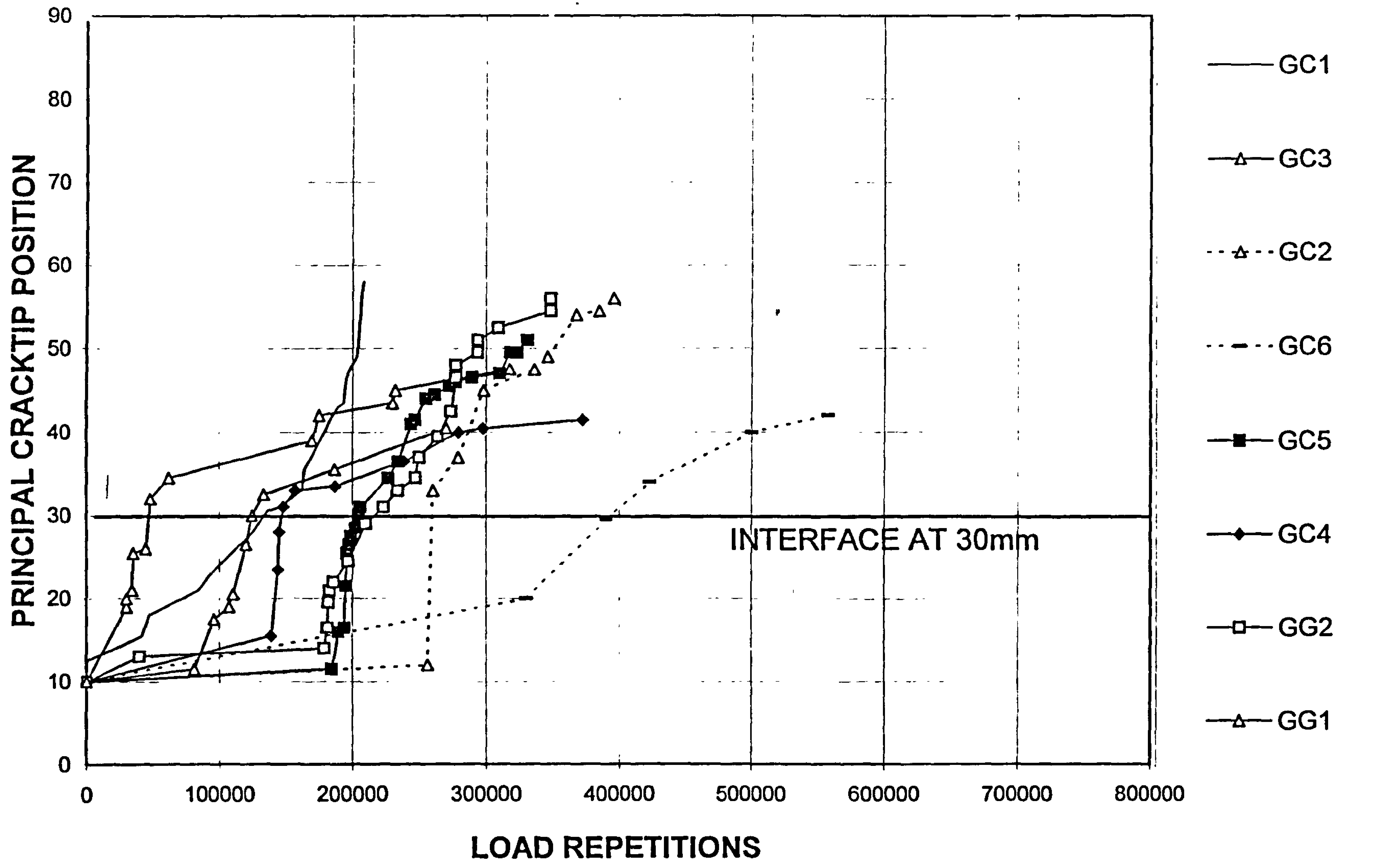


FIGURE 7.16
CRACK PROPAGATION: GLASS-REINFORCED BEAMS

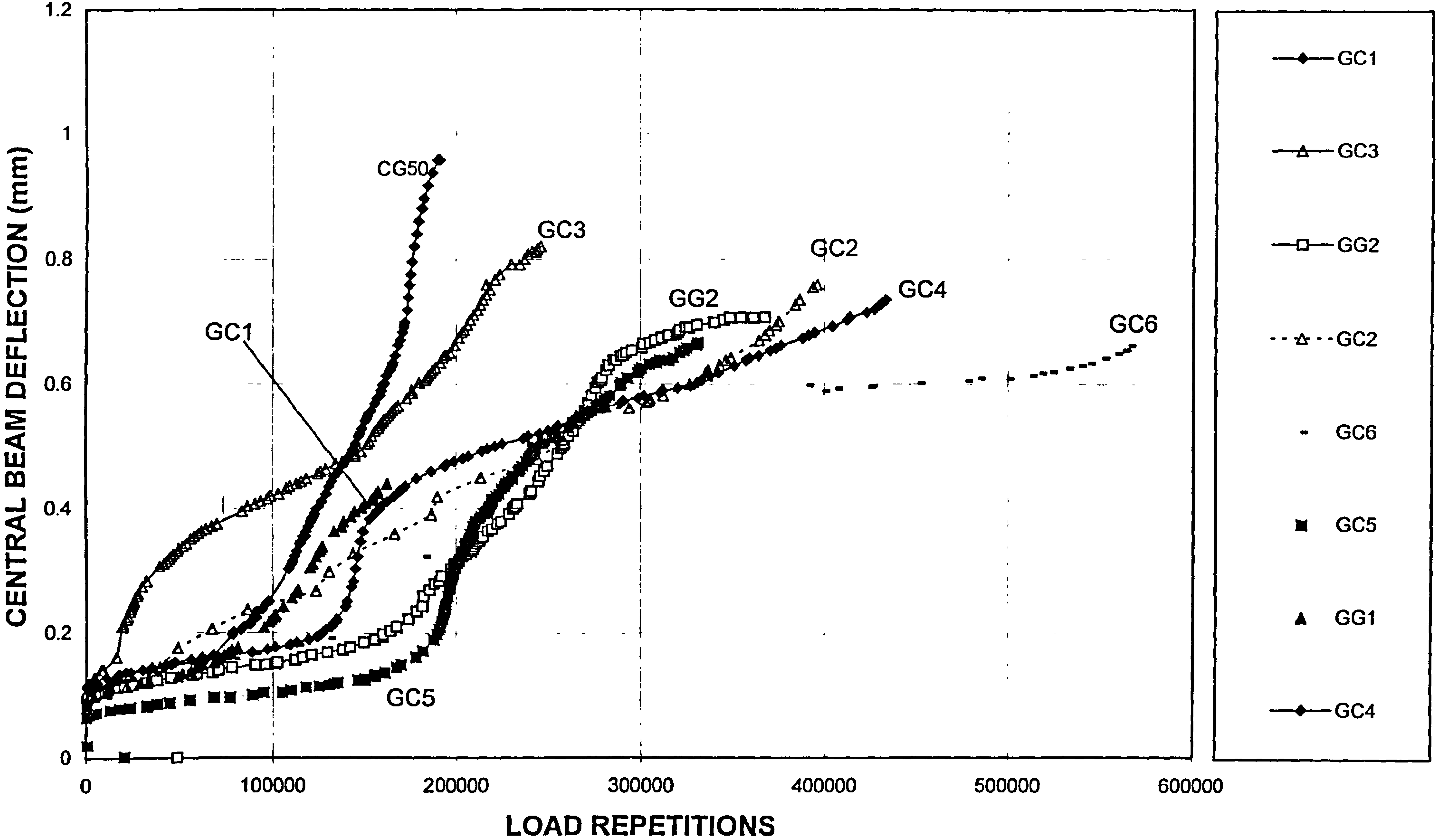


FIGURE 17.17
BEAM DEFLECTION: GLASS-REINFORCED BEAMS

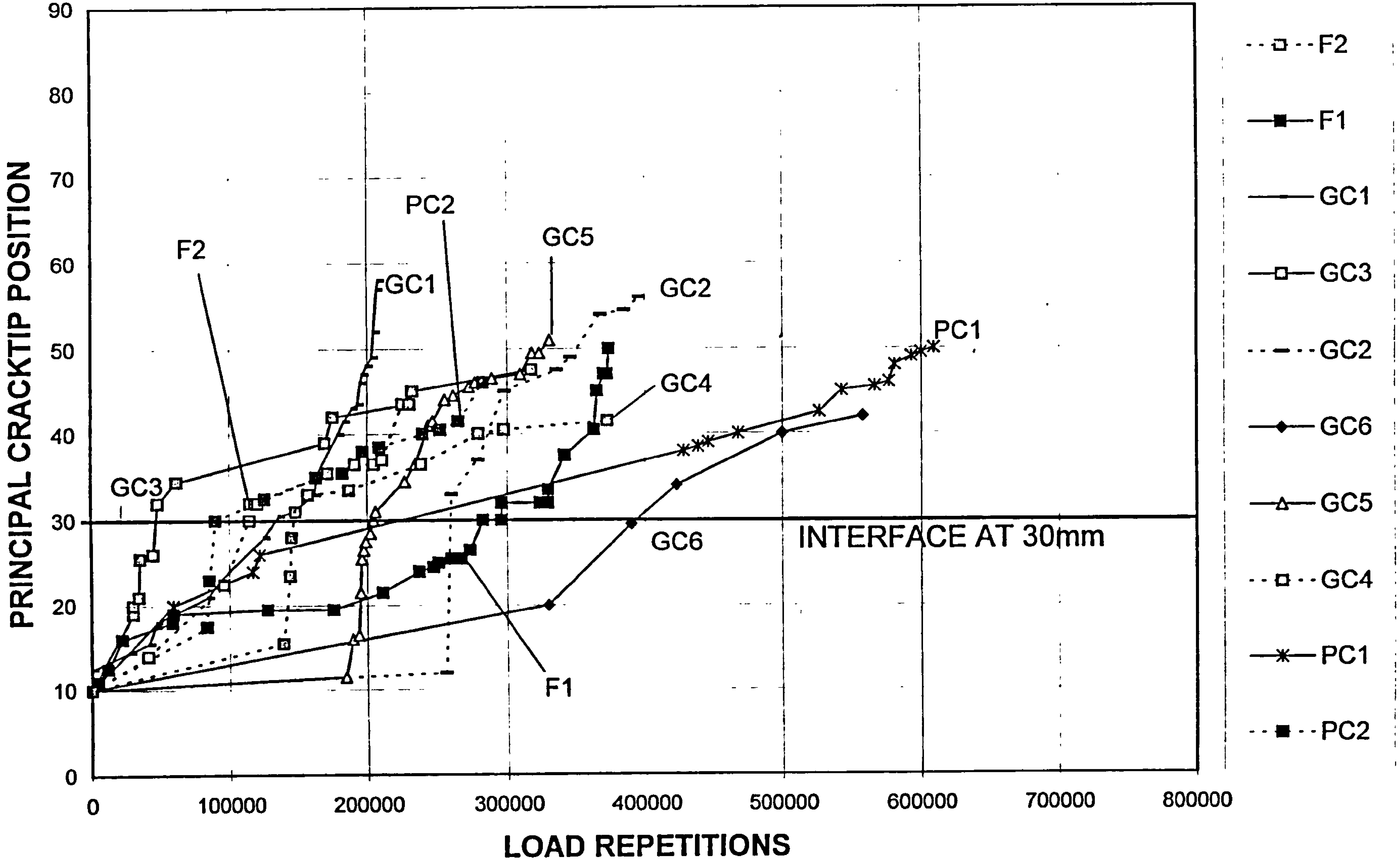


FIGURE 7.18
CRACK PROPAGATION: COMPOSITE AND FABRIC REINFORCED BEAMS

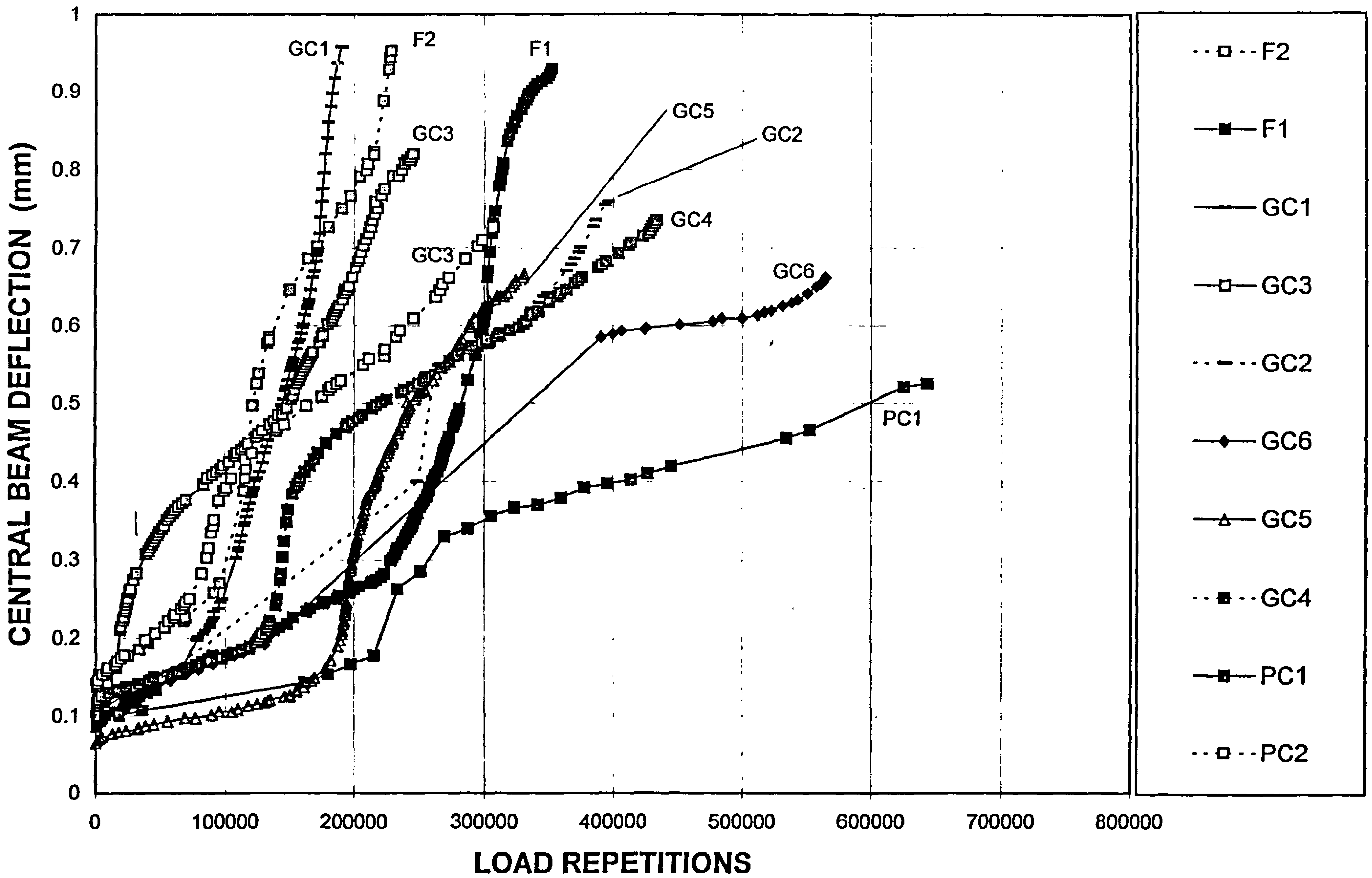


FIGURE 7.19

BEAM DEFLECTION: COMPOSITE AND FABRIC-REINFORCED SAMPLES

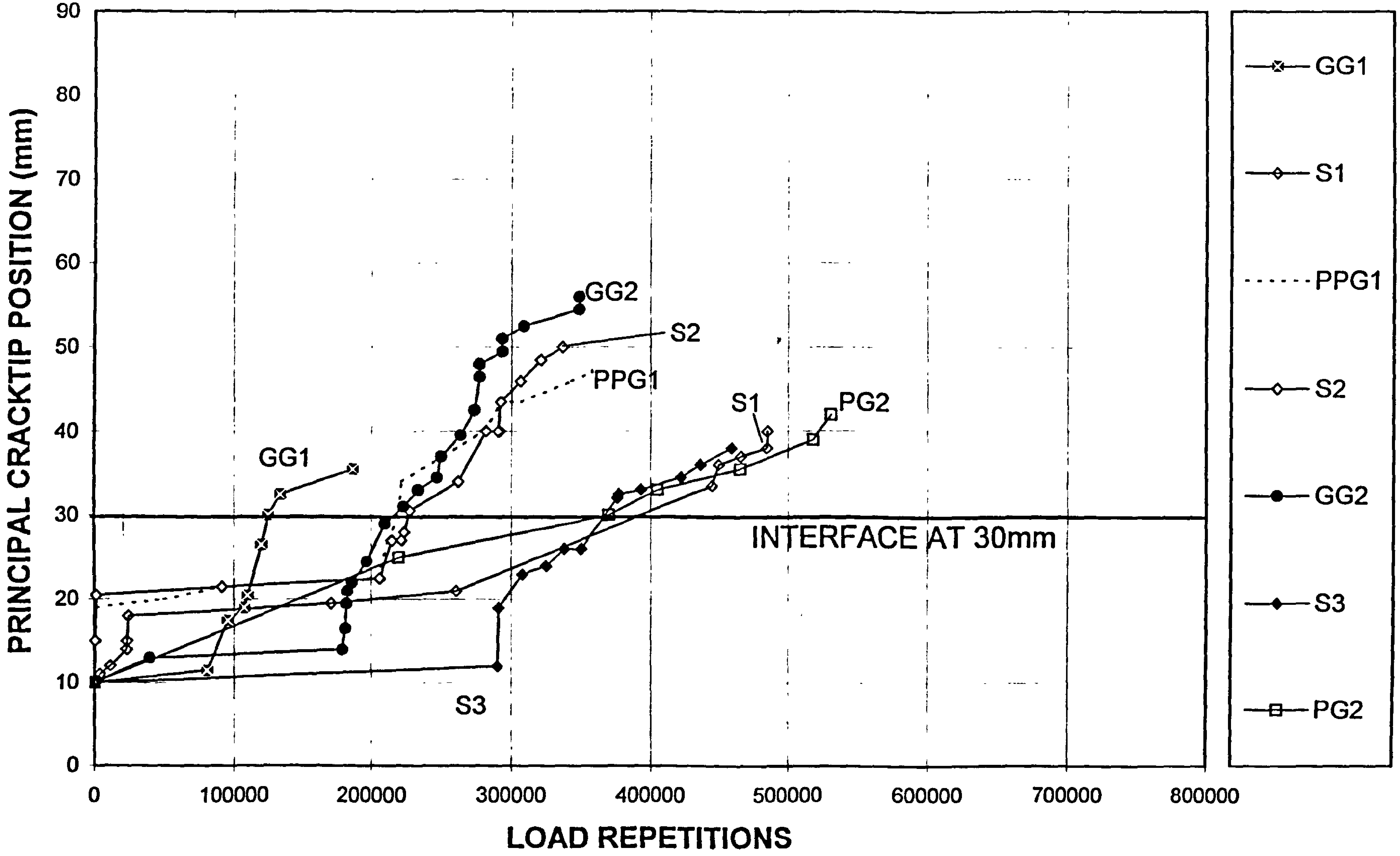


FIGURE 7.20
CRACK PROPAGATION: GRID-REINFORCED BEAMS

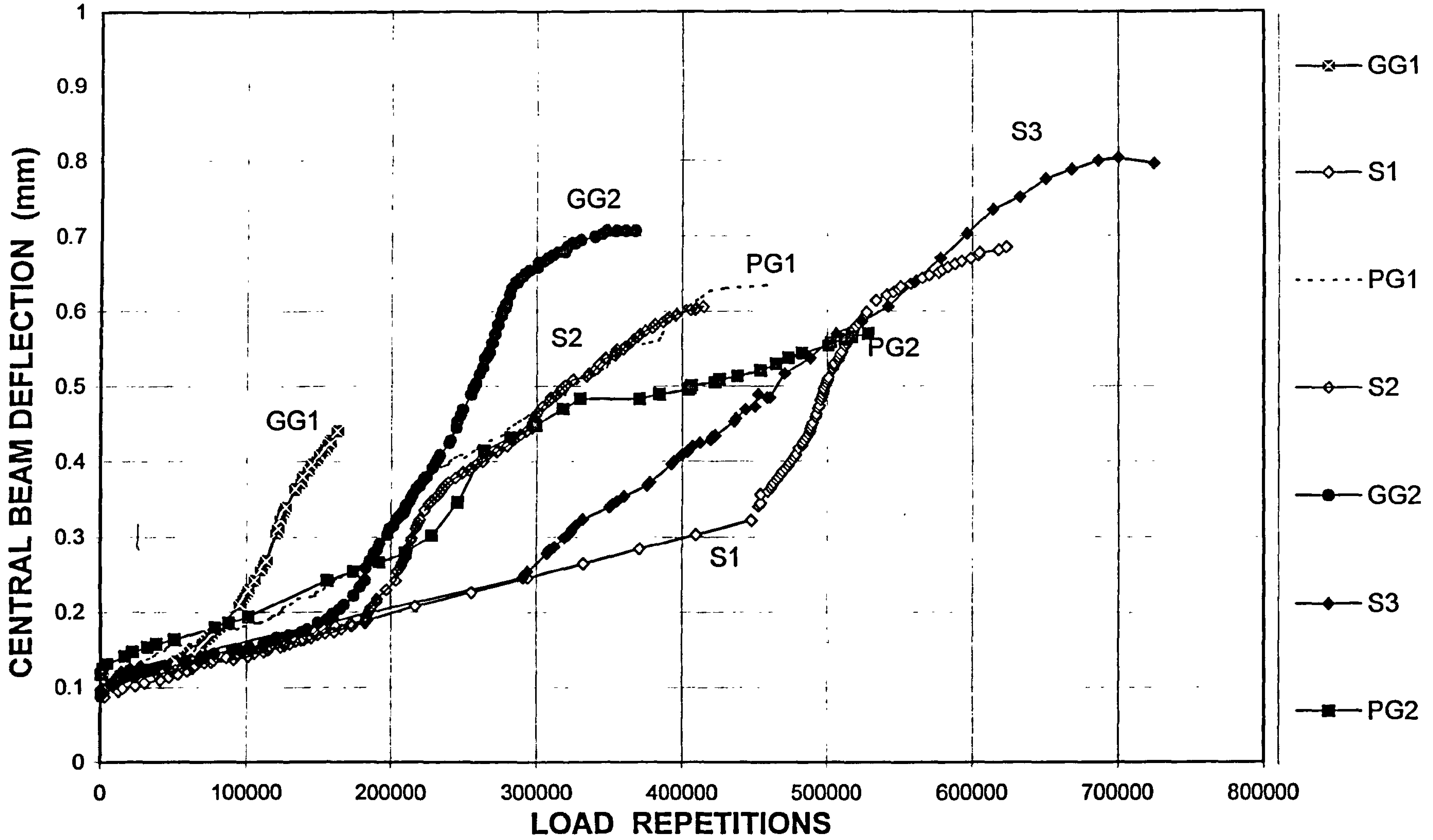


FIGURE 7.21
BEAM DEFLECTION: GRID-REINFORCED SAMPLES

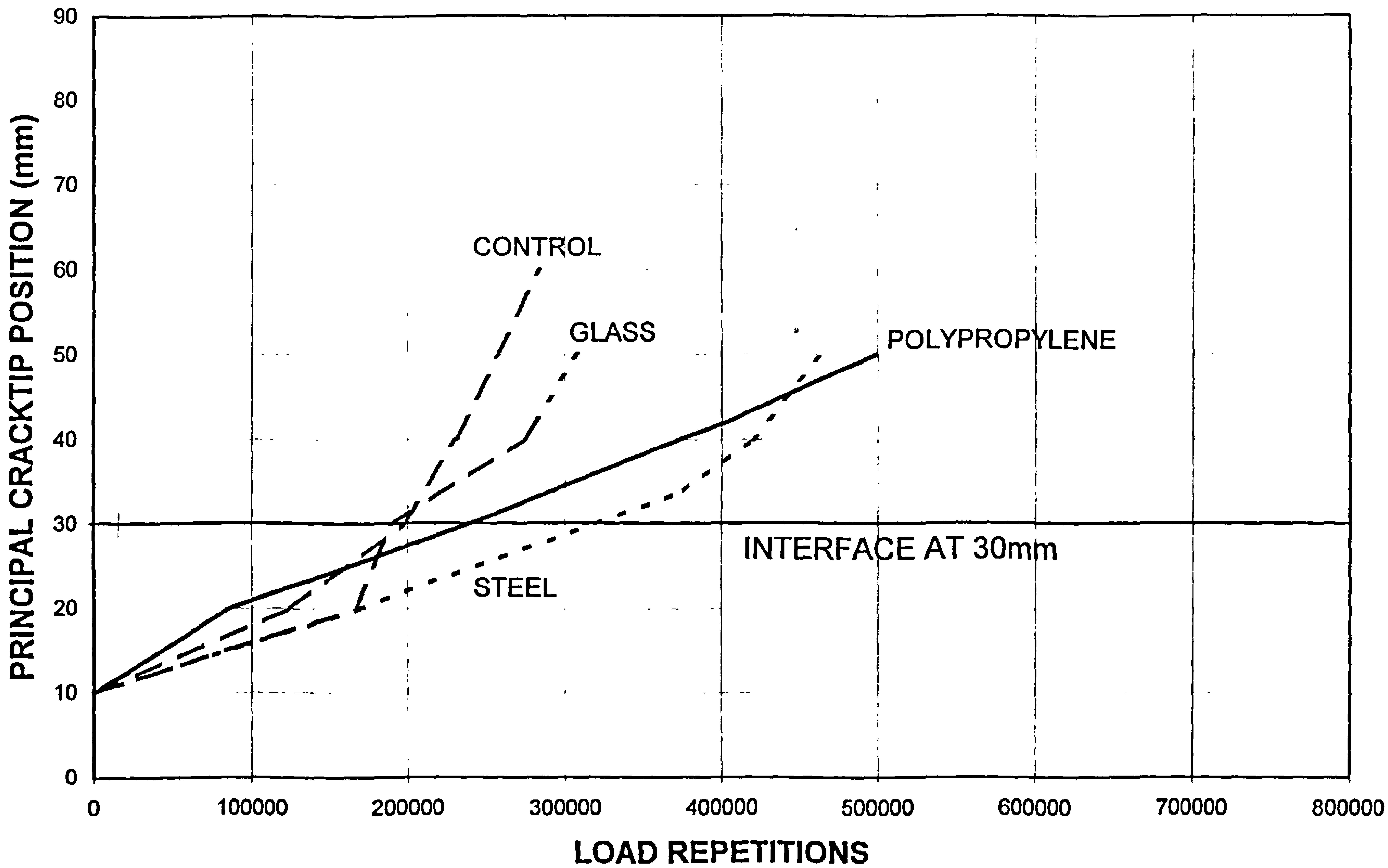


FIGURE 7.22
CRACK PROPAGATION: AVERAGE VALUES

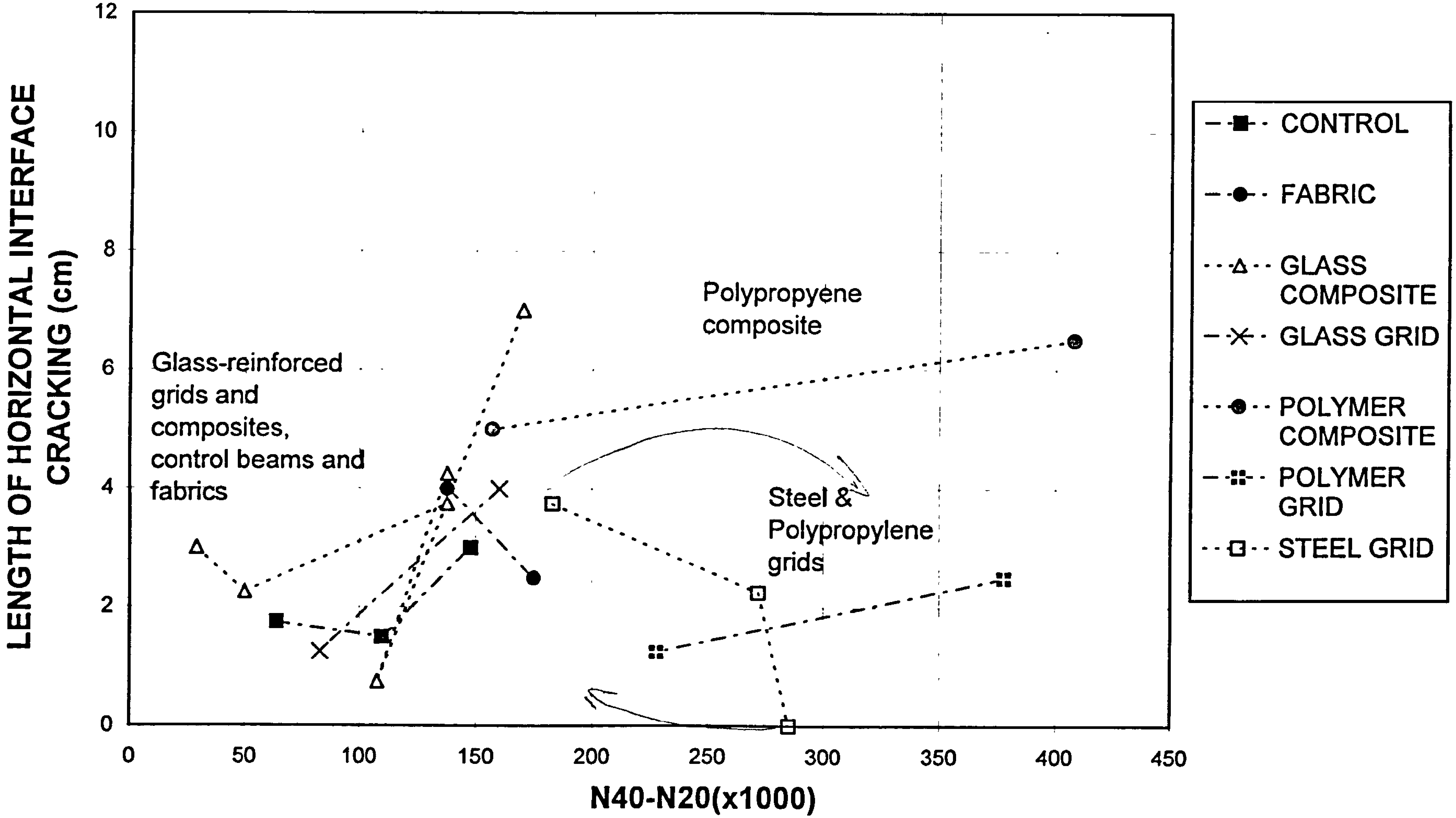


FIGURE 7.23
LENGTH OF INTERFACE CRACK vs (N40-N20)

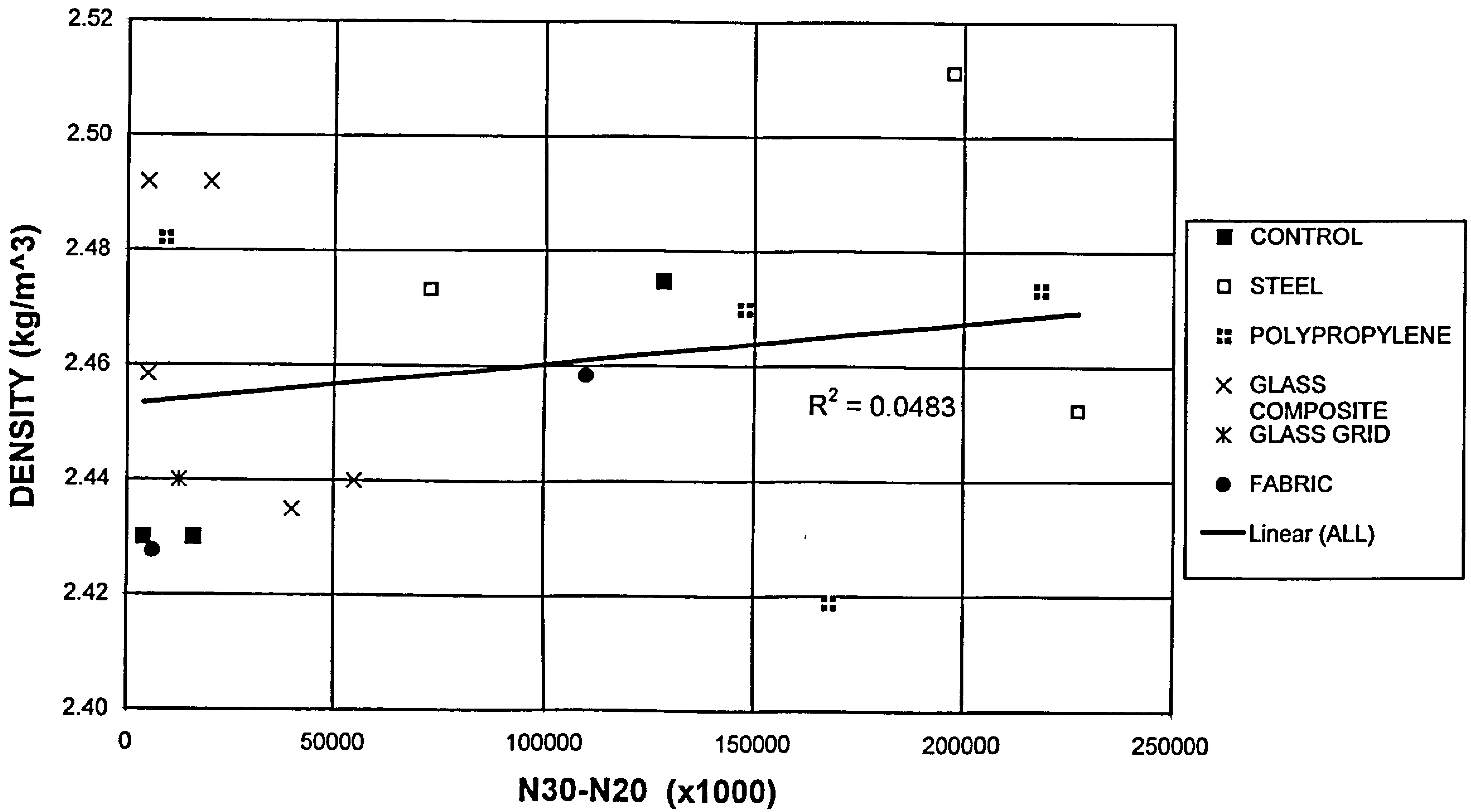


FIGURE 7.24
DENSITY BELOW INTERFACE vs (N30-N20)

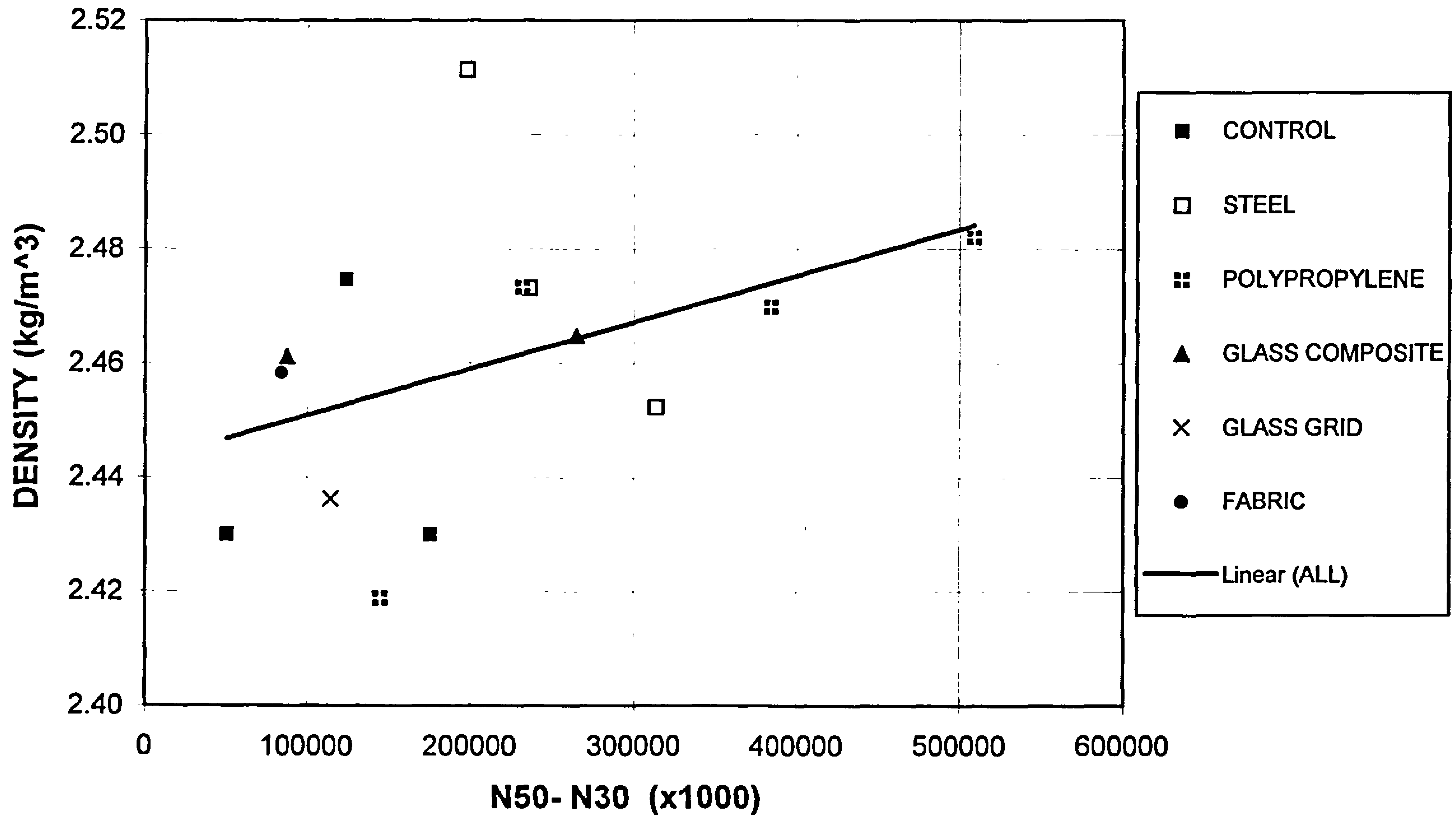


FIGURE 7.25
DENSITY ABOVE INTERFACE vs (N50-N30)

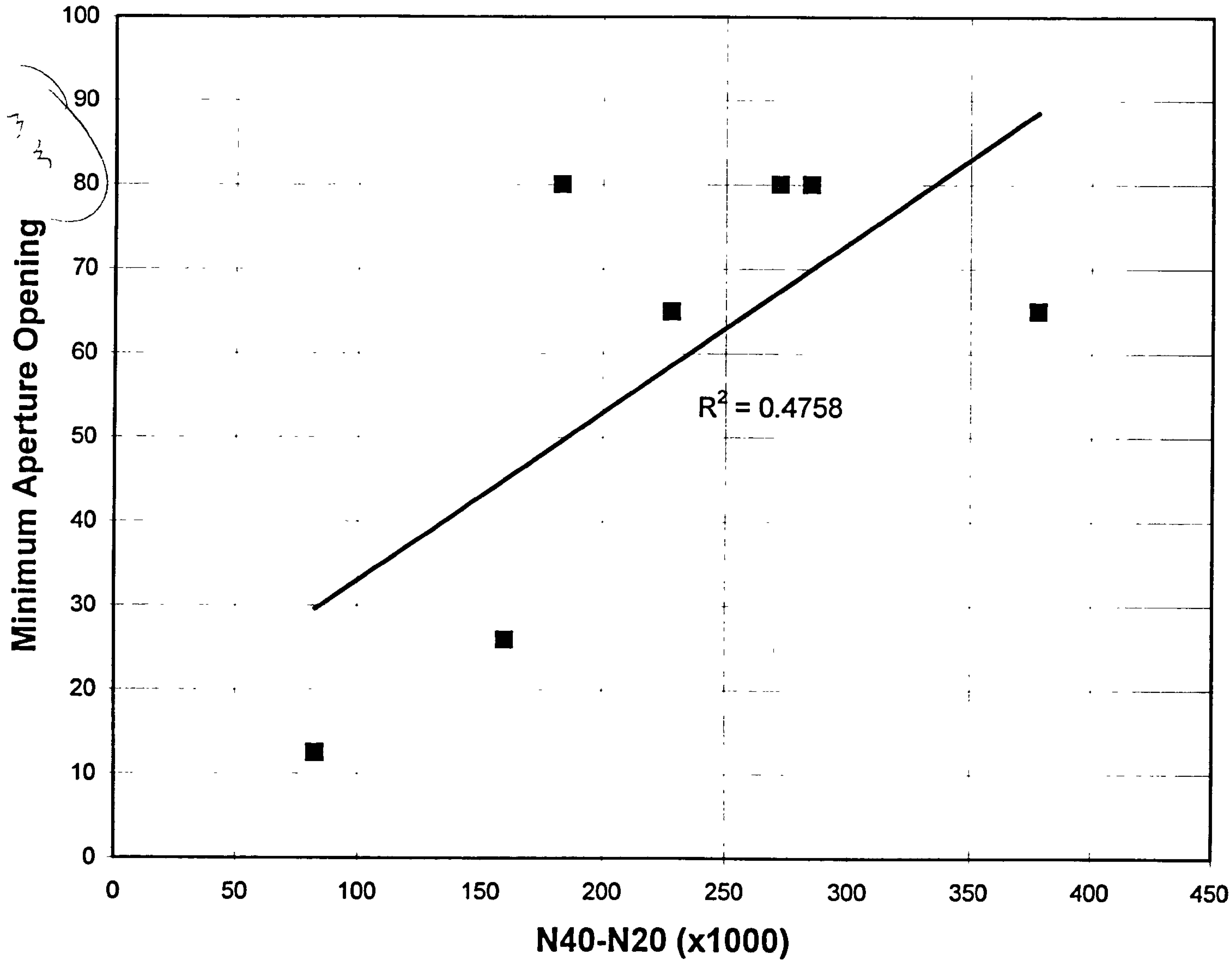


FIGURE 7.26
APERTURE SIZE vs (N40-N20)

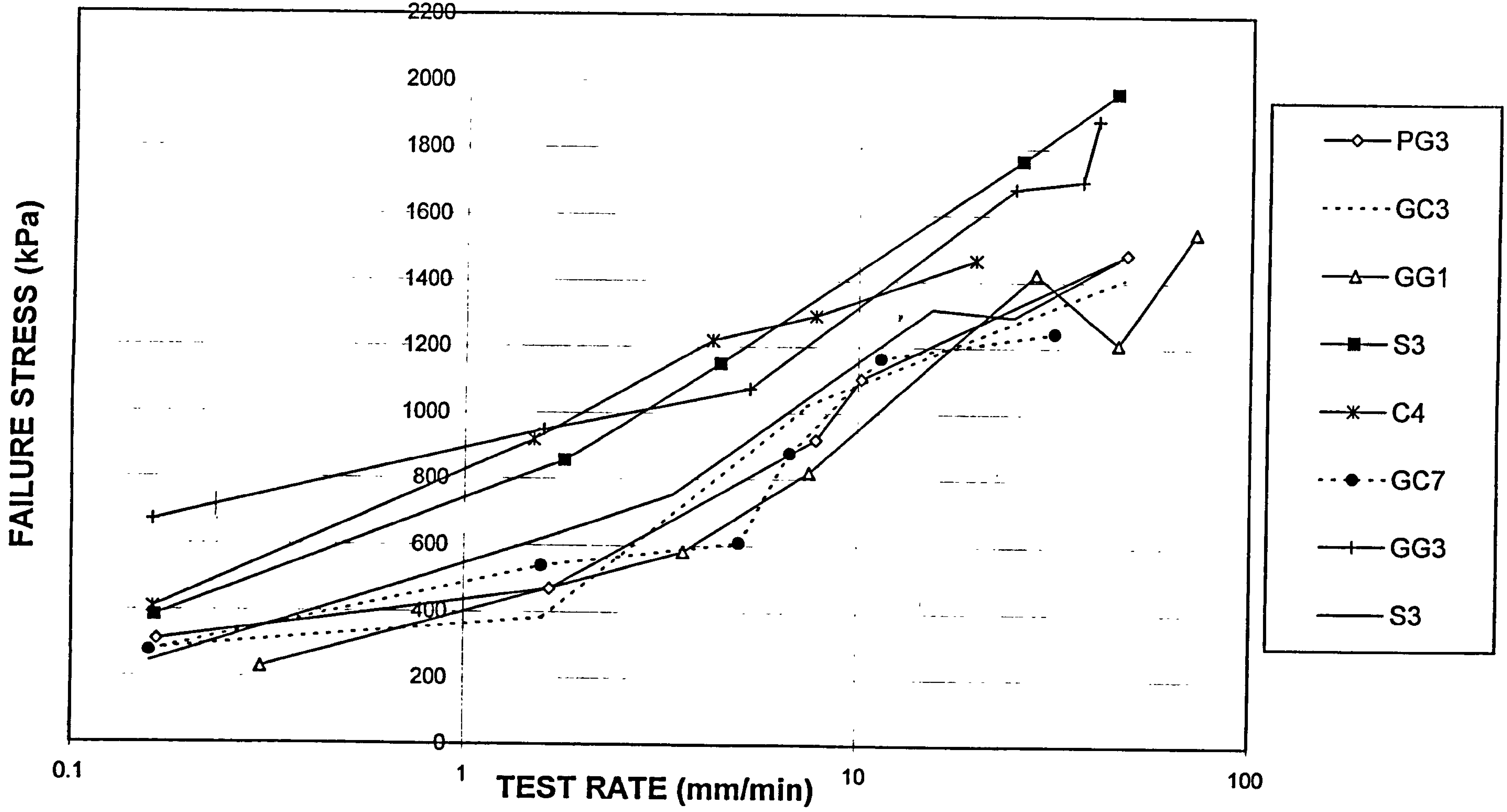


FIGURE 7.27
DIRECT TENSION TEST RESULTS

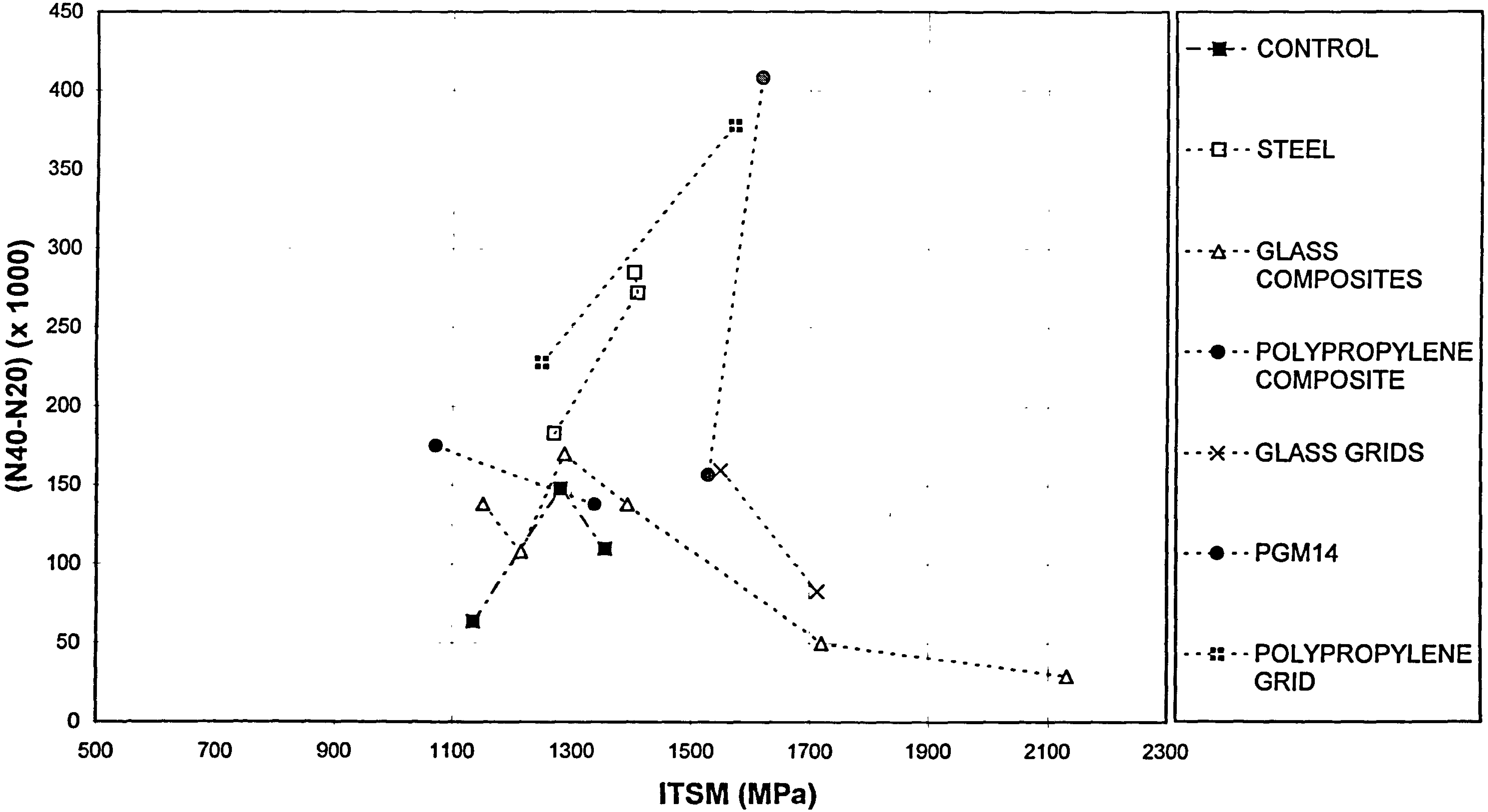


FIGURE 7.28
ASPHALT STIFFNESS vs (N40-N20)

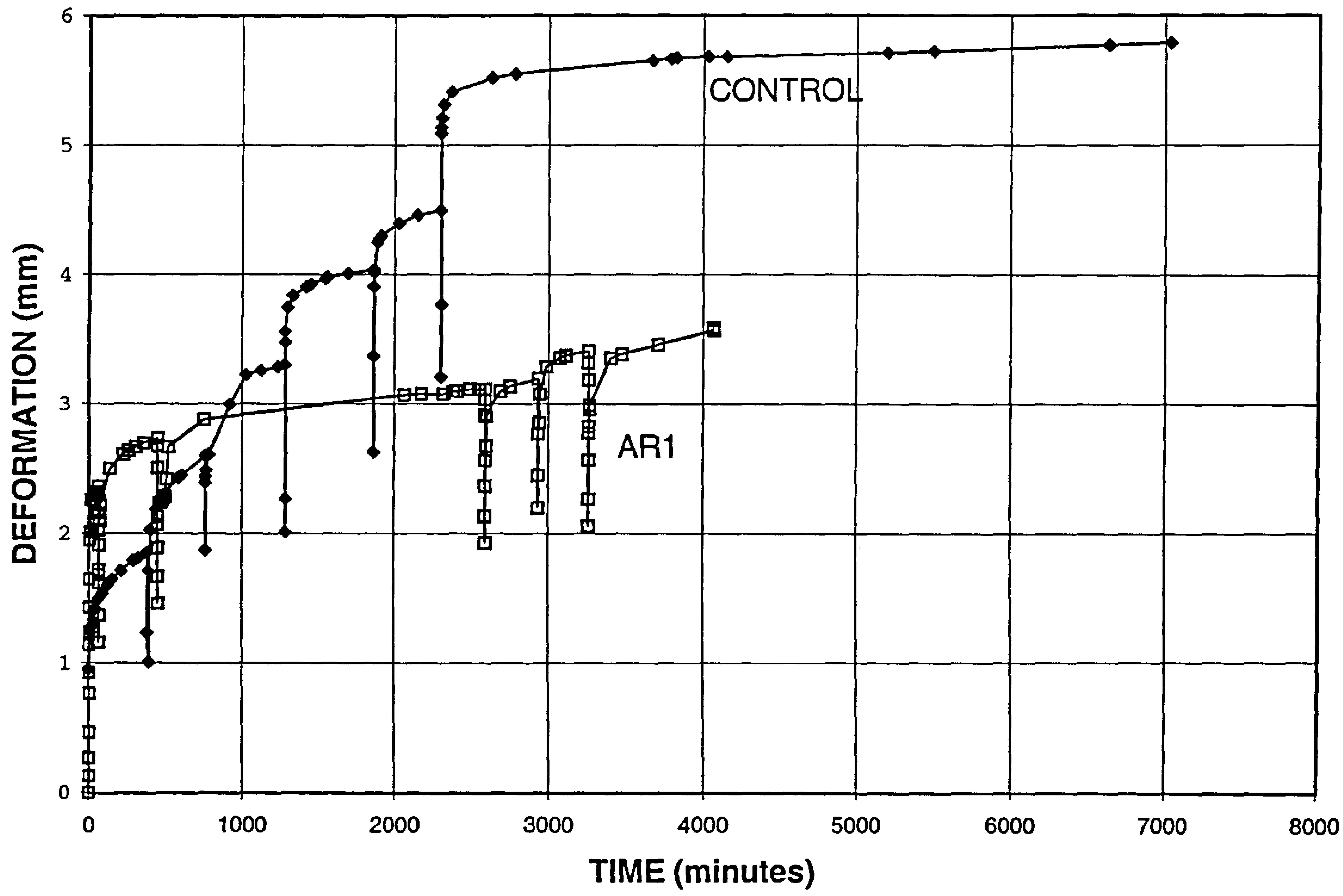


FIGURE 7.29
MEASUREMENT OF CREEP DEFORMATION

APPENDIX 7A
SAMPLE PREPARATION

Details of the material constituents of laboratory samples are given together with a description of sample preparation.

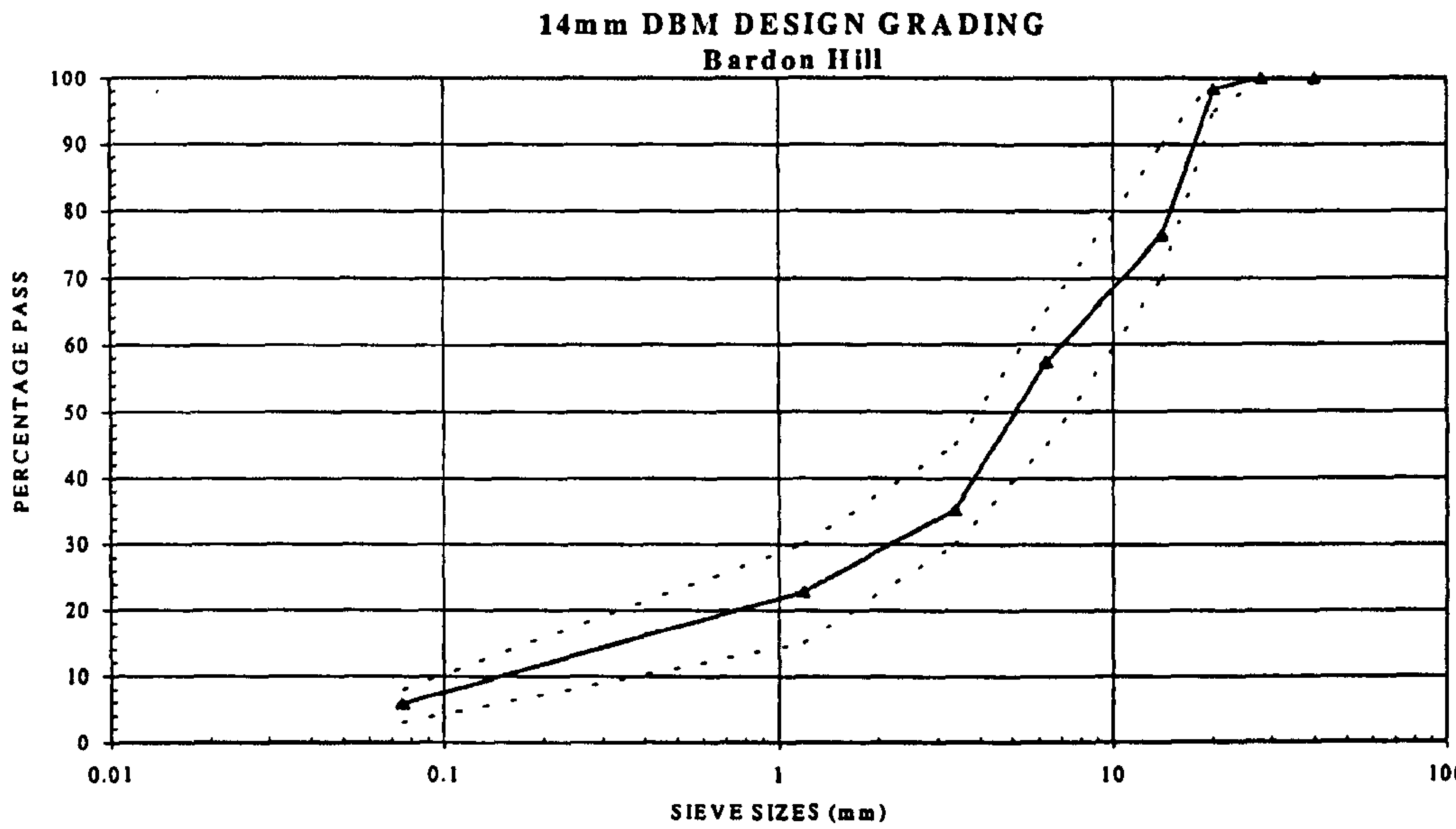
The 14mm wearing course mixture was constructed in accordance with BS 4987:Part 1: 1988, with the following properties:

BS 4987:Part 1: 1988, Section two:

Table 7A Aggregate grading for 14mm size open graded wearing course.

Test sieve aperture size (mm)	Aggregate: crushed rock or slag. % by mass passing
20	100
14	90-100
6.3	55-75
10	25-45
3.35	15-25
0.075	2-7

The grading of the actual aggregate used is plotted below.



Source of bitumen: Total Bitumen Products.

Bitumen Content: 4.8% bitumen in accordance with Table 18.

Bitumen Grade: 100 Pen, in accordance with Table 19.
SPT=45.6° C, Penetration = 94 x 0.1mm

Bitumen emulsion: K1-70
(for inter-layer bond between asphalt and reinforcement)

Particle charge:	Positive
Viscosity secs. Redwood II @ 85° C	27
Binder content (%m/m)	69.0

Sample Construction
Beams were constructed in the following manner:

(1) A 30mm layer was first compacted at 140°C
(2) When the asphalt temperature had reduced to 50°C, bitumen emulsion was applied at the following rates:

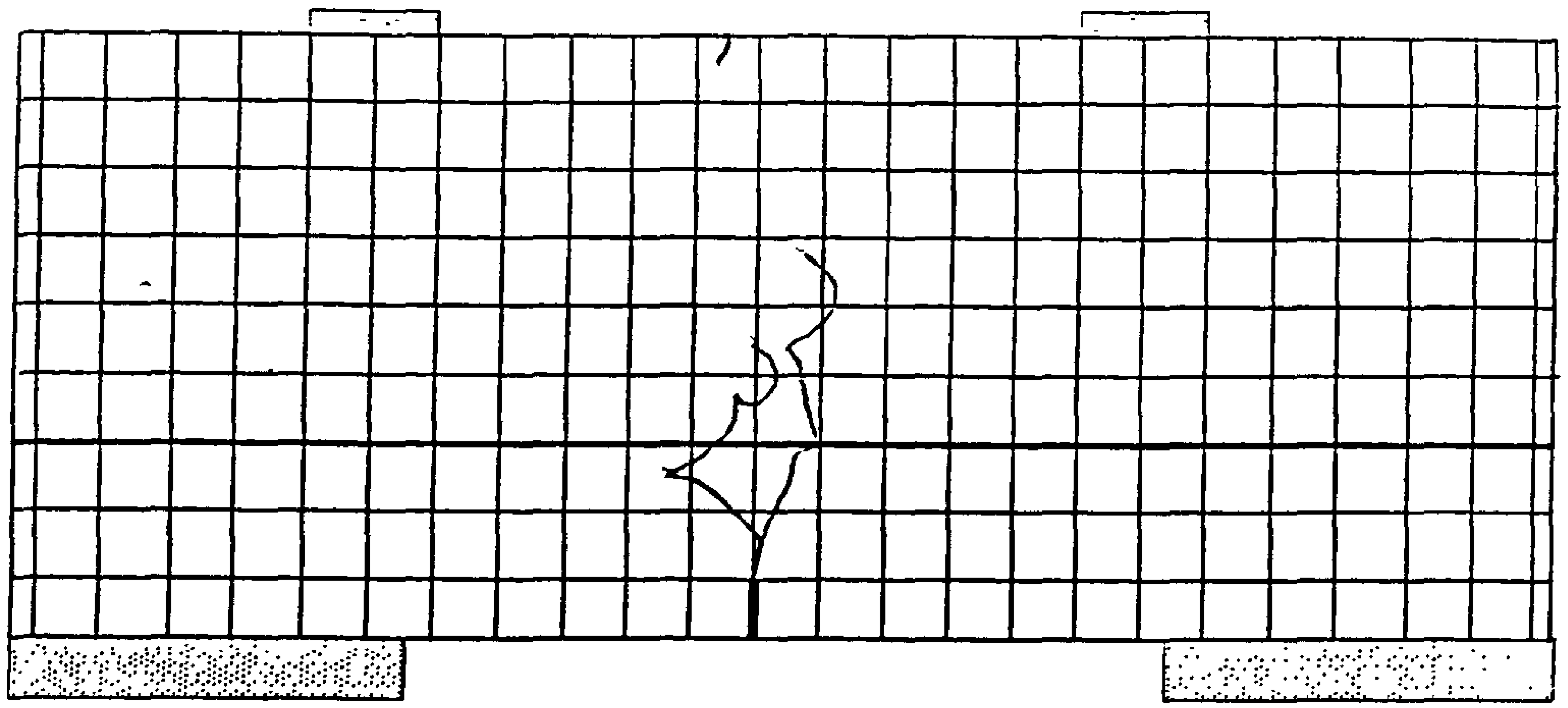
Reinforcement type	Emulsion Spread Rate (l/m^{2})
Unreinforced	0.3
Rotaflex 833	1.15
WG2303	0.3
AR1	0.35
AR-G	1.5
Road Mesh	0.3

Reinforcement was placed after the emulsion had broken.

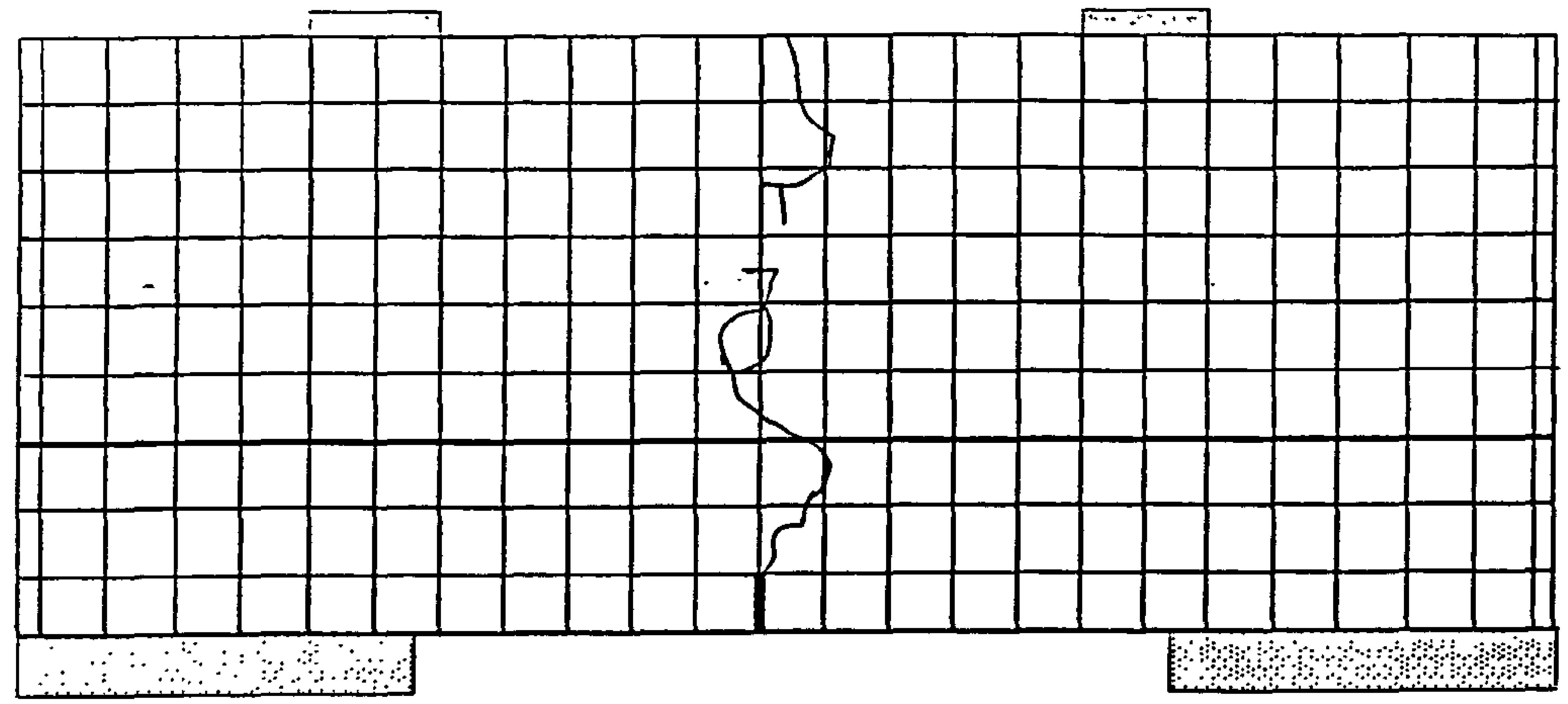
The second layer of asphalt was then placed at 140°C and compacted.

APPENDIX 7B
PLOTS OF CRACKS vs LOAD REPETITIONS: 'CRACK MAPS'

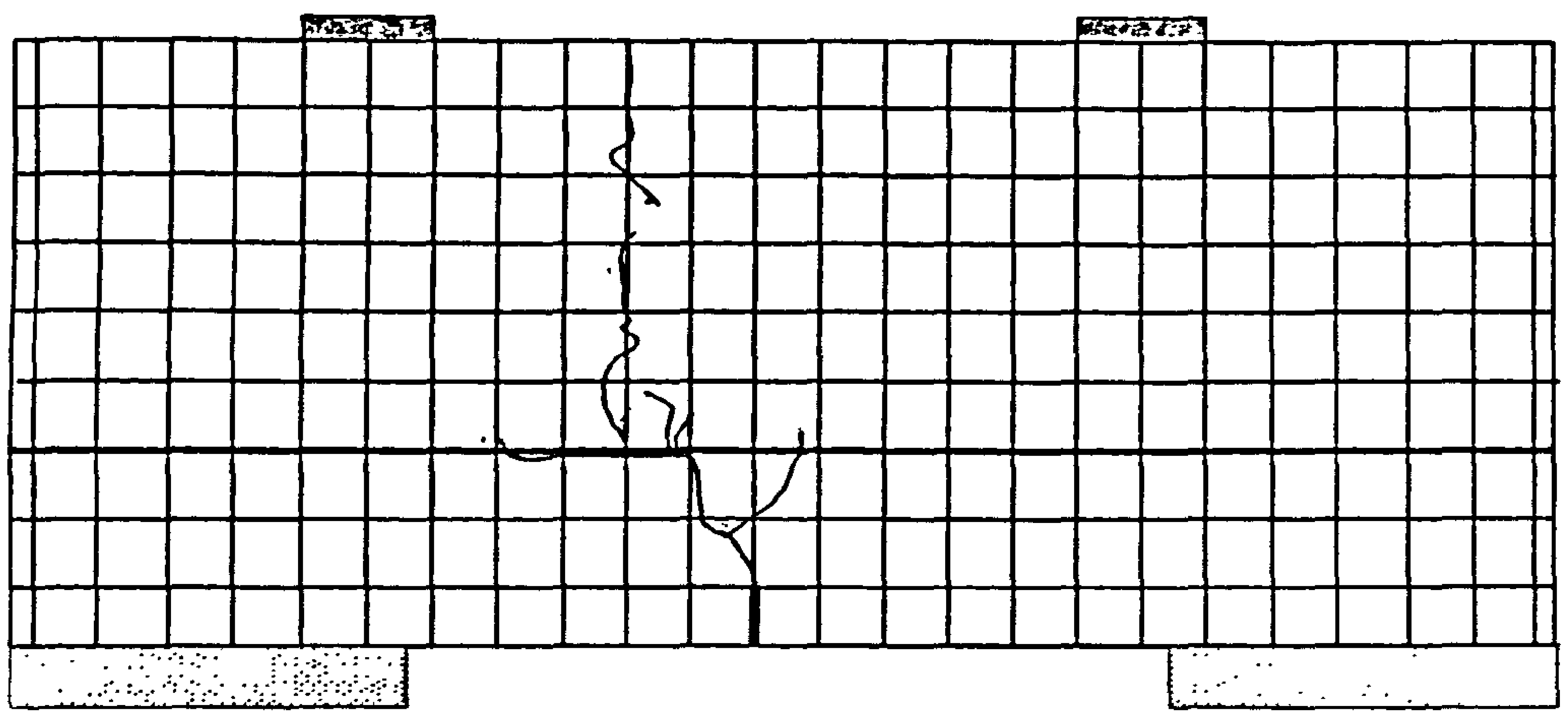
SPECIMEN:- C3
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



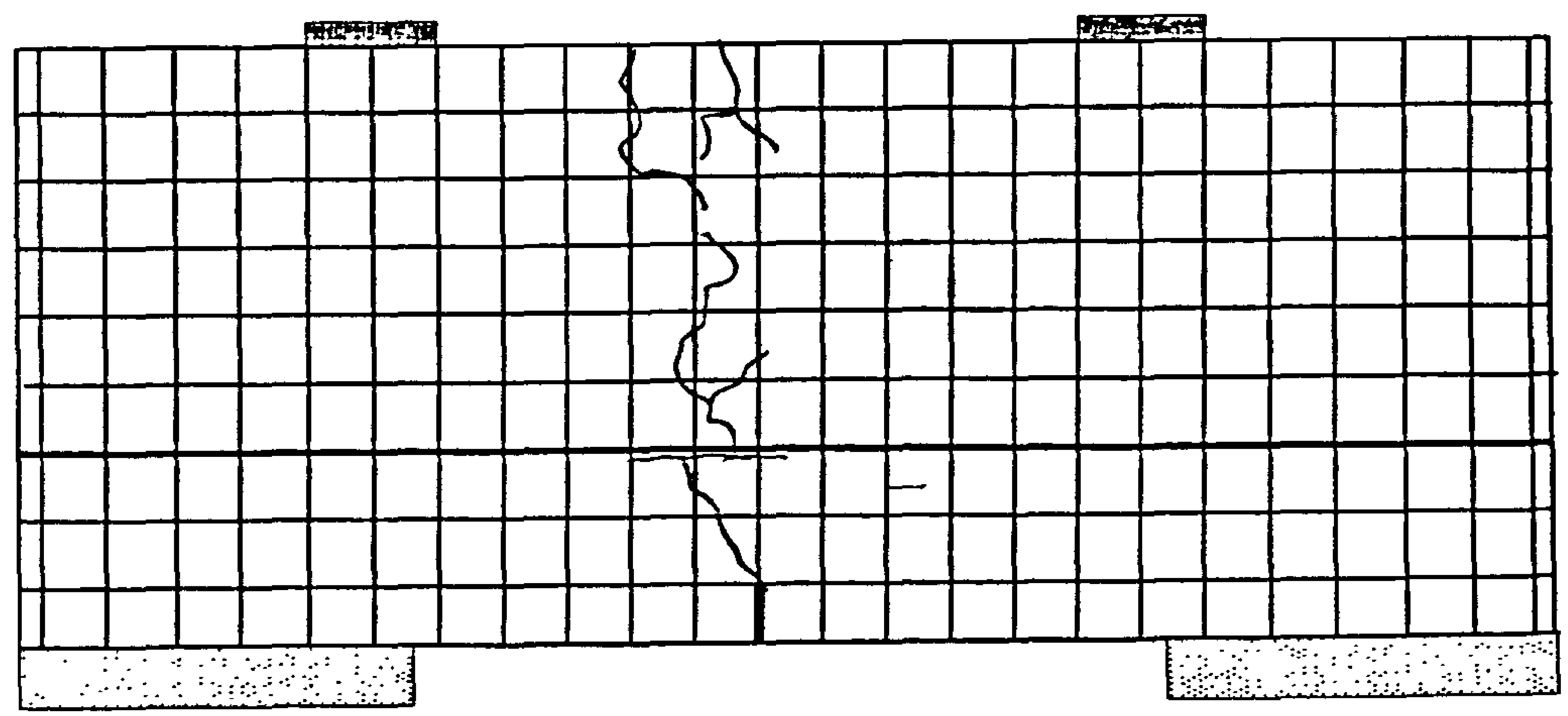
SPECIMEN:- C2
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



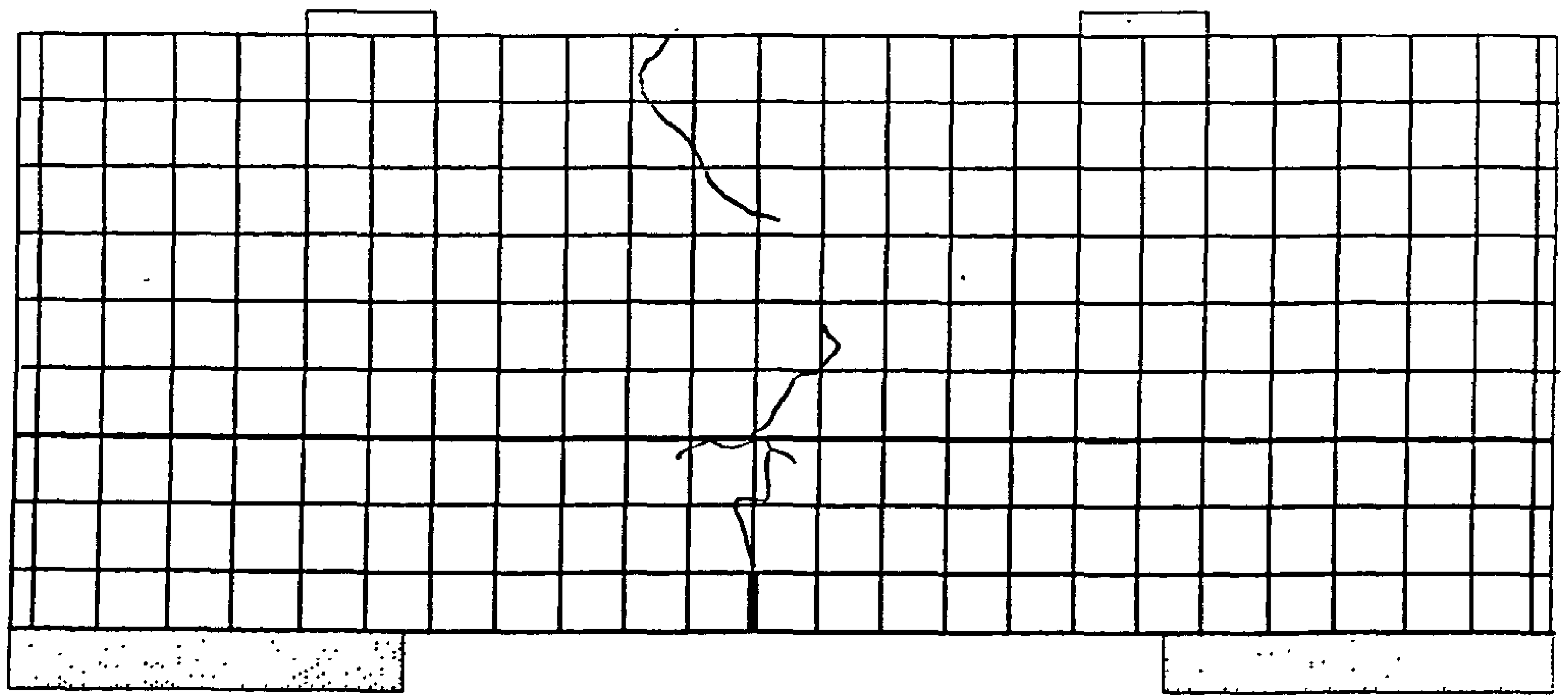
SPECIMEN:- C3
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



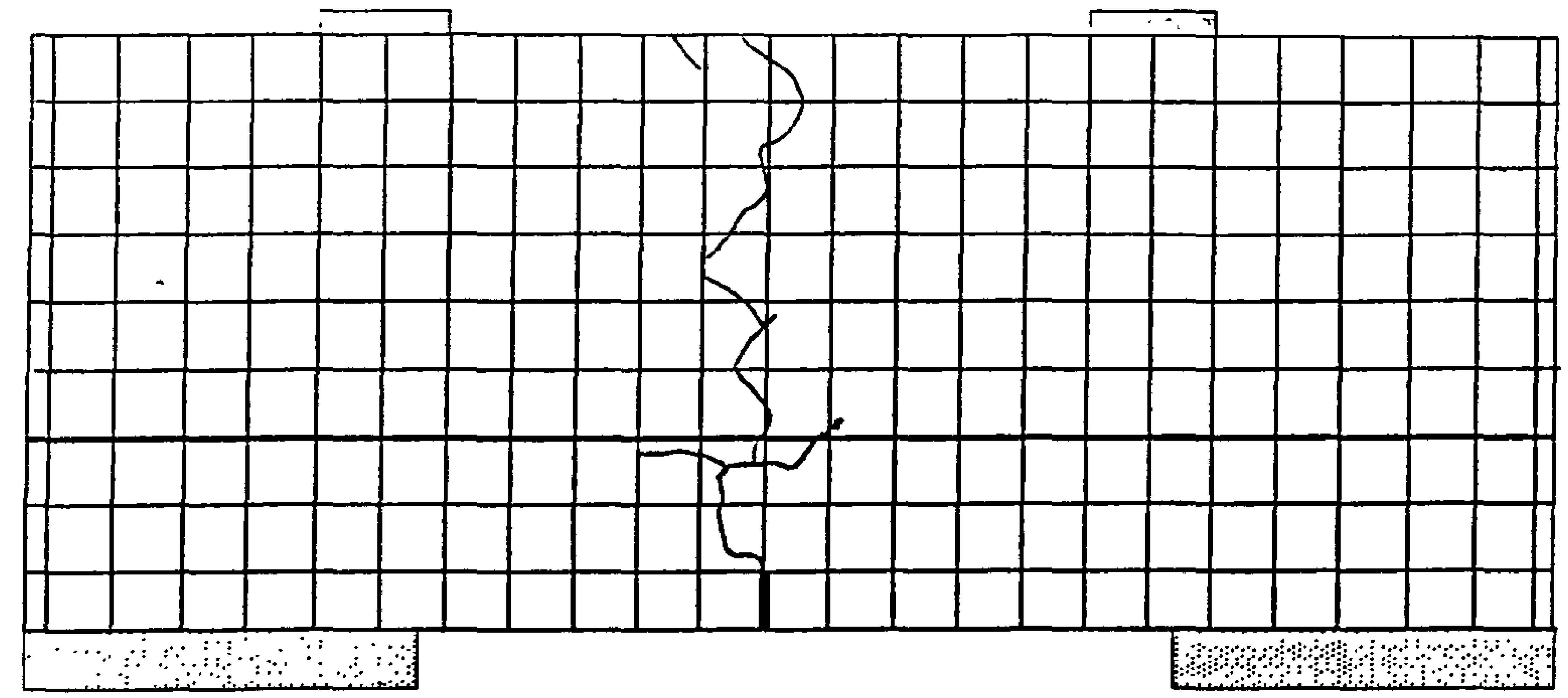
SPECIMEN:- C2
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



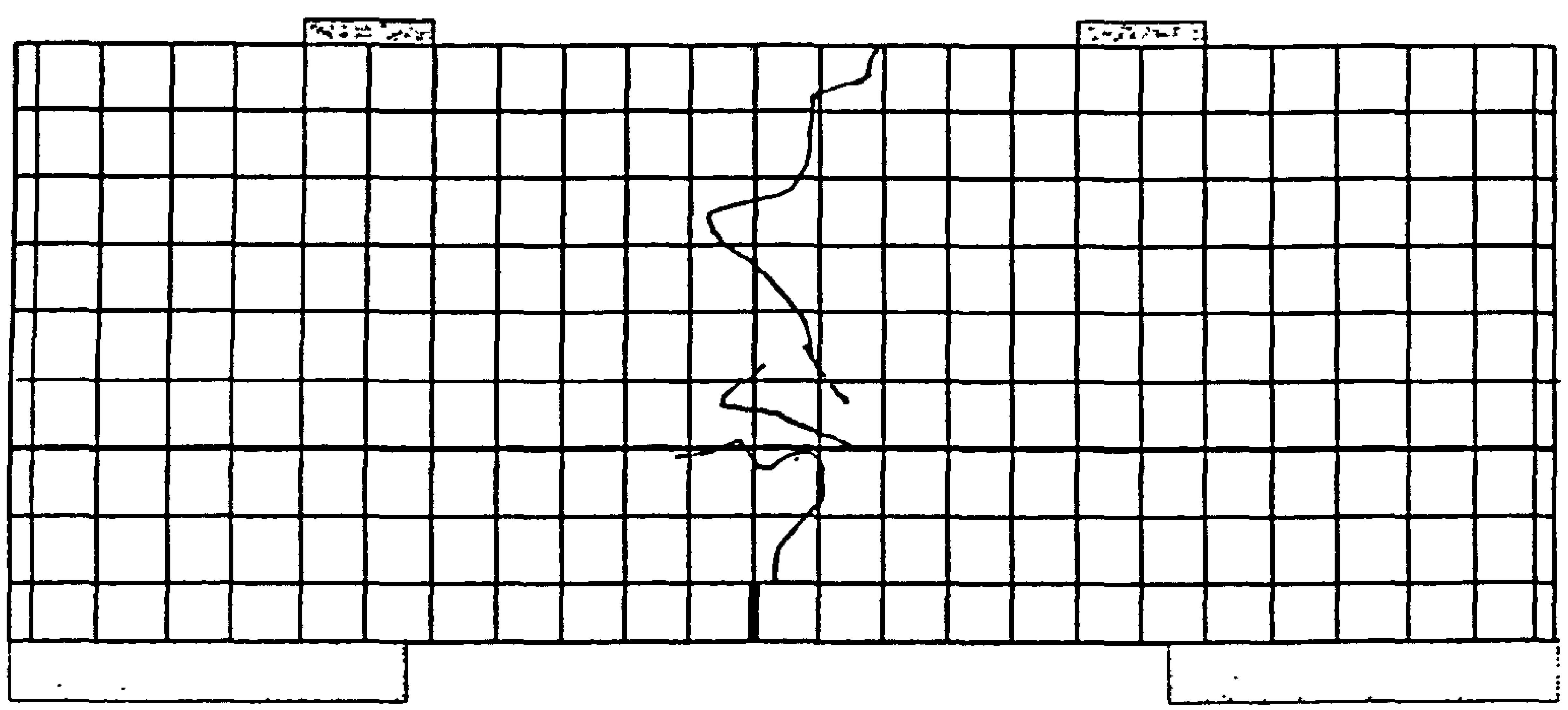
SPECIMEN:- C1
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



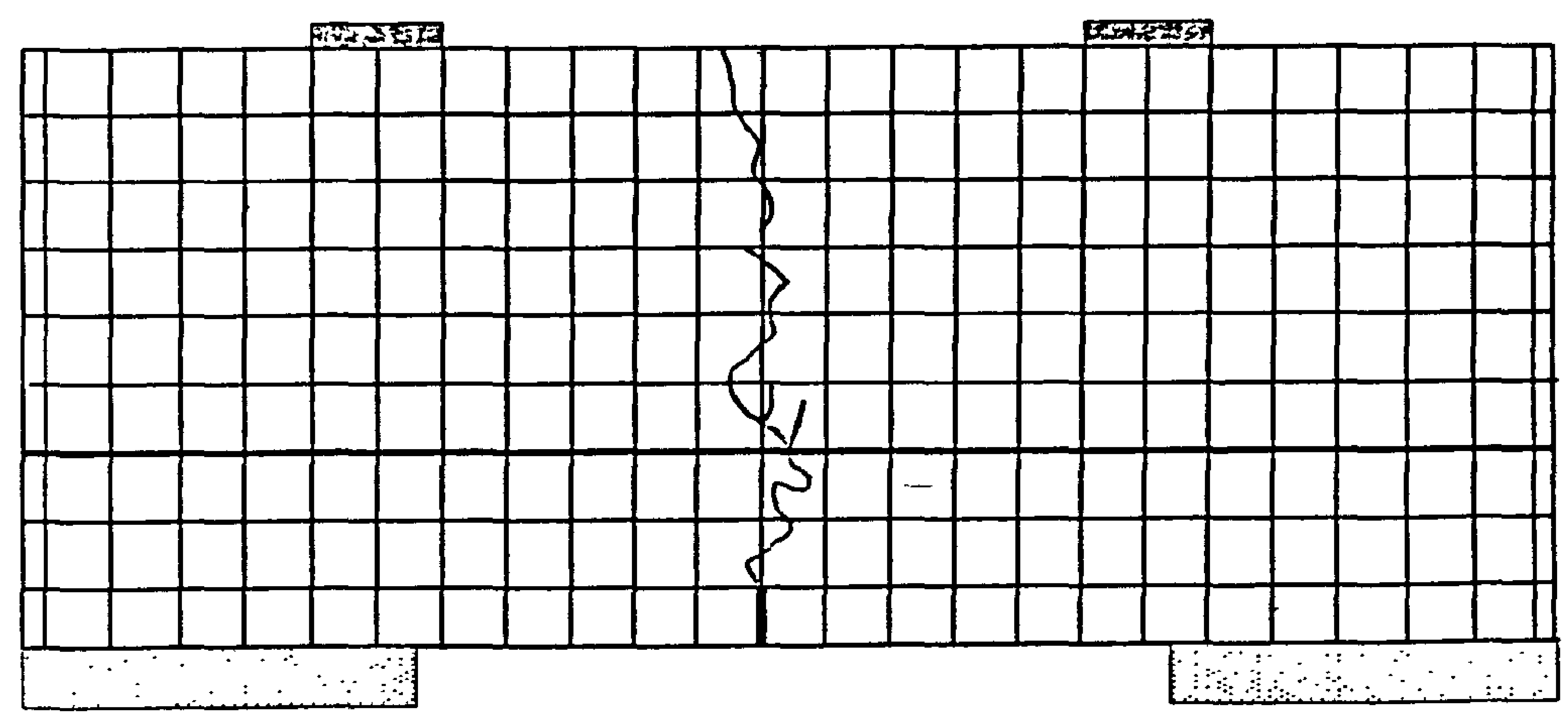
SPECIMEN:- GG2
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



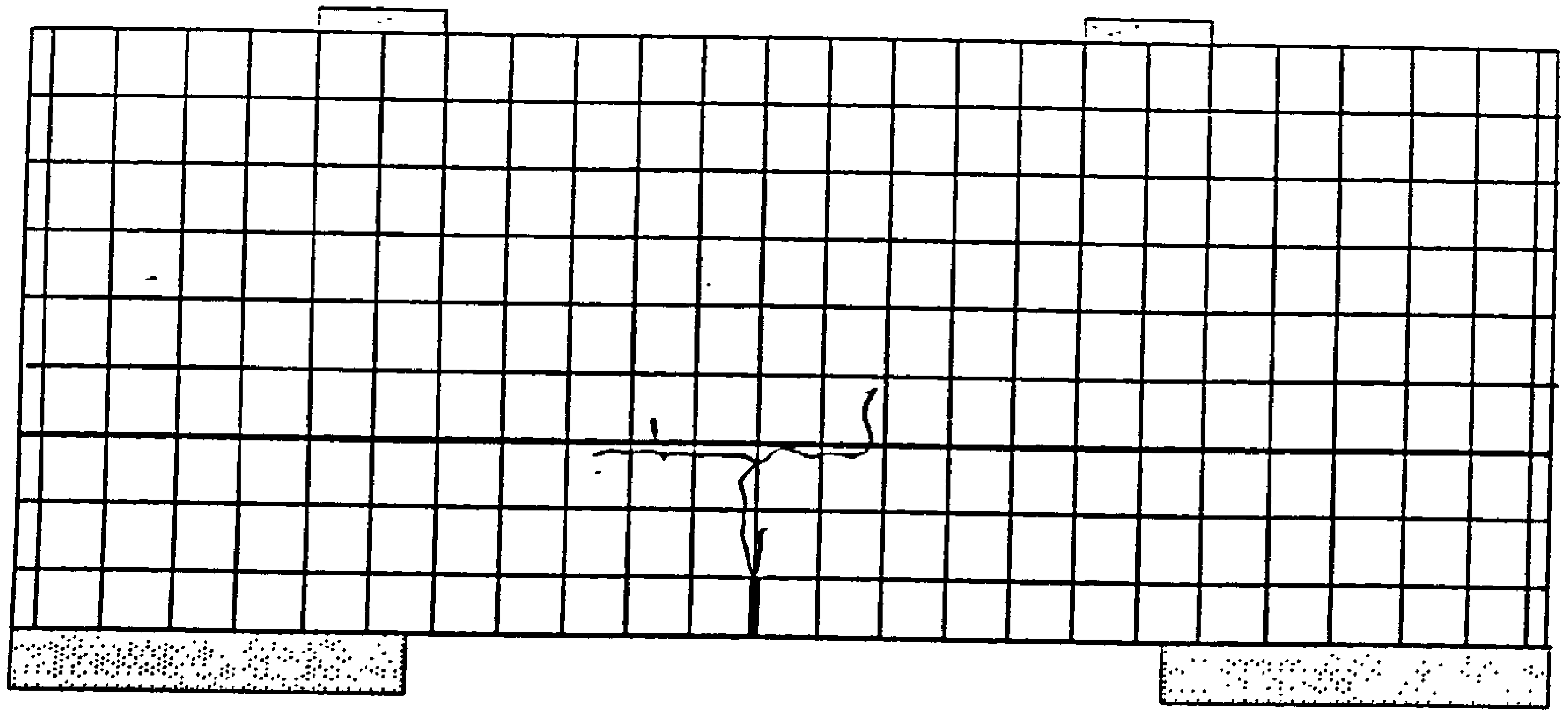
SPECIMEN:- C1
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



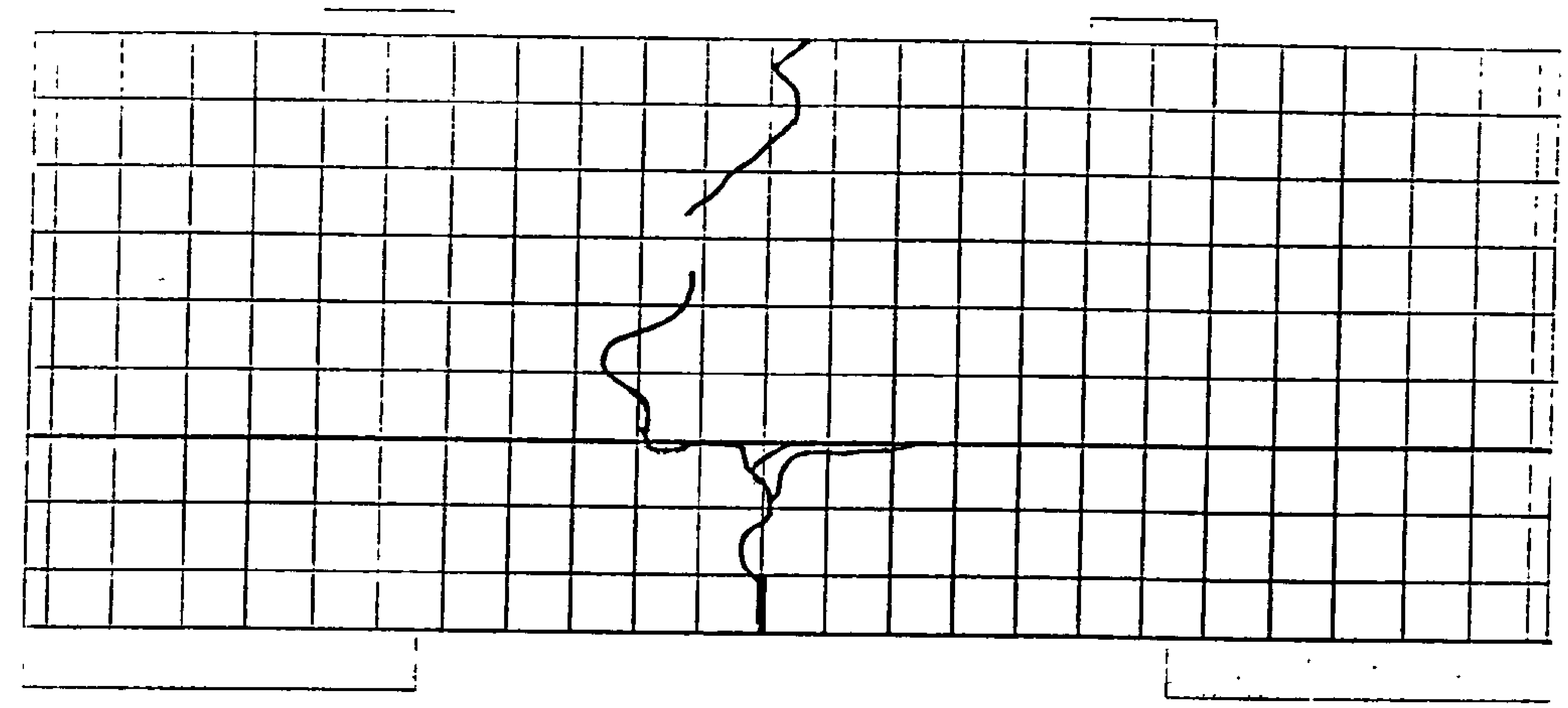
SPECIMEN:- GG2
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



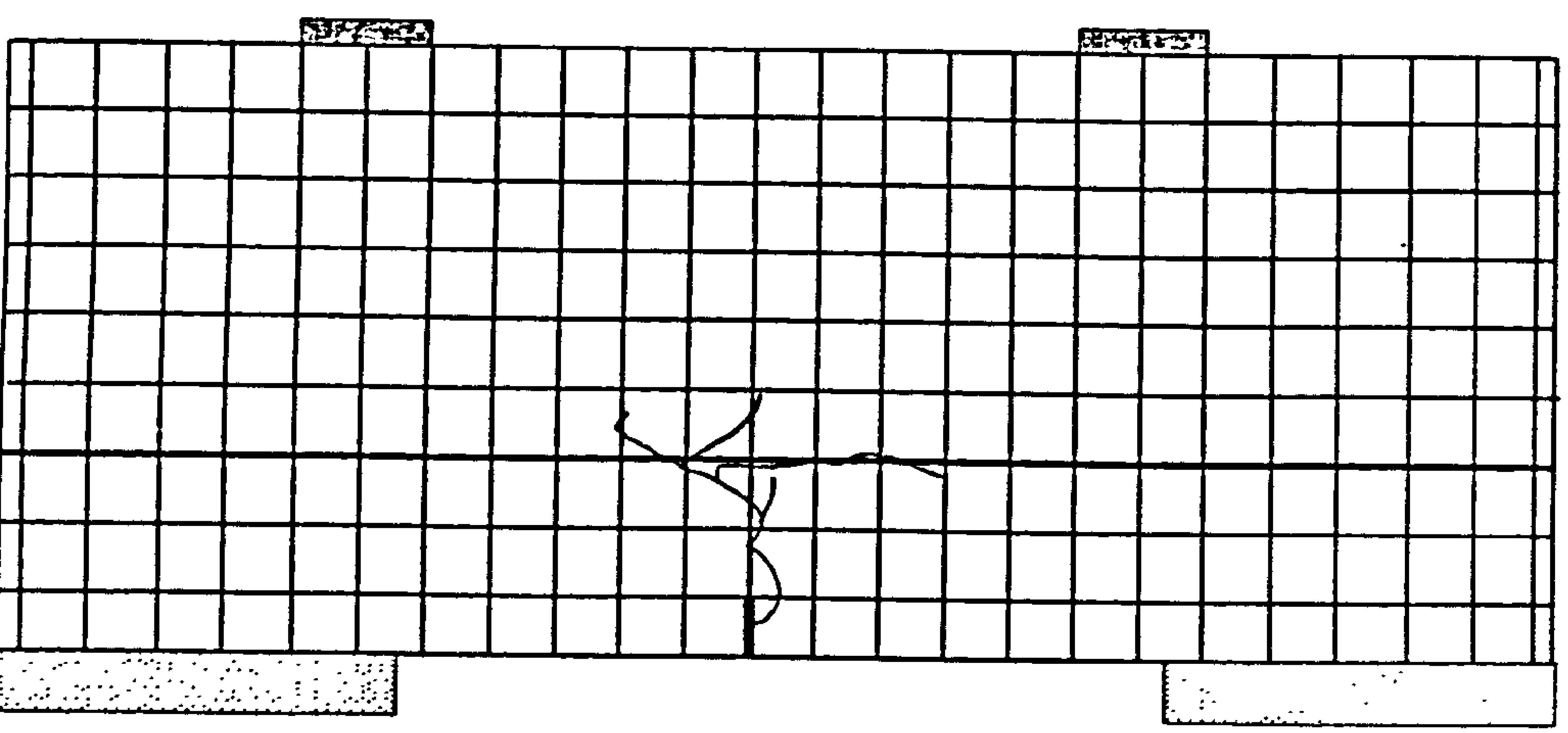
SPECIMEN:- GG1
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



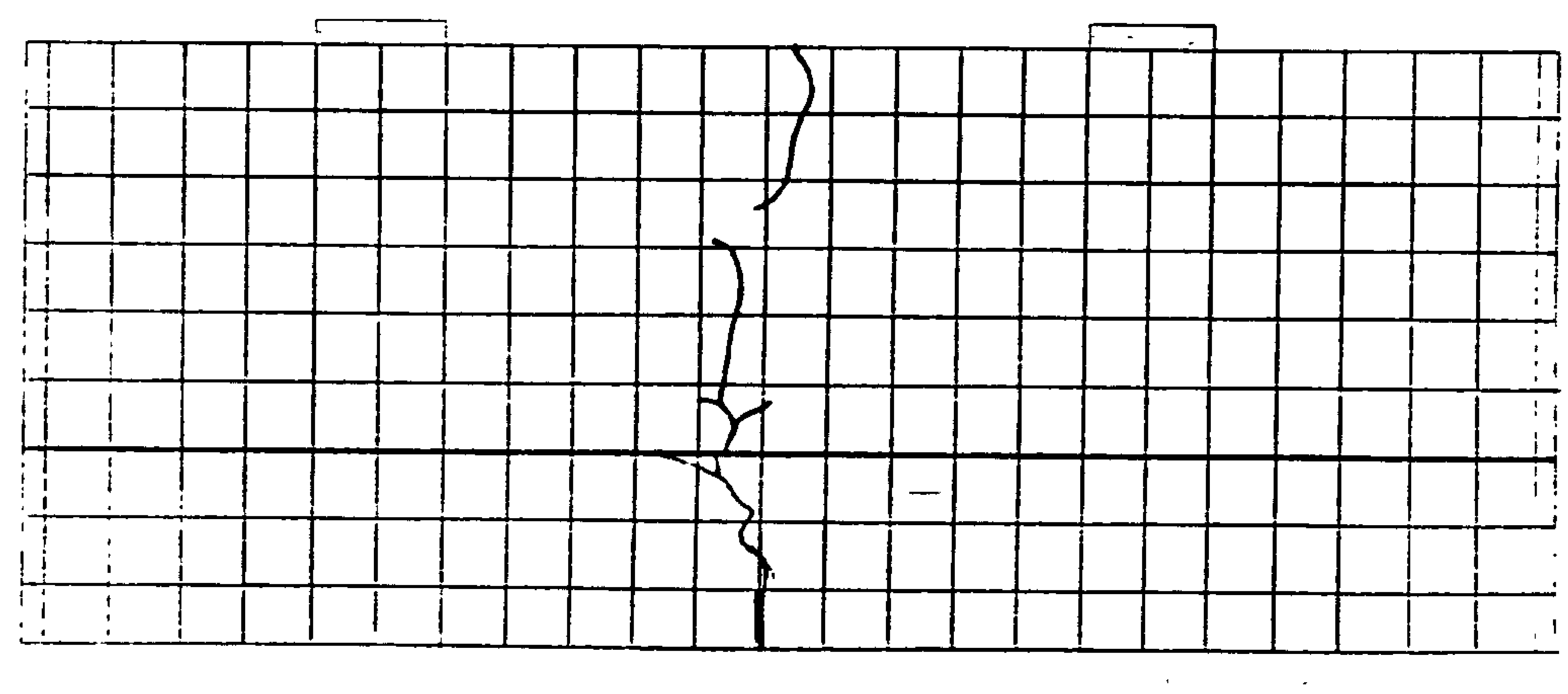
SPECIMEN:- PG2
FRONT/BACK:-
TEST MODE:- Stress Control



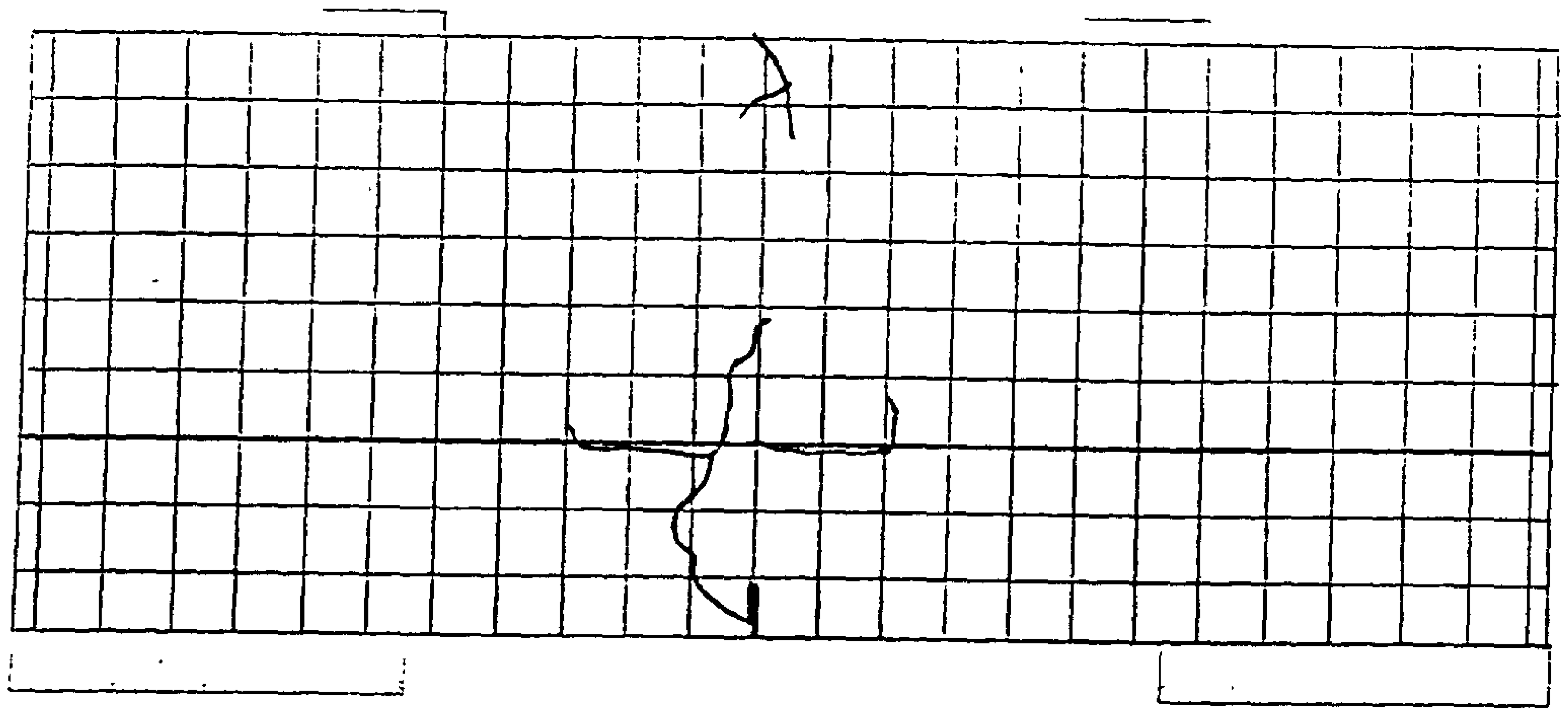
SPECIMEN:- GG1
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



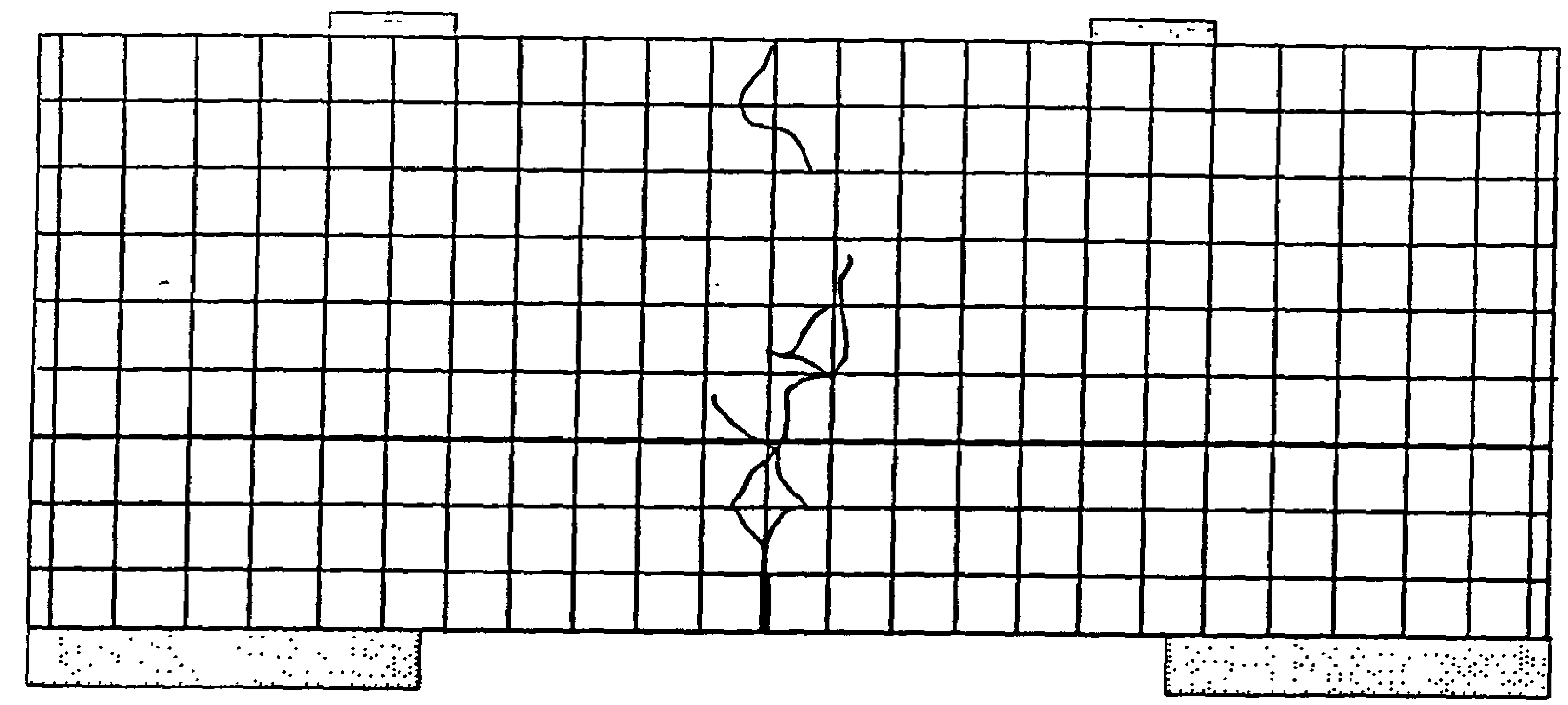
SPECIMEN:- PG2
FRONT/BACK:-
TEST MODE:- Stress Control



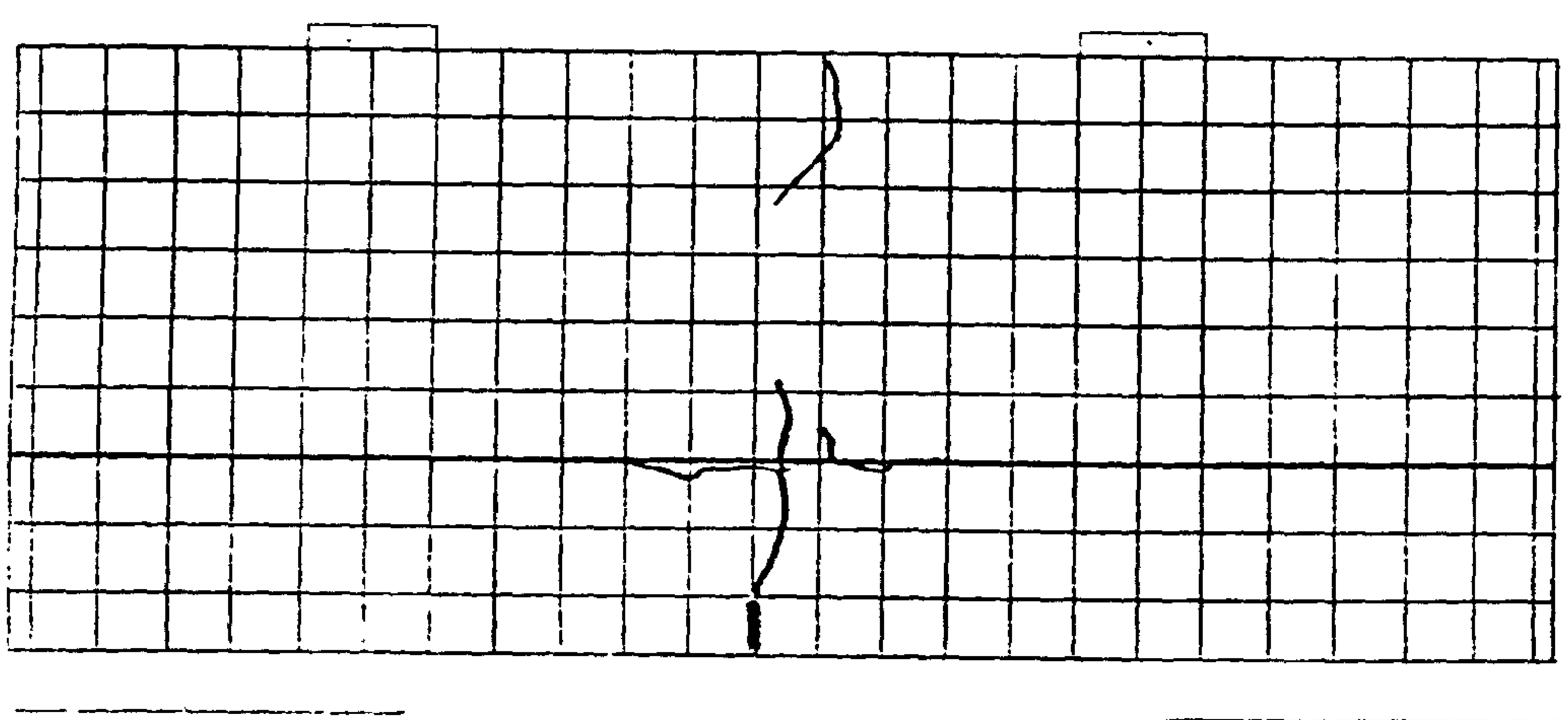
SPECIMEN:- PC2
FRONT/BACK:-
TEST MODE:- Stress Control



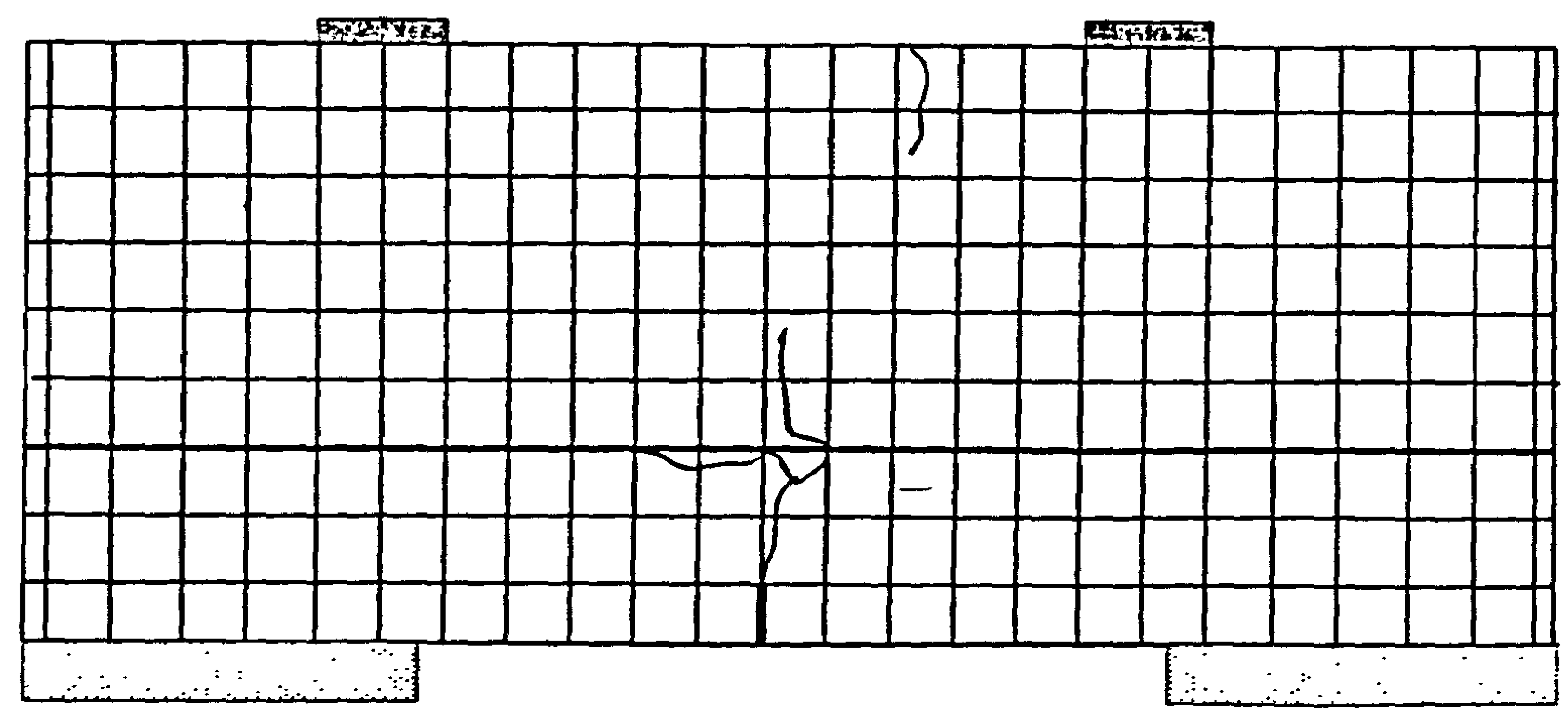
SPECIMEN:- PG1
FRONT/BACK:-
TEST MODE:- Stress Control



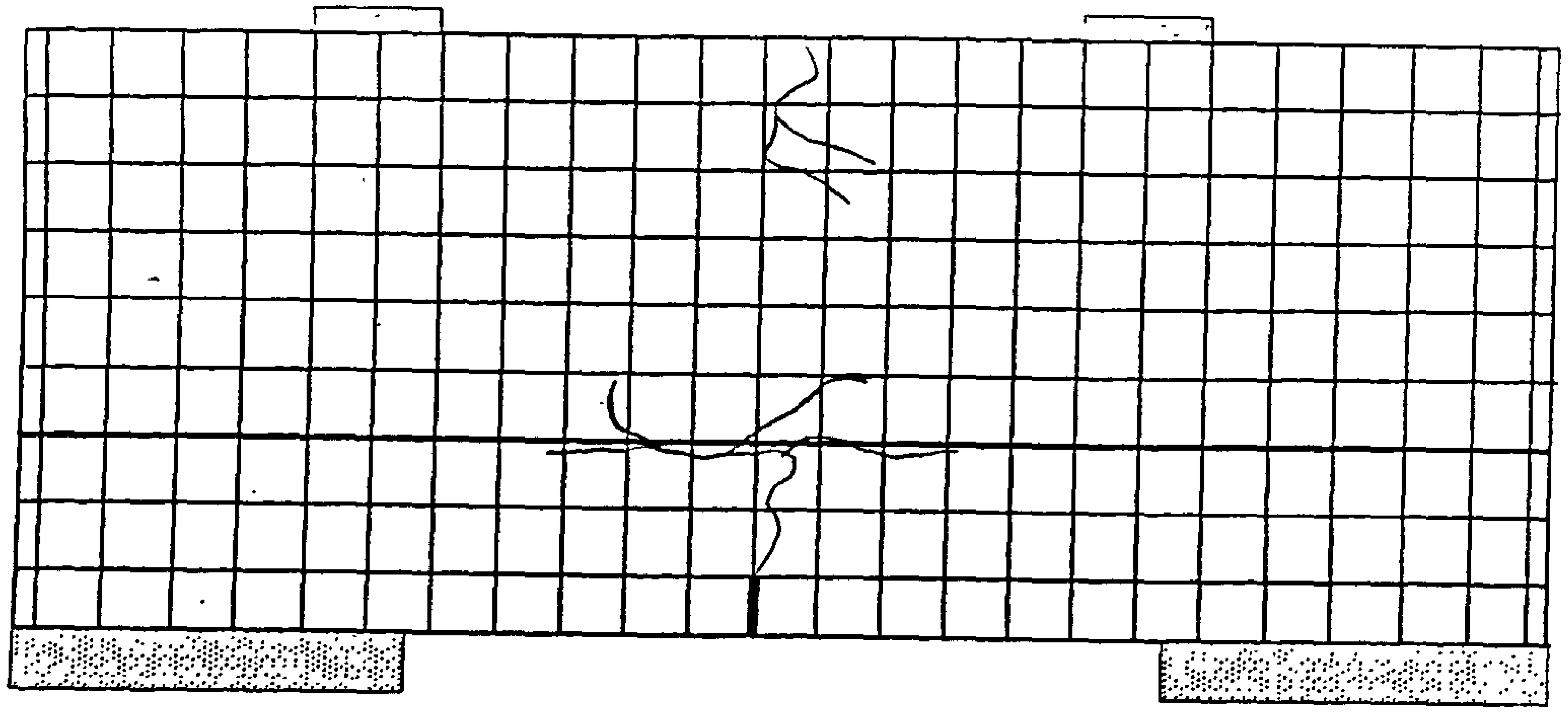
SPECIMEN:- PC2
FRONT/BACK:-
TEST MODE:- Stress Control



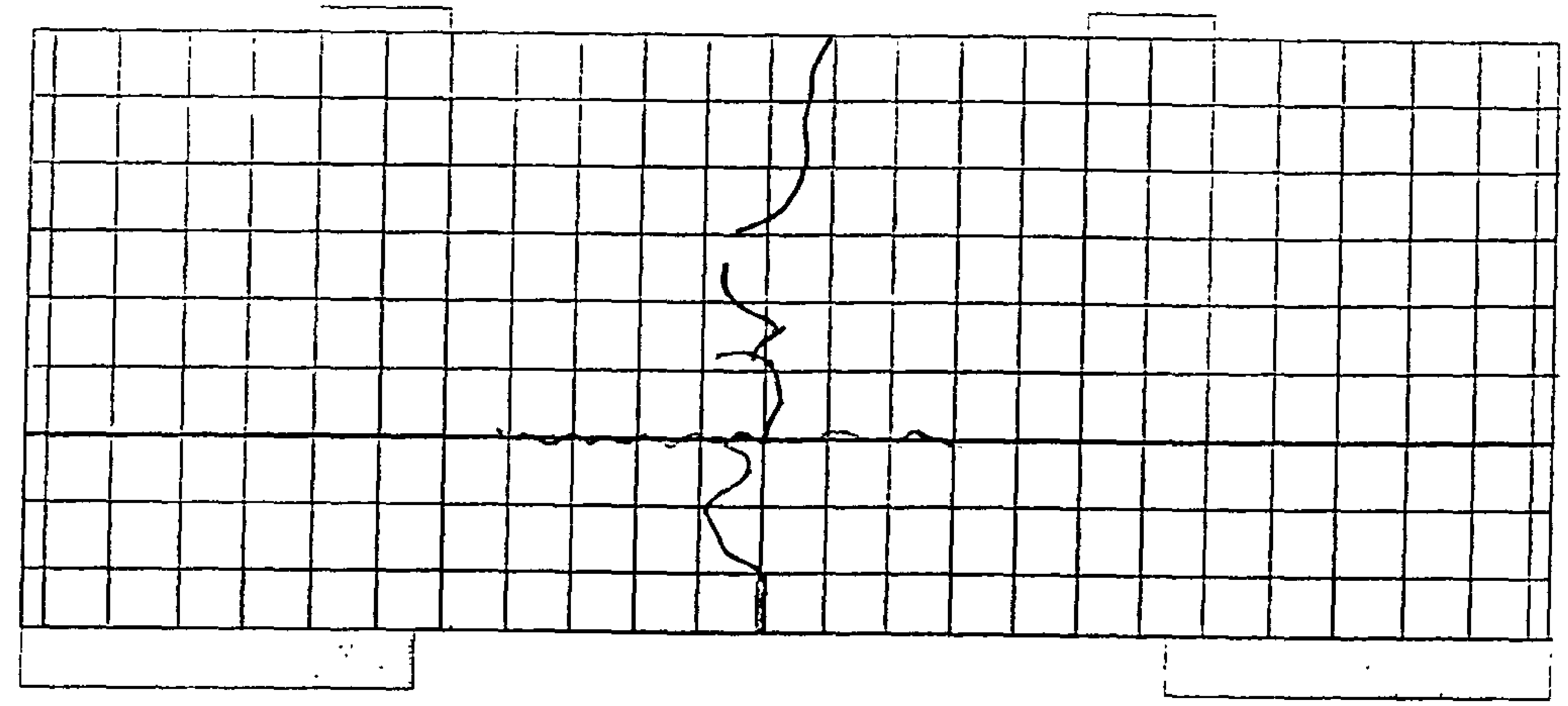
SPECIMEN:- PG1
FRONT/BACK:-
TEST MODE:- Stress Control



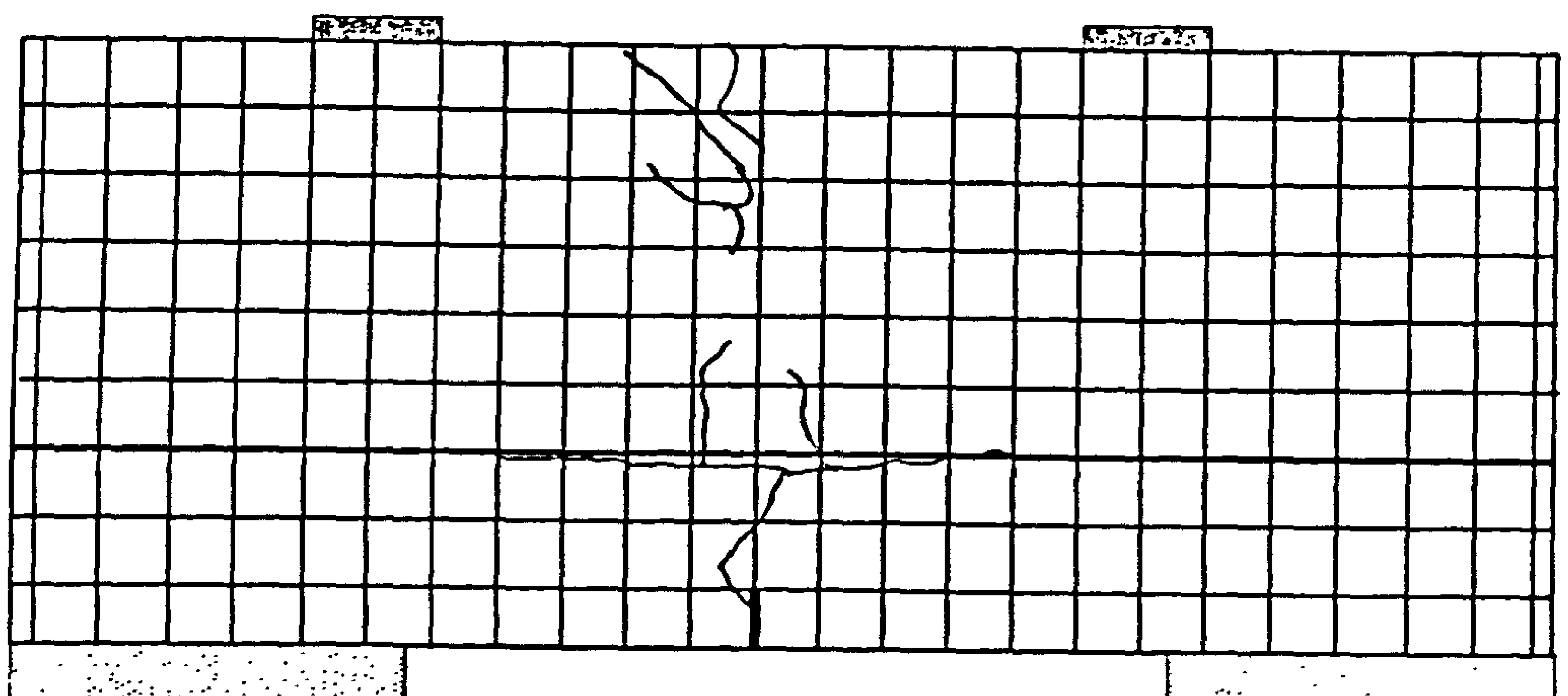
SPECIMEN:- GC6
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



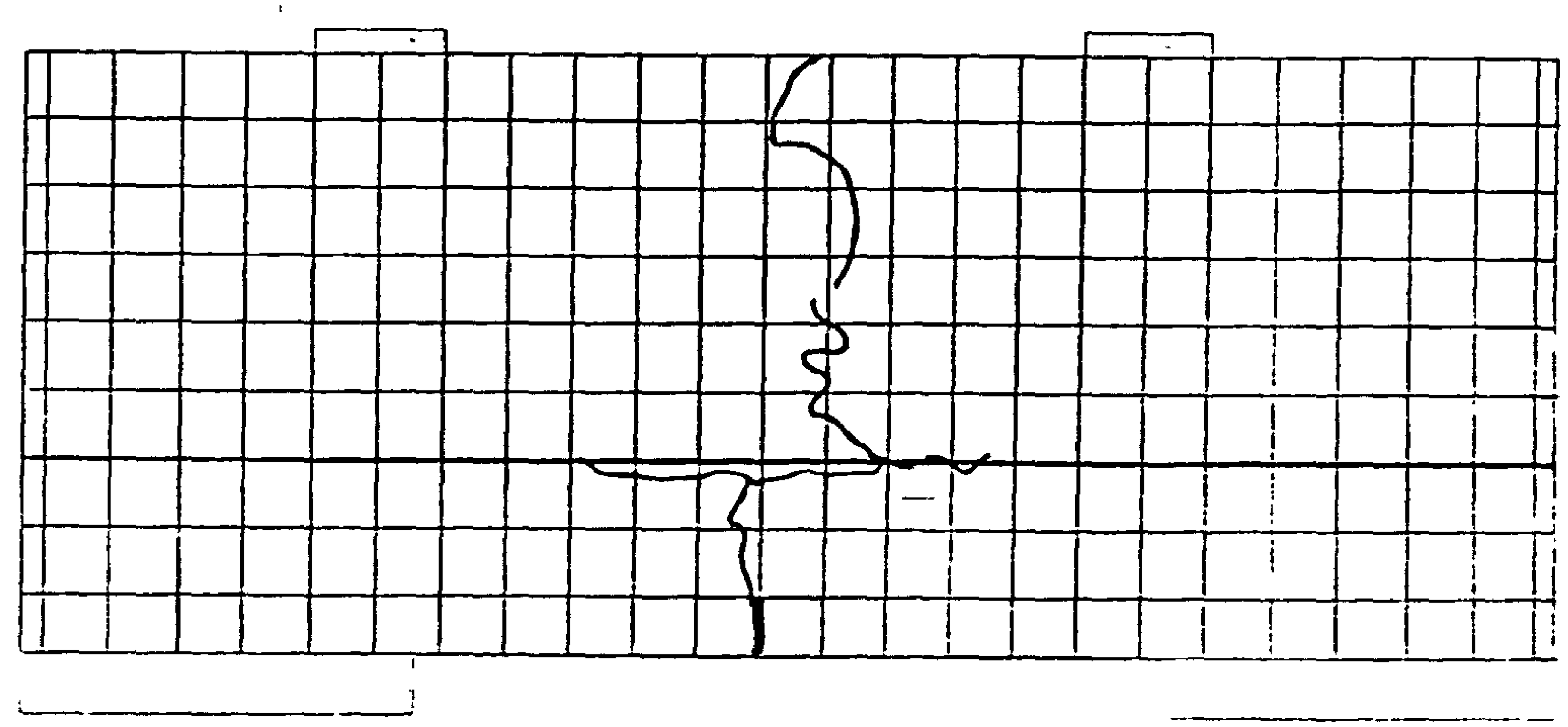
SPECIMEN:- PC1
FRONT/BACK:-
TEST MODE:- Stress Control



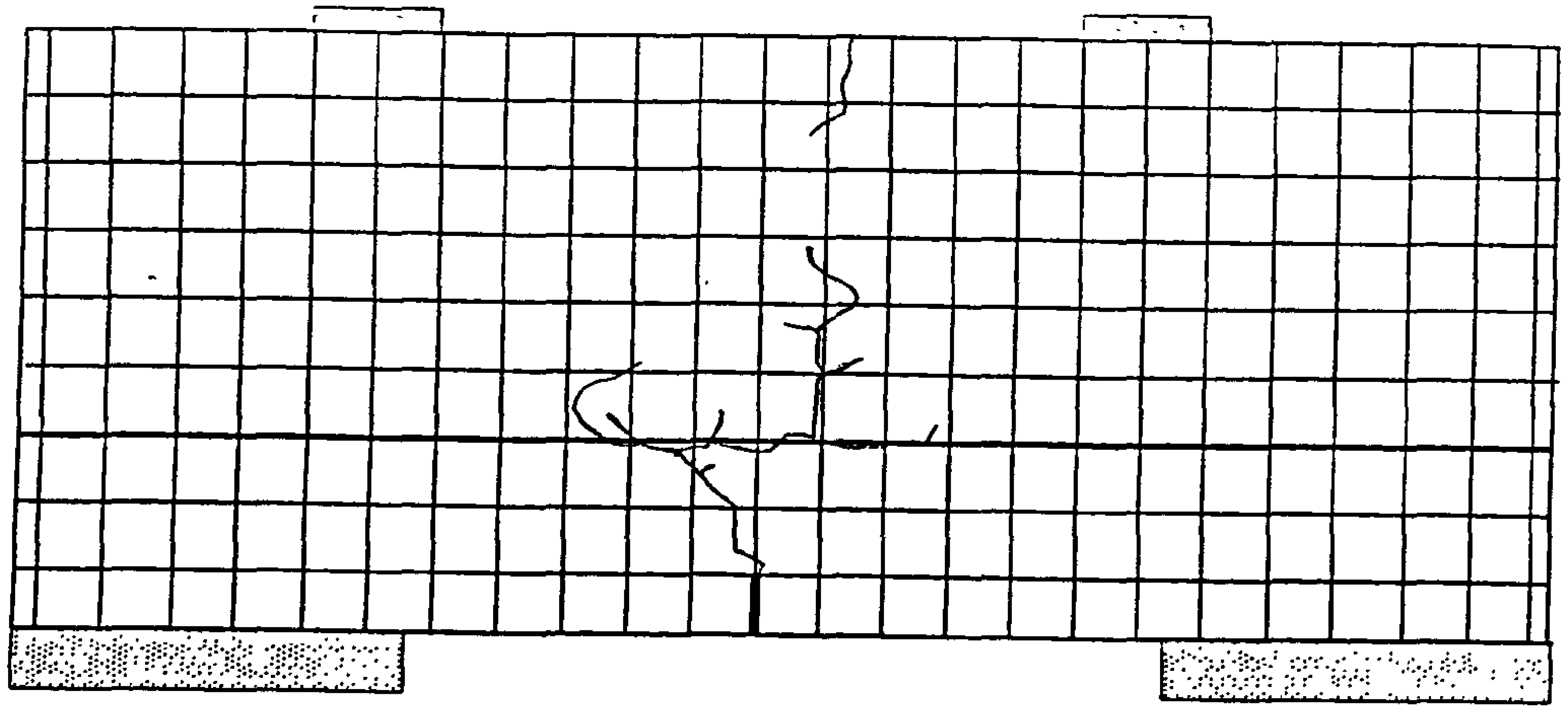
SPECIMEN:- GC6
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



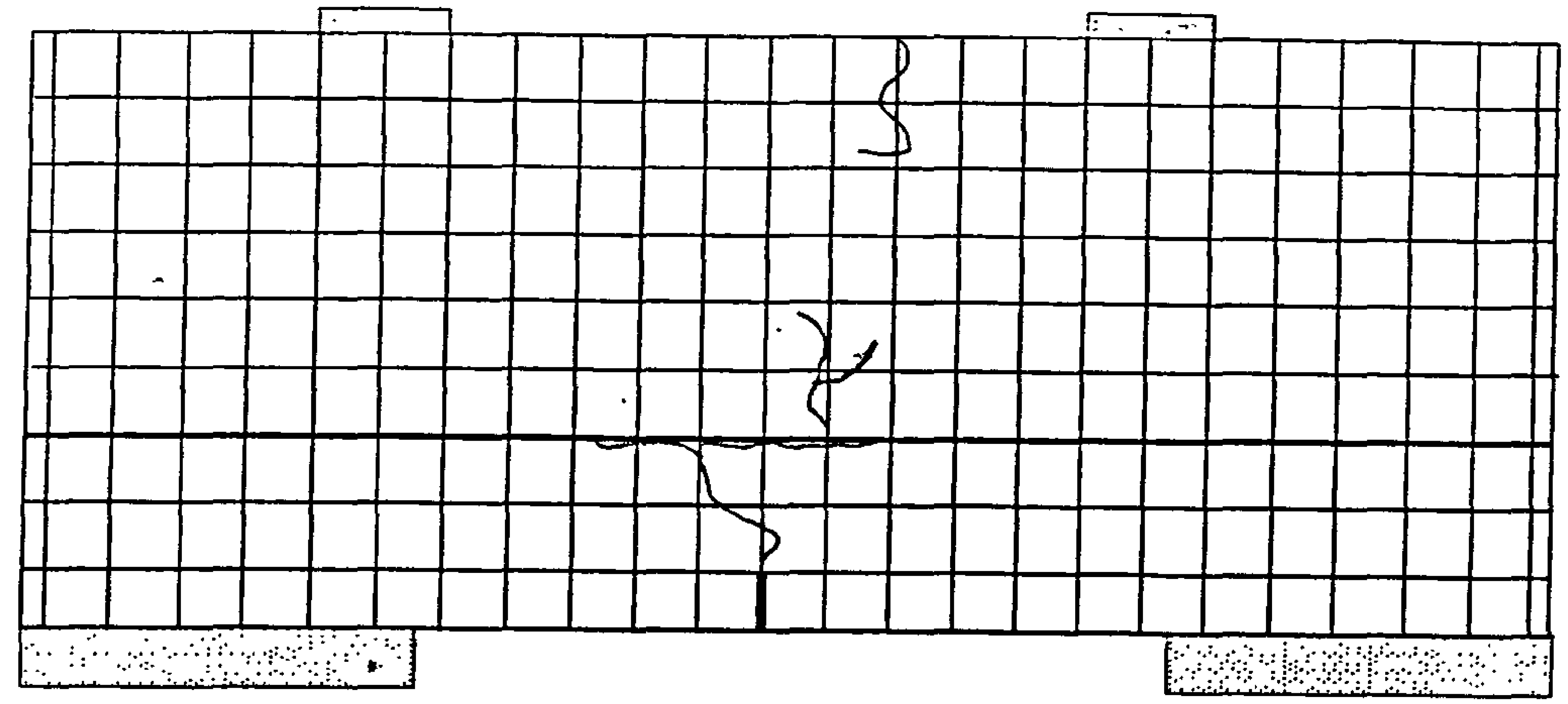
SPECIMEN:- PC1
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



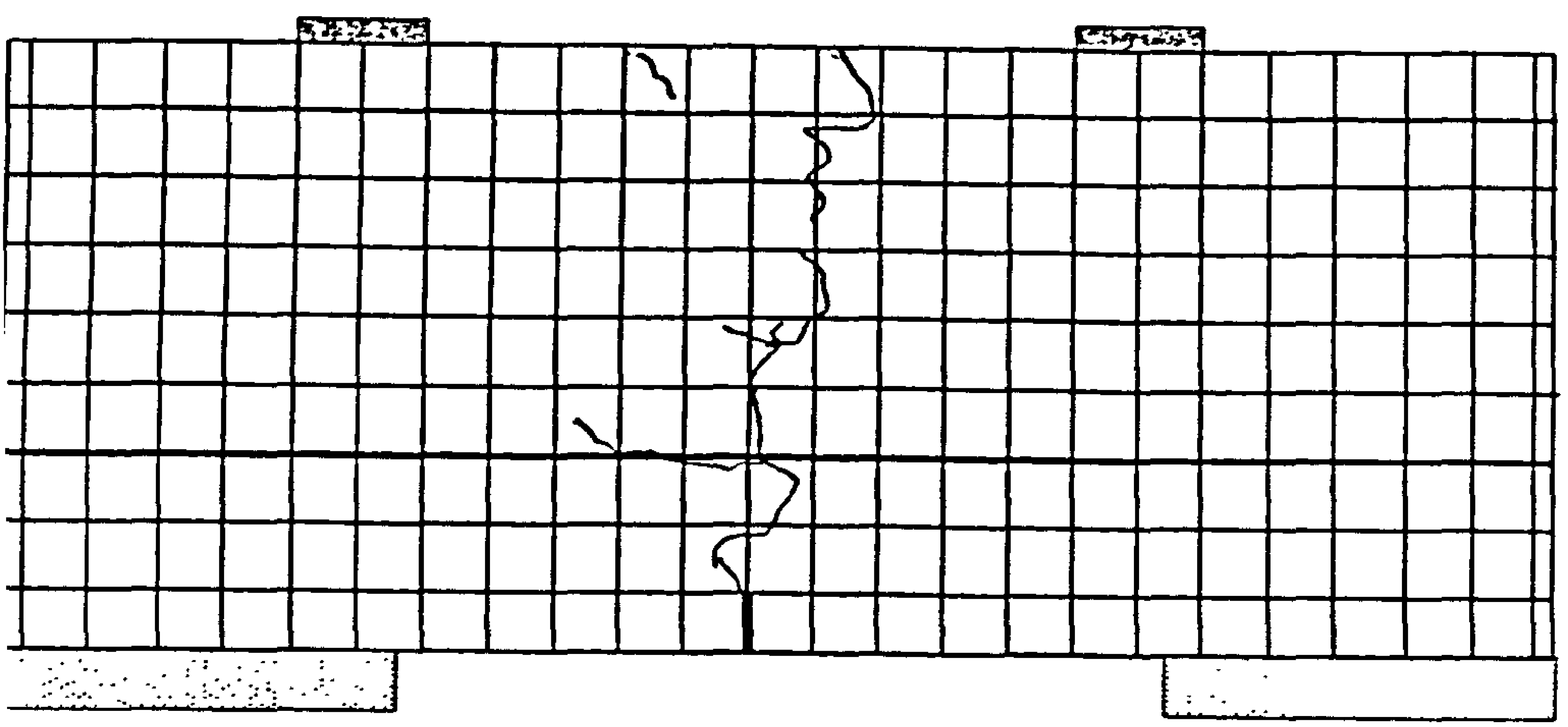
SPECIMEN:- GC4
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



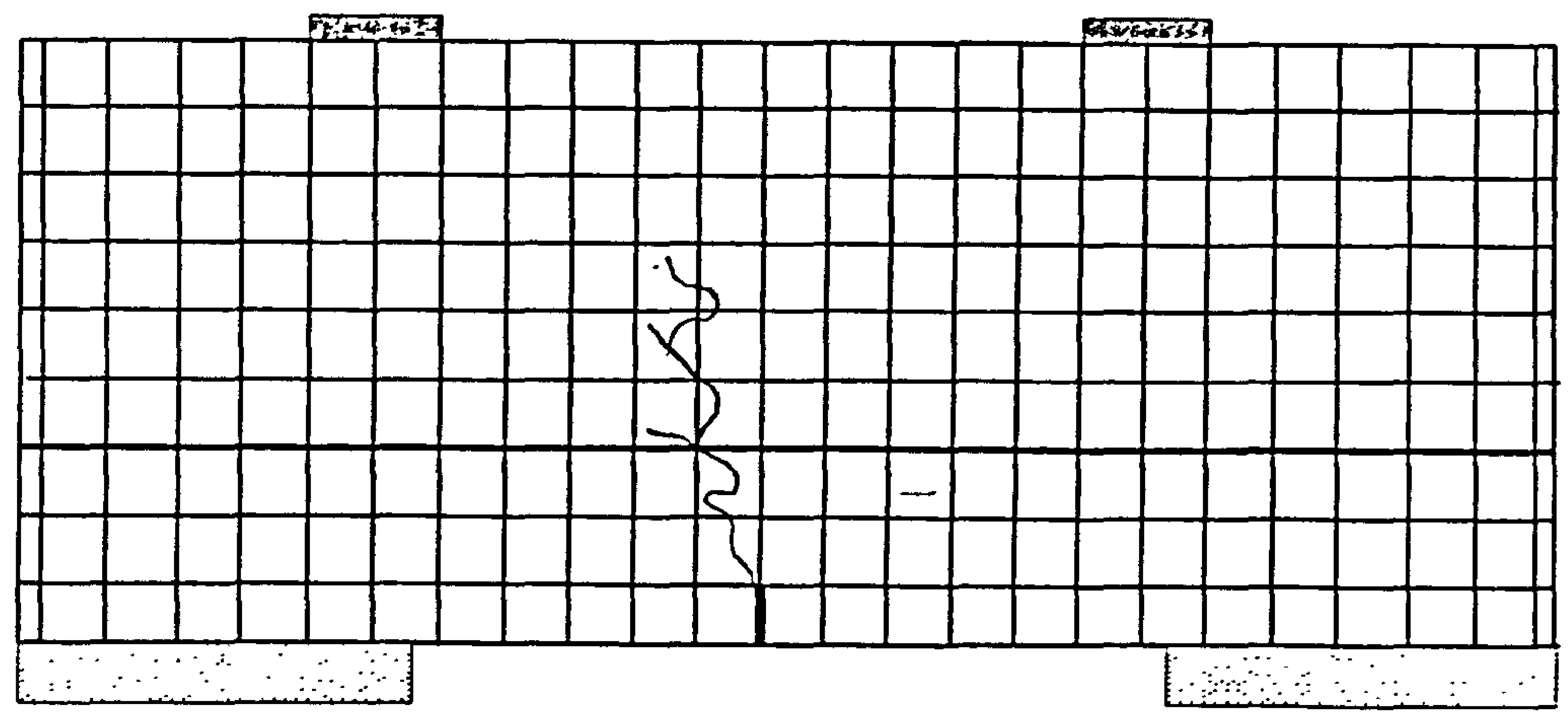
SPECIMEN:- GC5
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



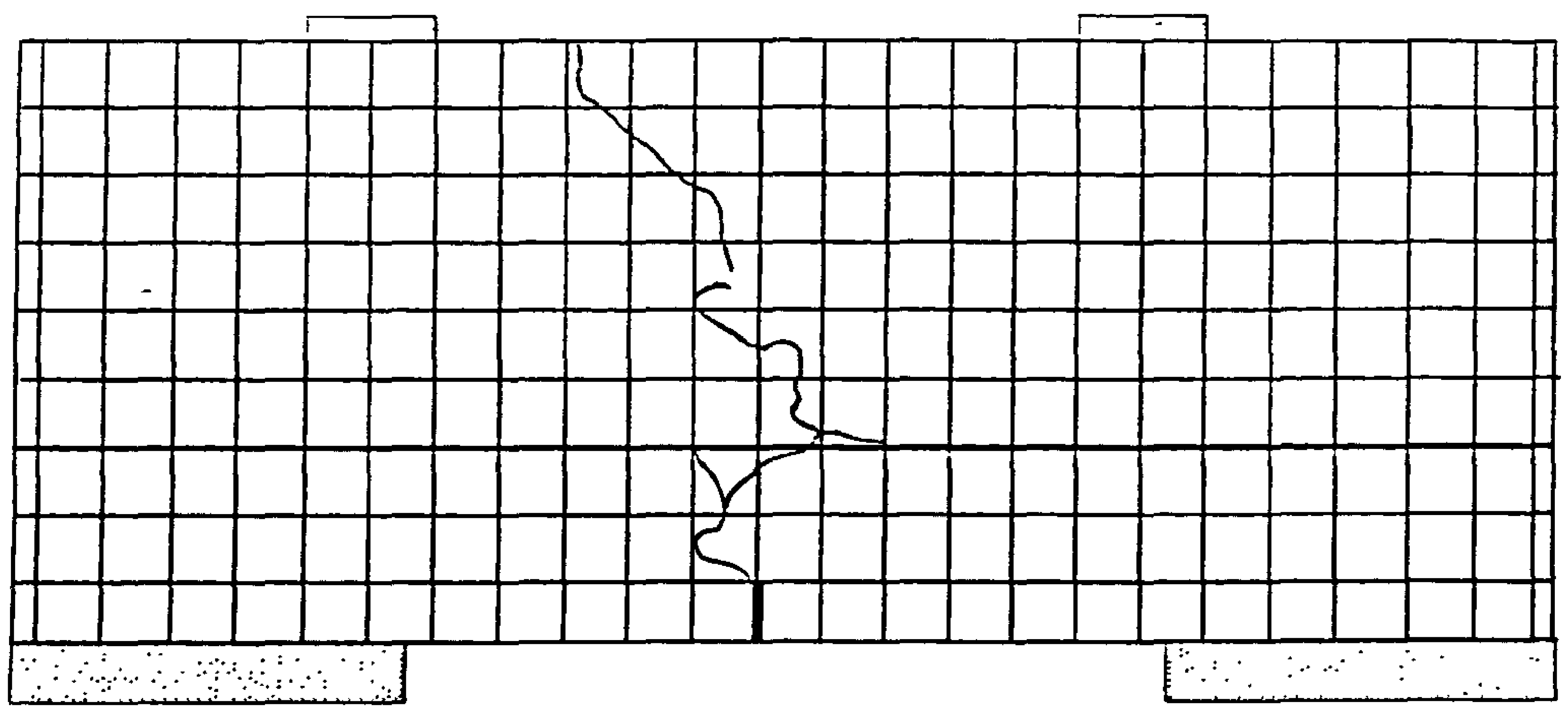
SPECIMEN:- GC4
FRONT/BACK:-
TEST MODE:- Stress Control



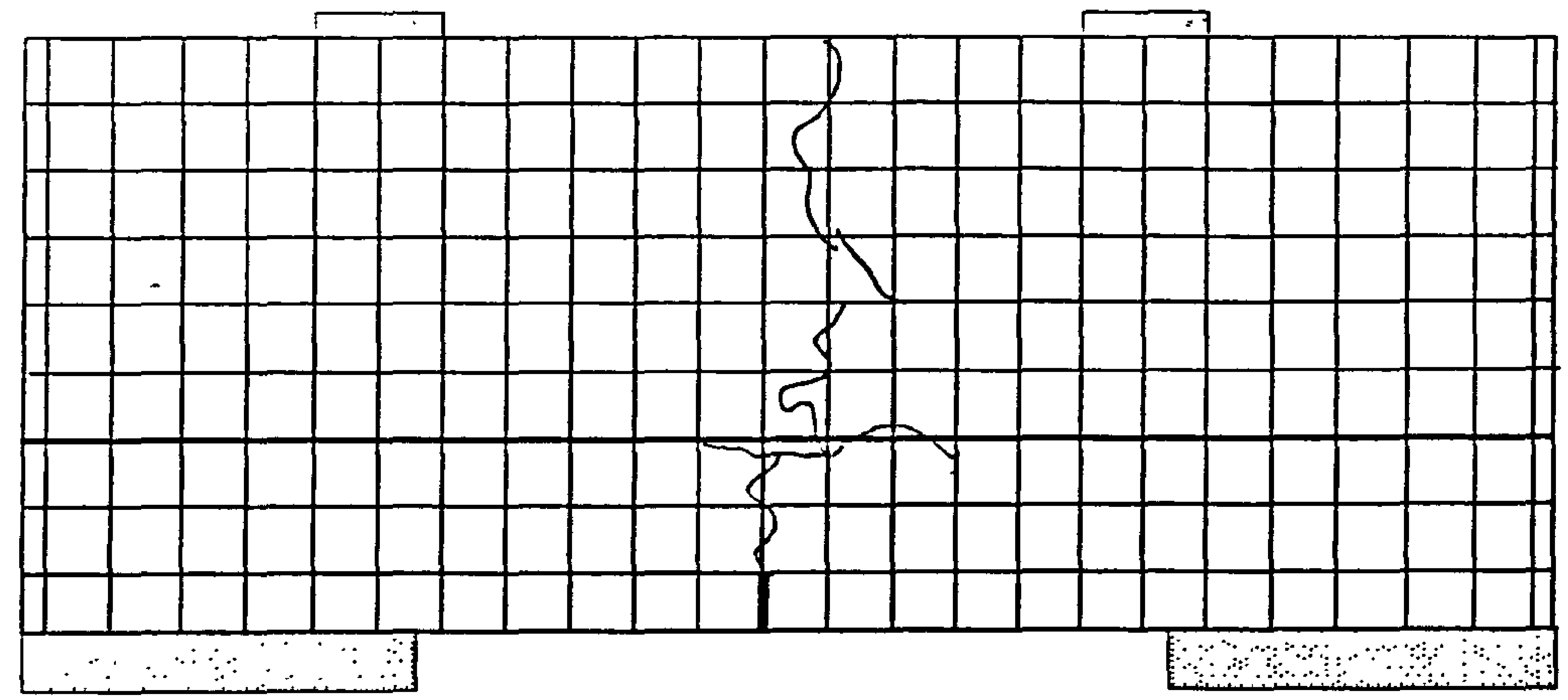
SPECIMEN:- GC5
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



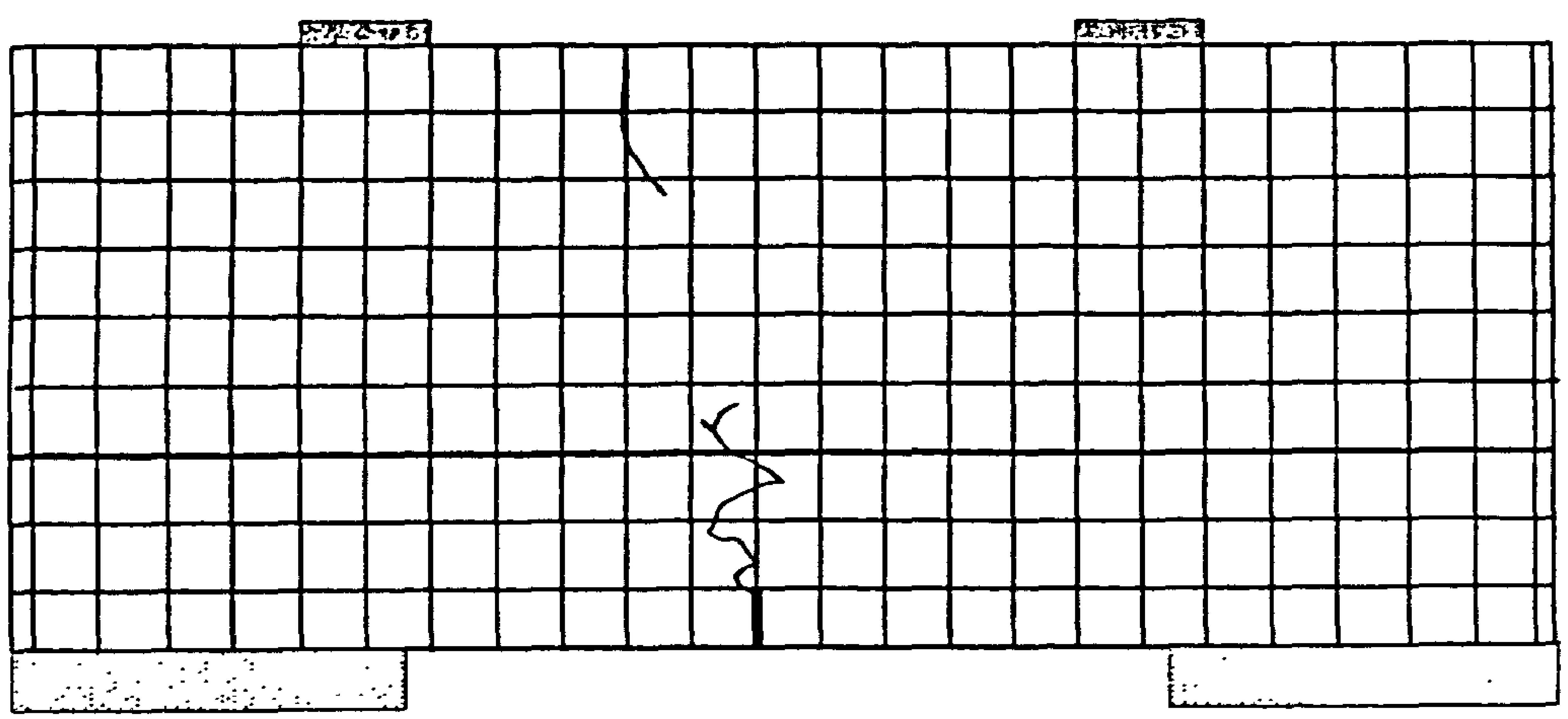
SPECIMEN:- GC2
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



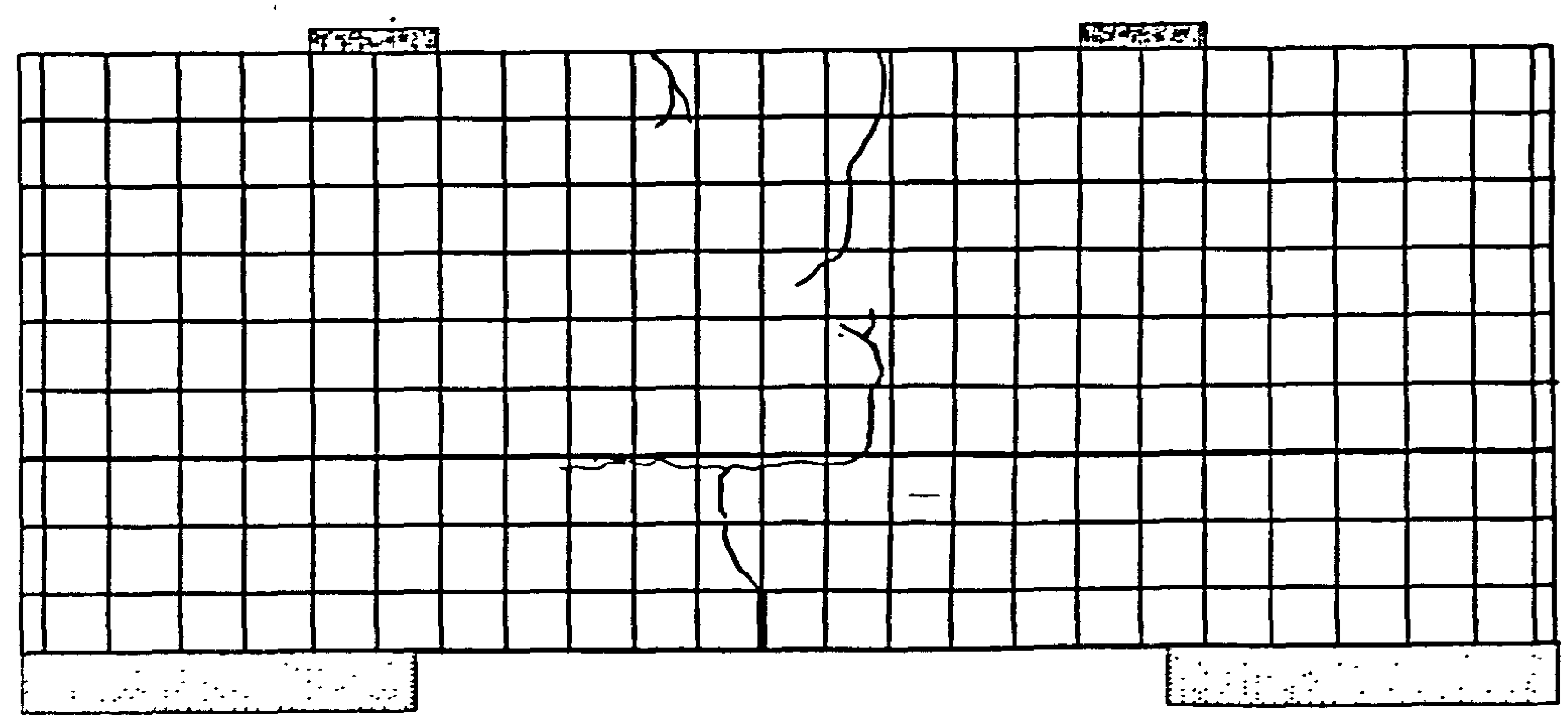
SPECIMEN:- GC3
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



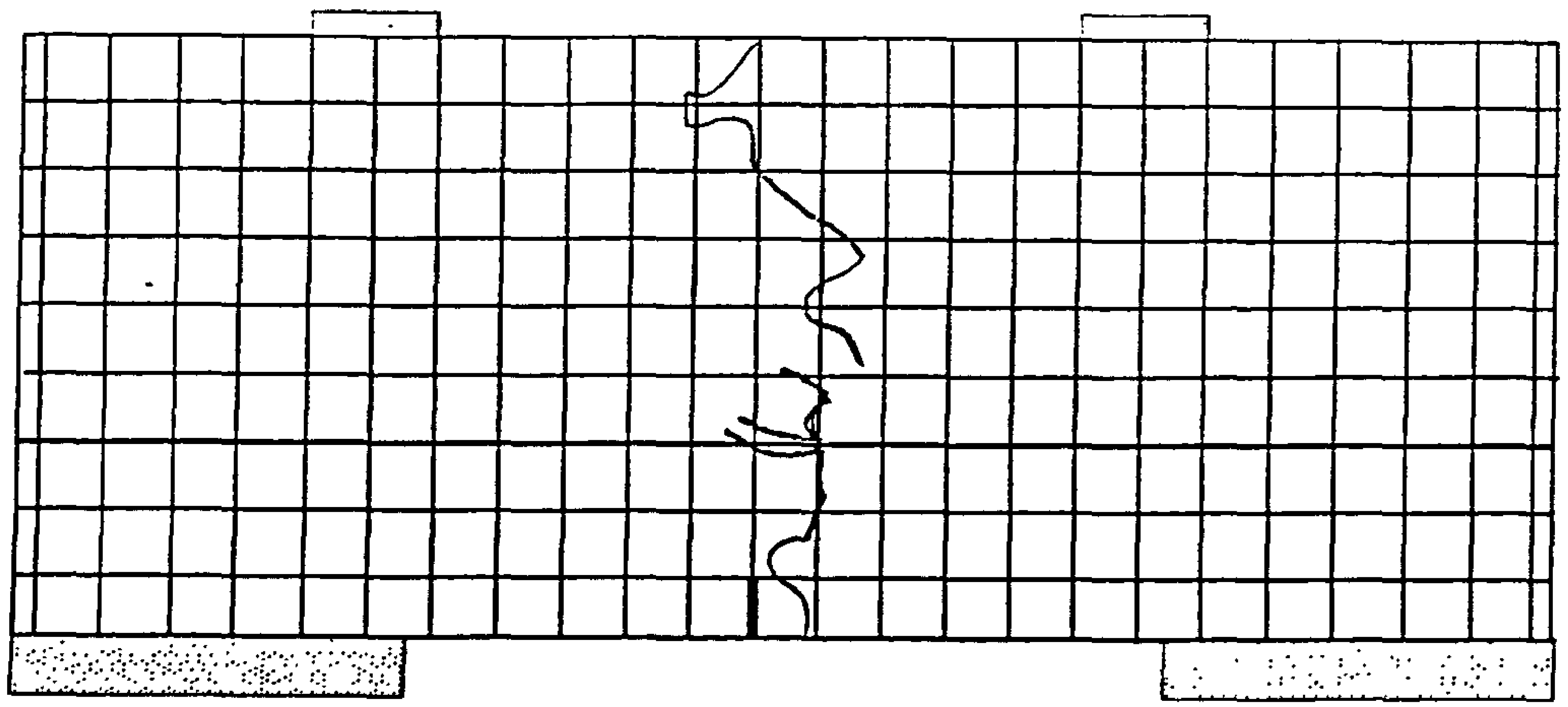
SPECIMEN:- GC2
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



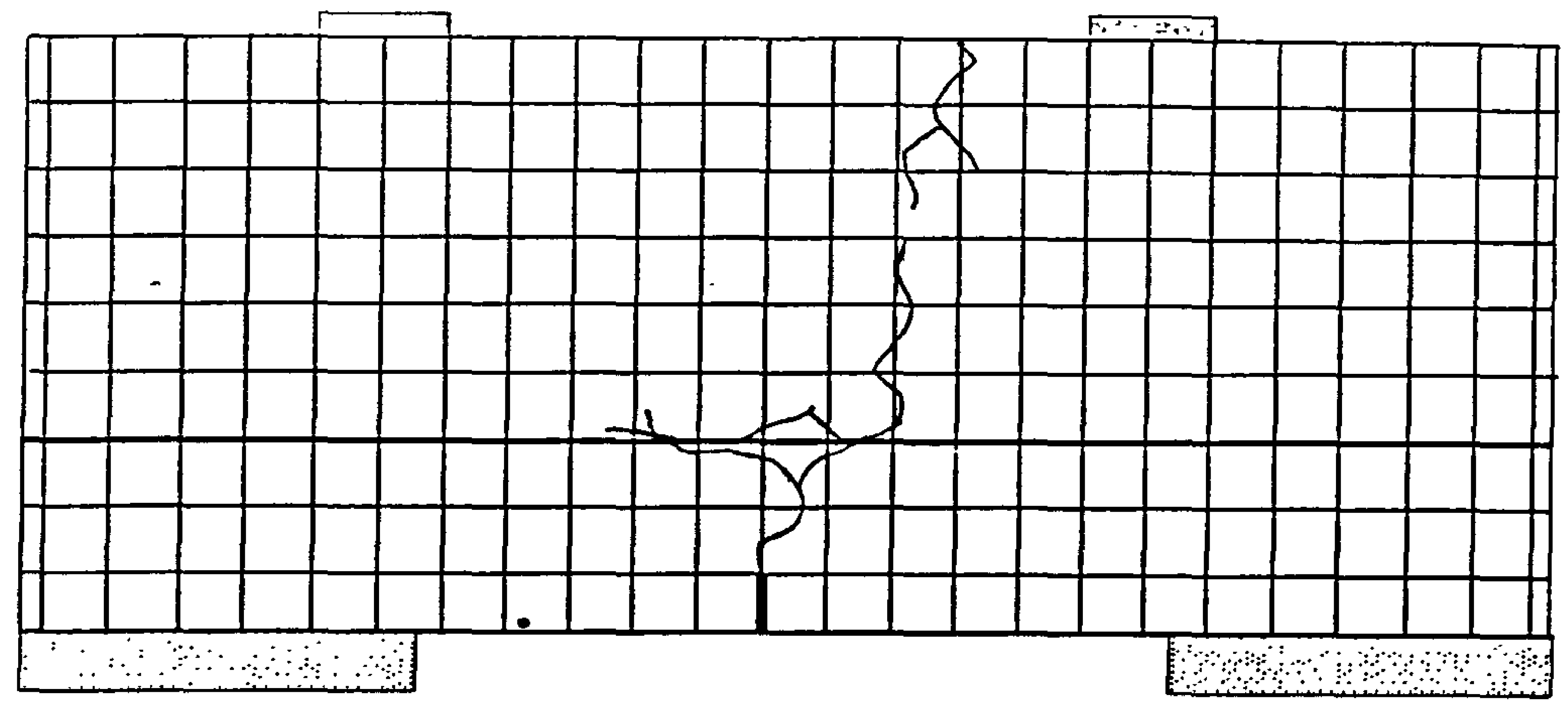
SPECIMEN:- GC3
FRONT/BACK:-
TEST MODE:- Stress Control



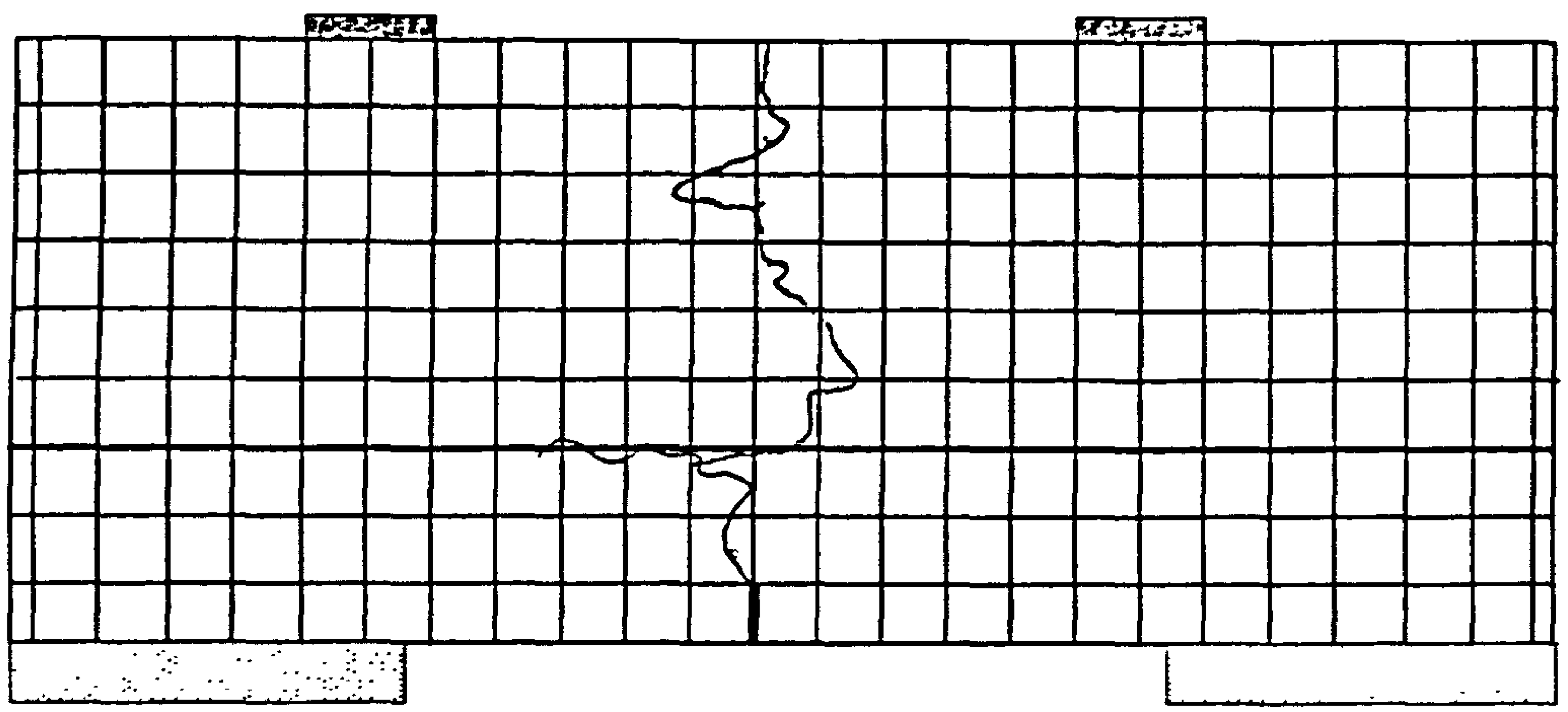
SPECIMEN:- F2
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



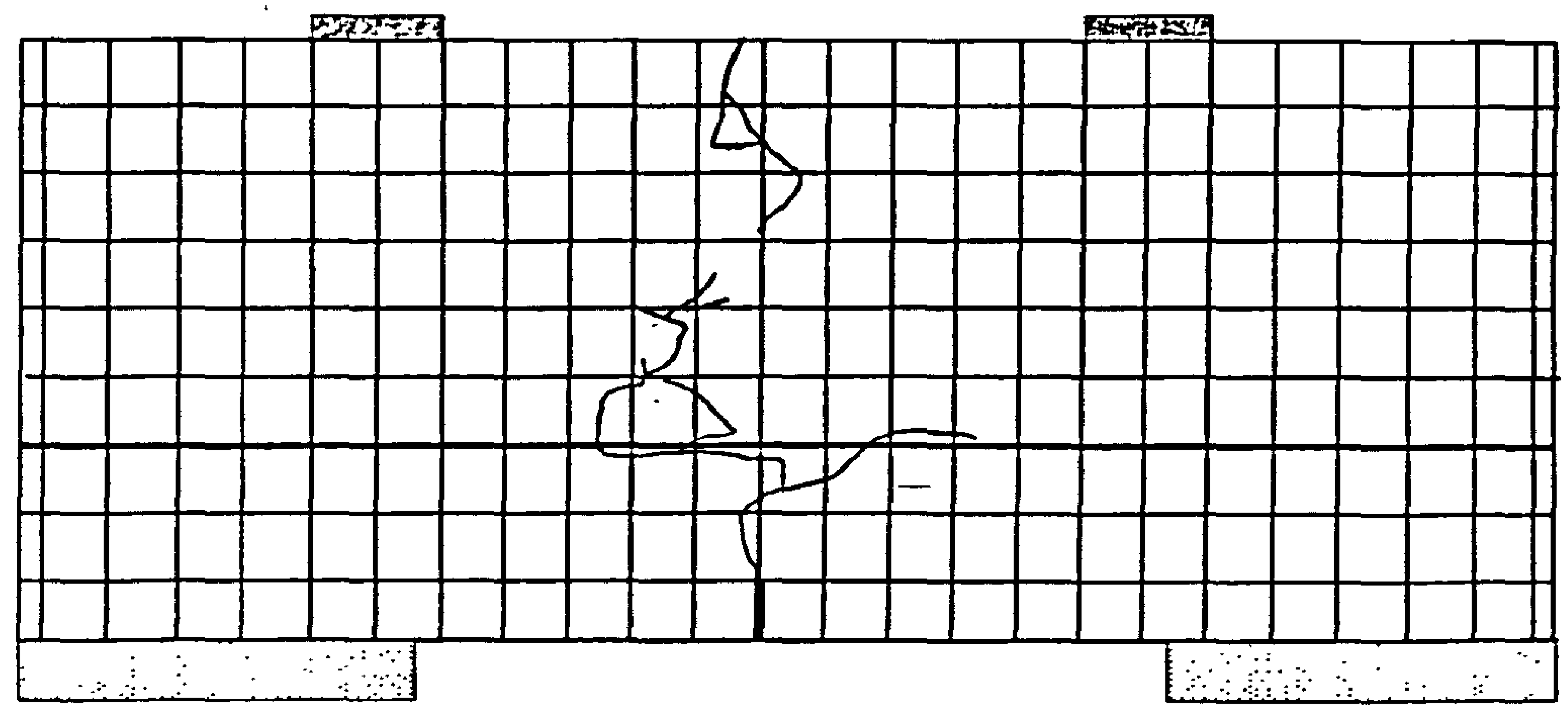
SPECIMEN:- GC1
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



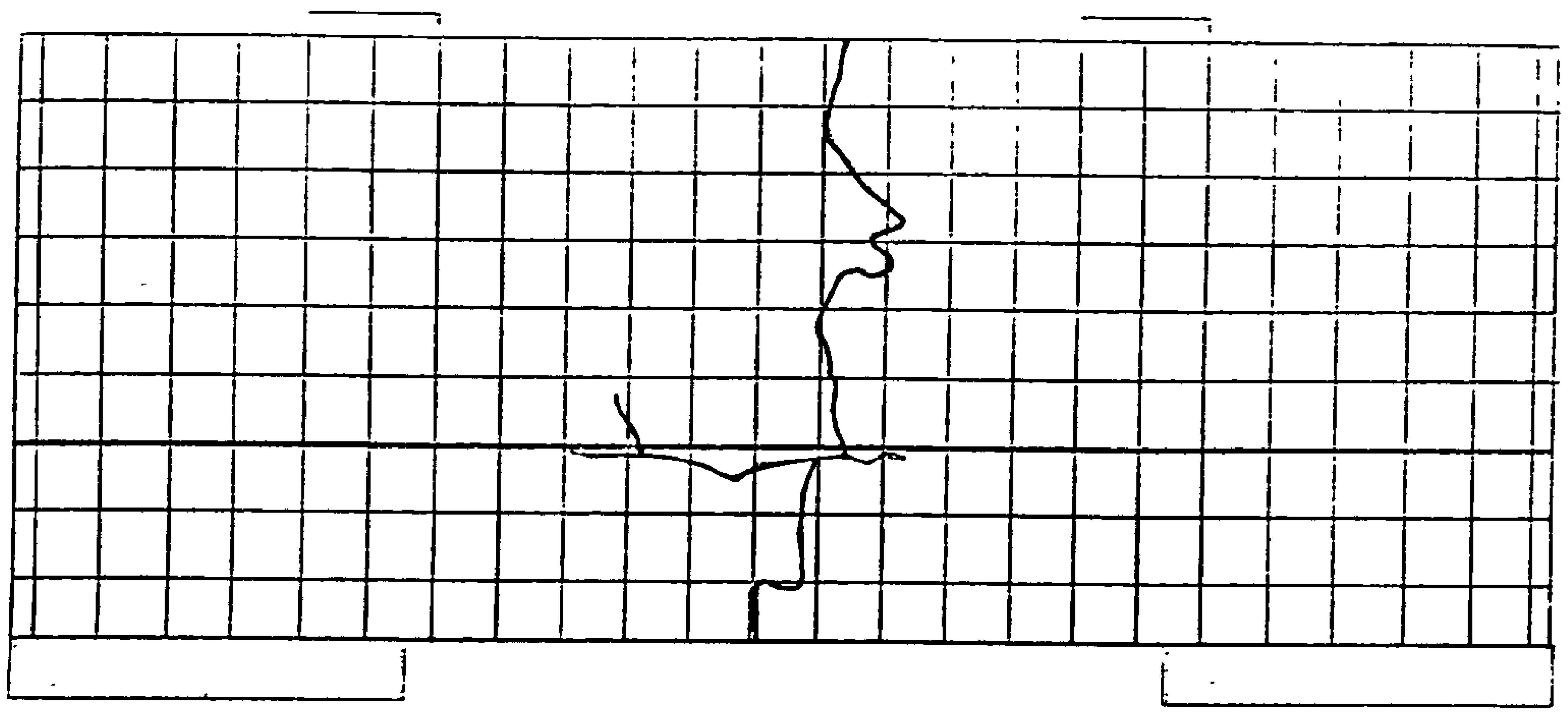
SPECIMEN:- F2
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



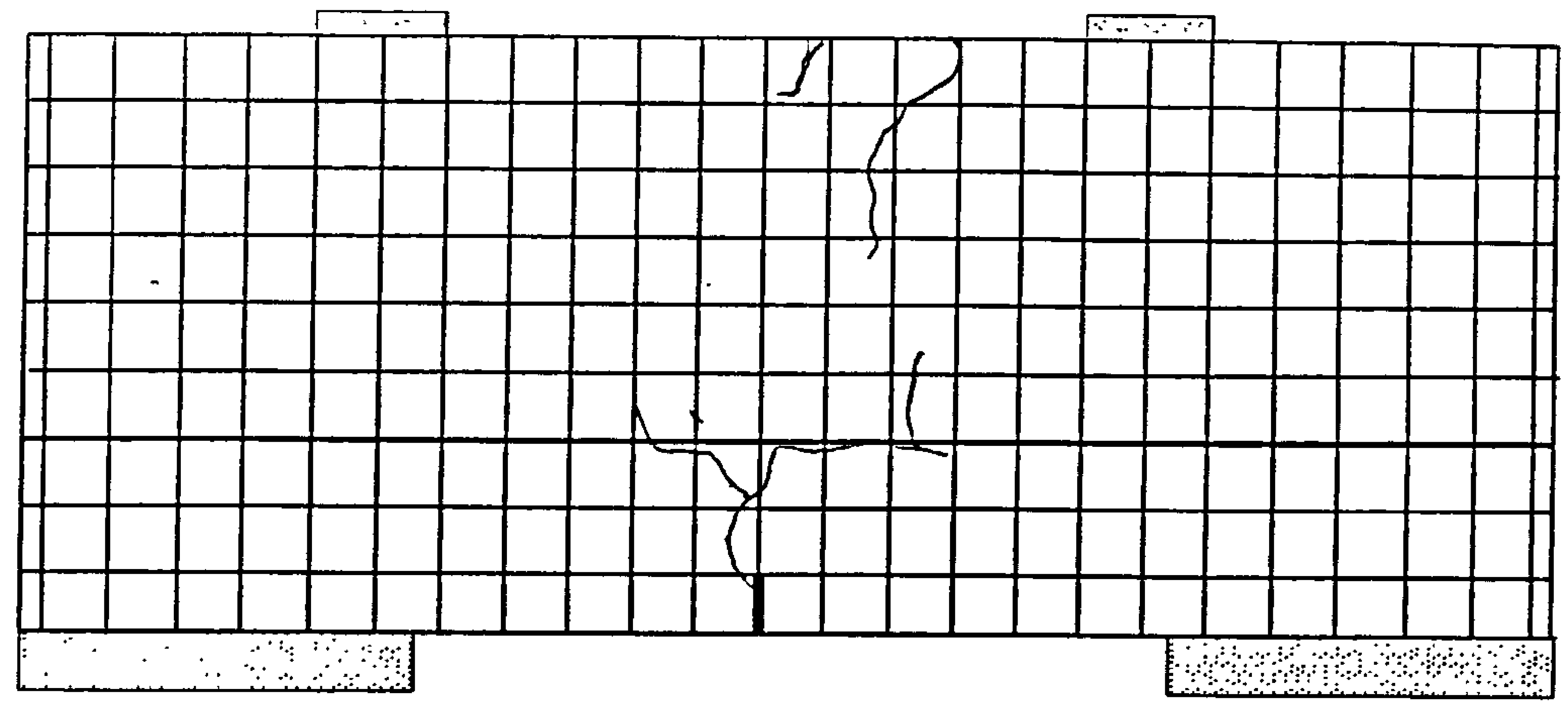
SPECIMEN:- GC1
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



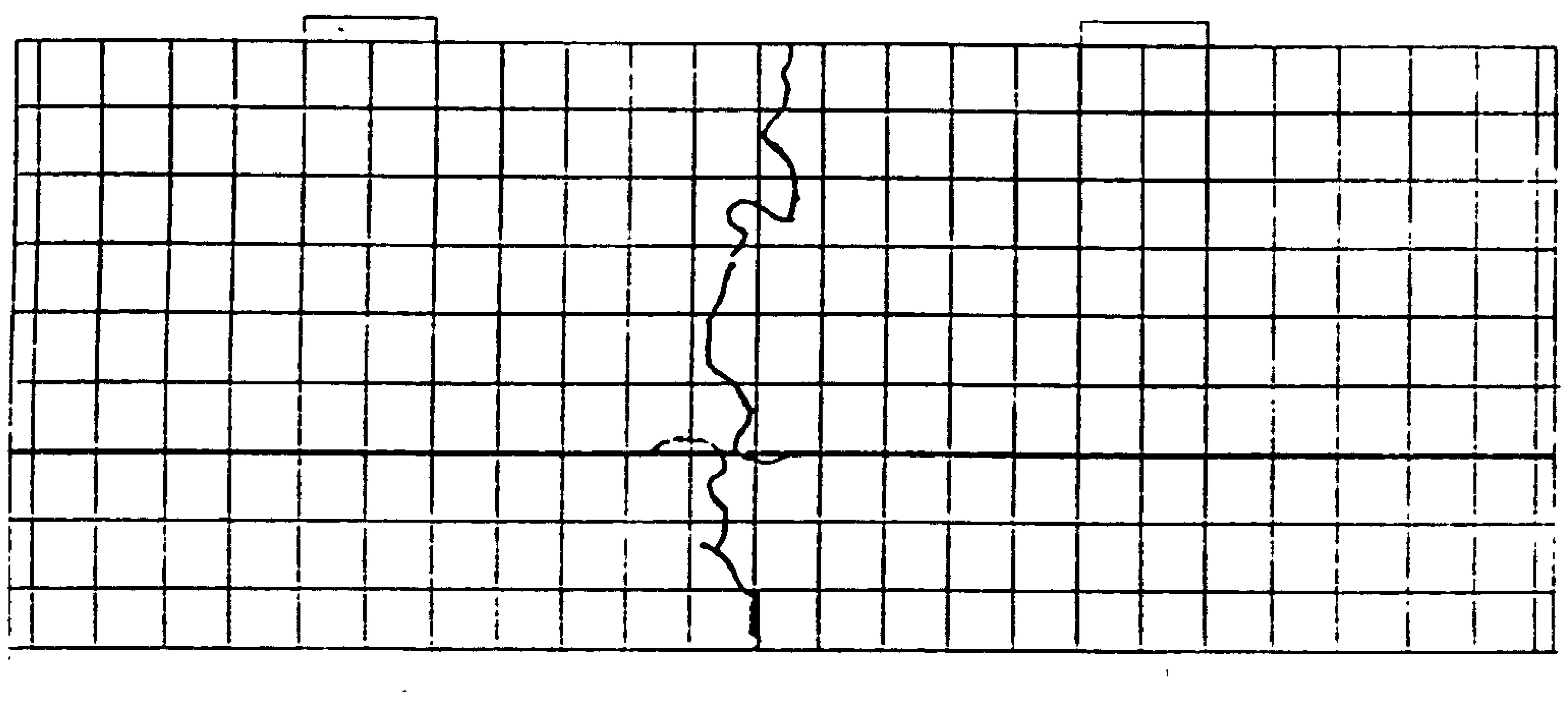
SPECIMEN:- S3
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



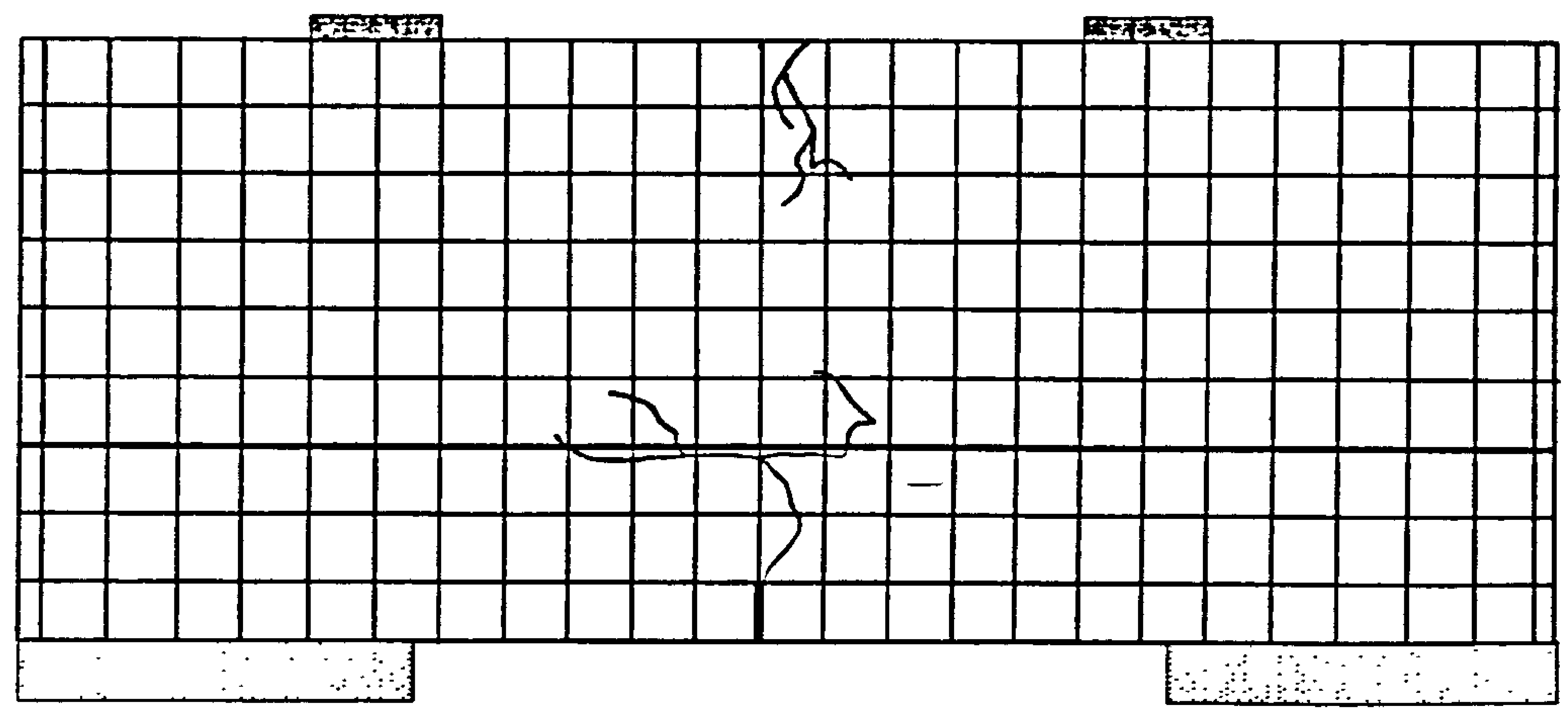
SPECIMEN:- F1
FRONT/~~BACK~~:-
TEST MODE:- Stress Control



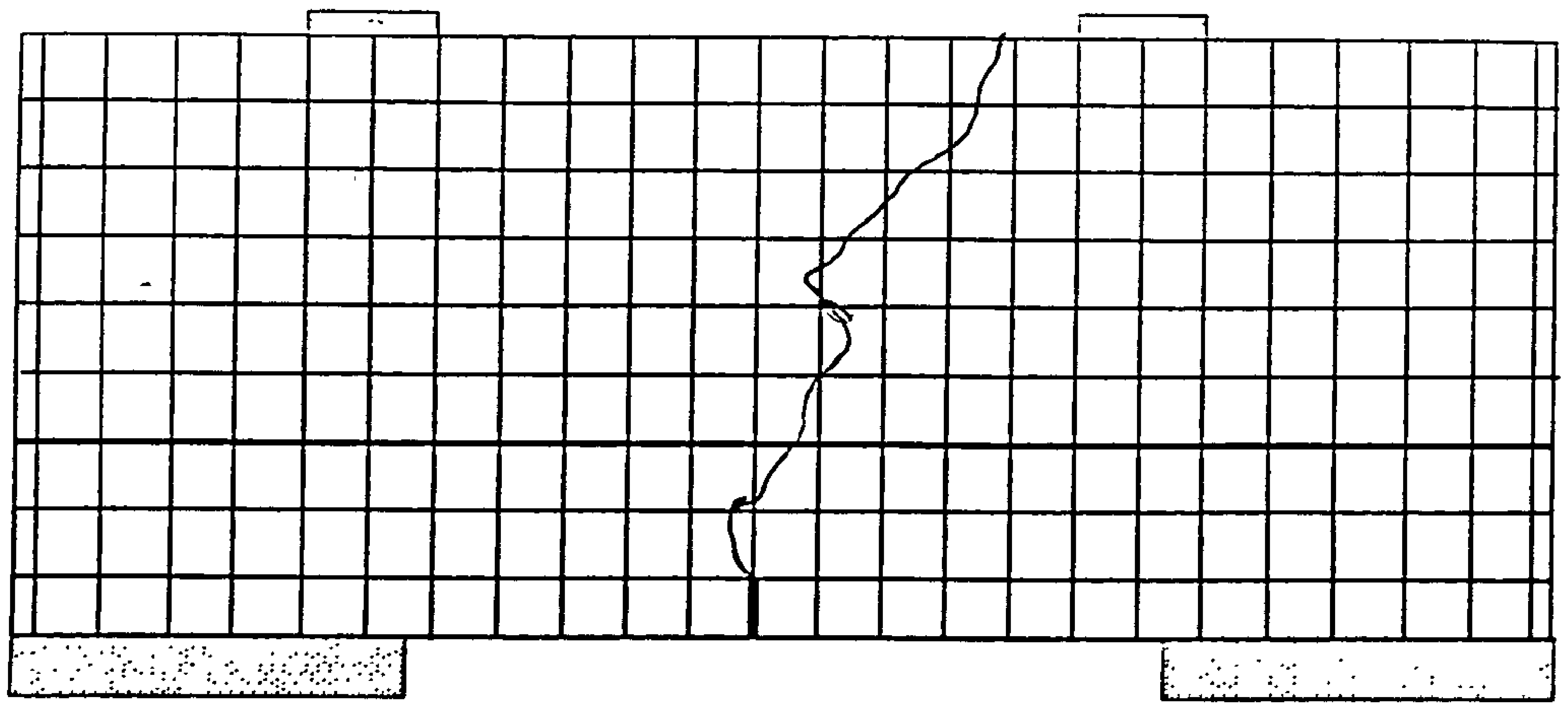
SPECIMEN:- S3
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



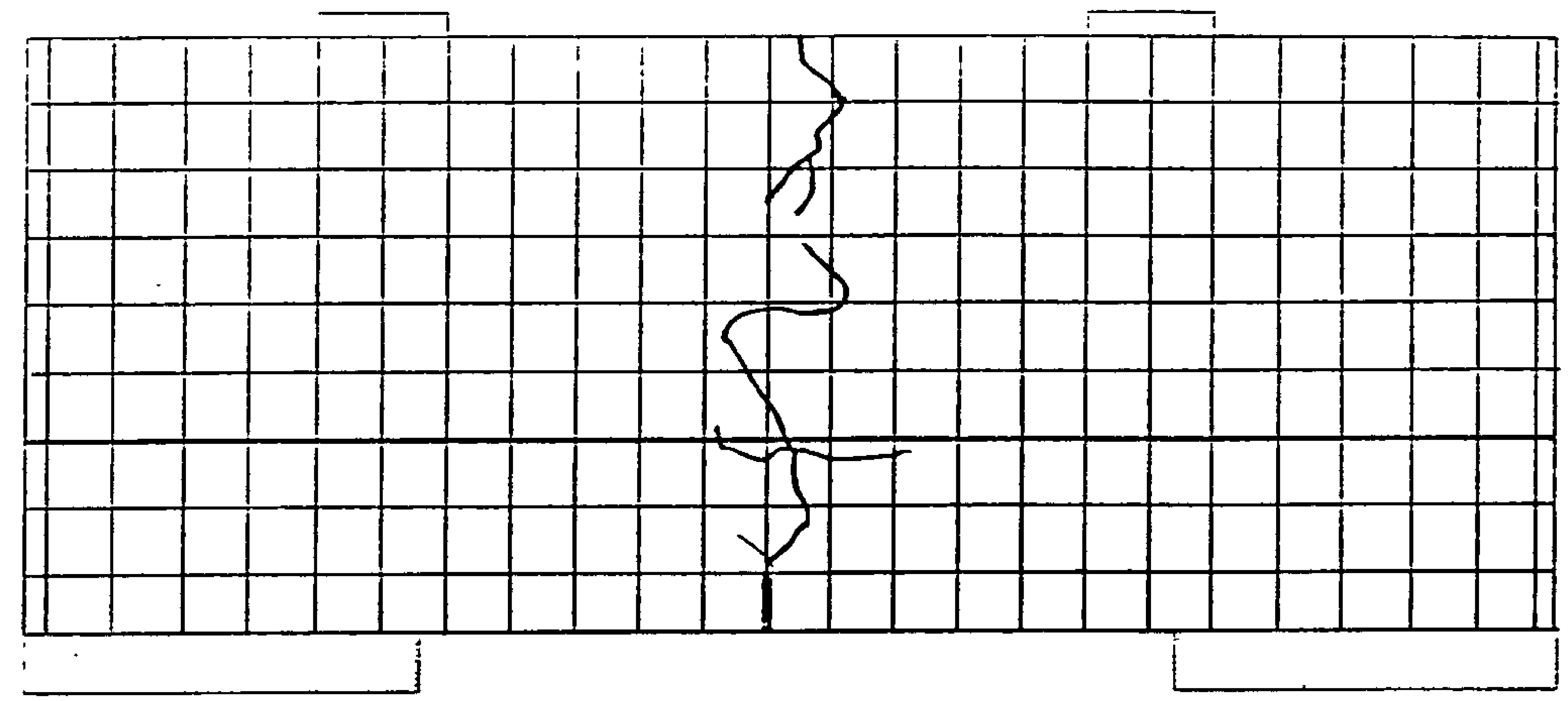
SPECIMEN:- F1
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



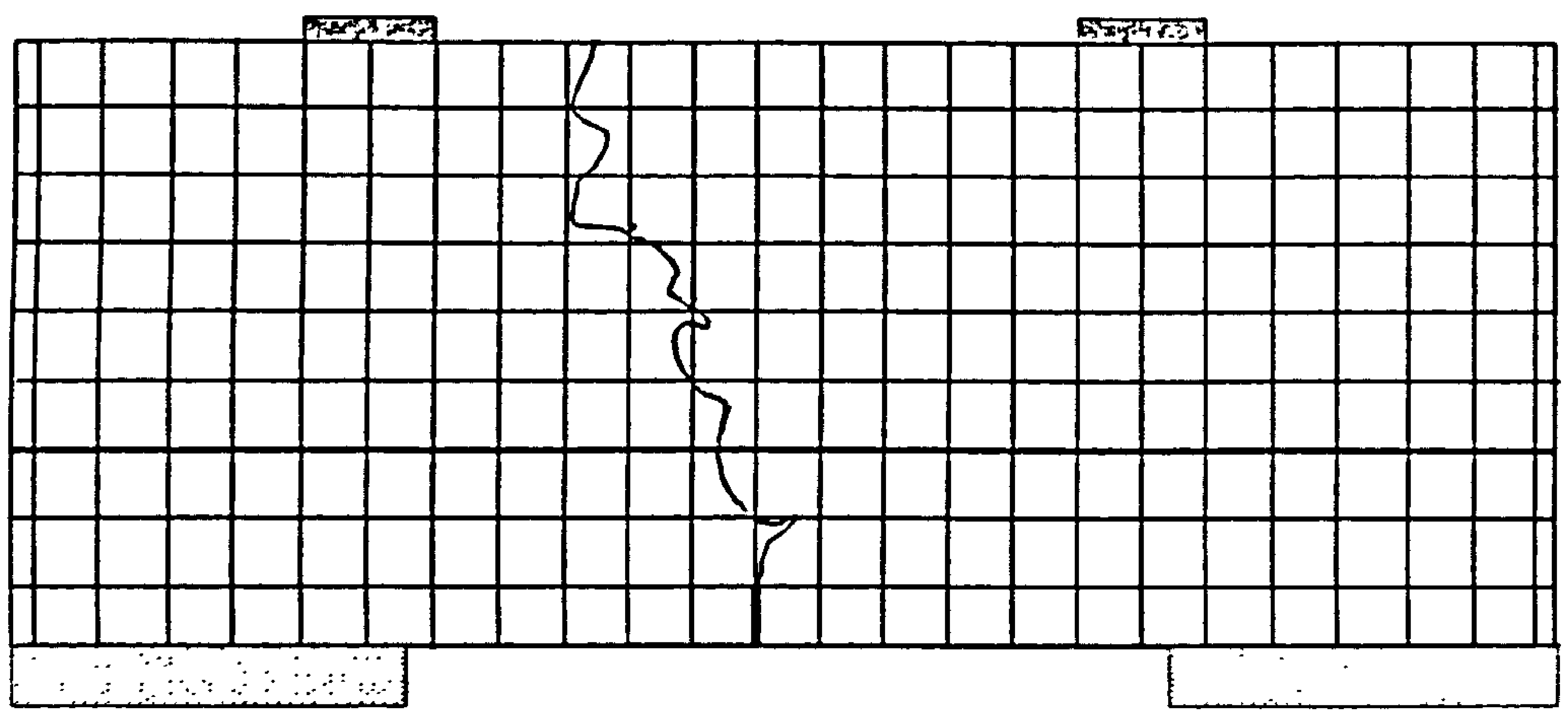
SPECIMEN:- S1
FRONT/~~BACK~~:-
TEST MODE:- Control



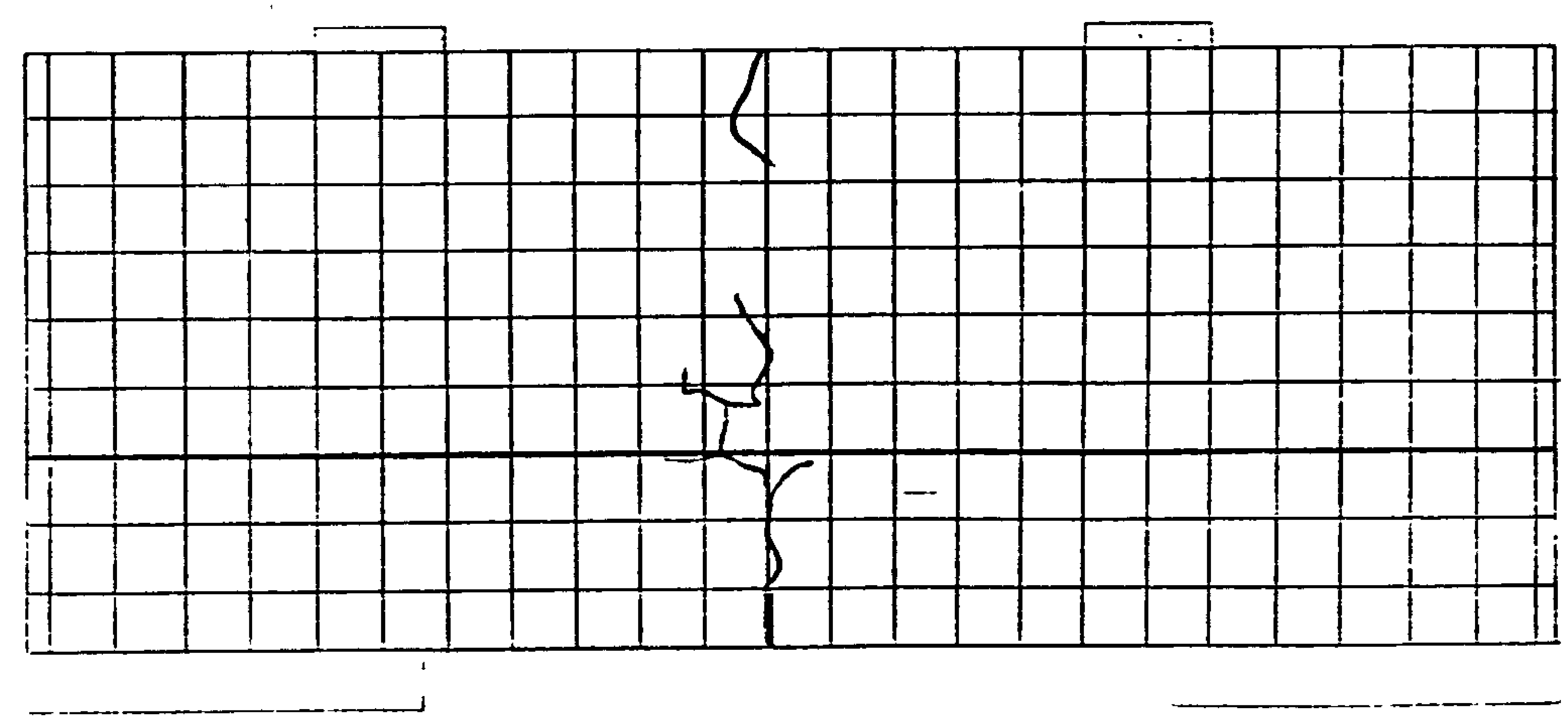
SPECIMEN:- S2
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



SPECIMEN:- S1
~~FRONT~~/BACK:-
TEST MODE:- Stress Control



SPECIMEN:- S2
FRONT/BACK:-
TEST MODE:- Stress Control



APPENDIX 7C
PHOTOS OF BEAM INTERFACES (POST TESTING)

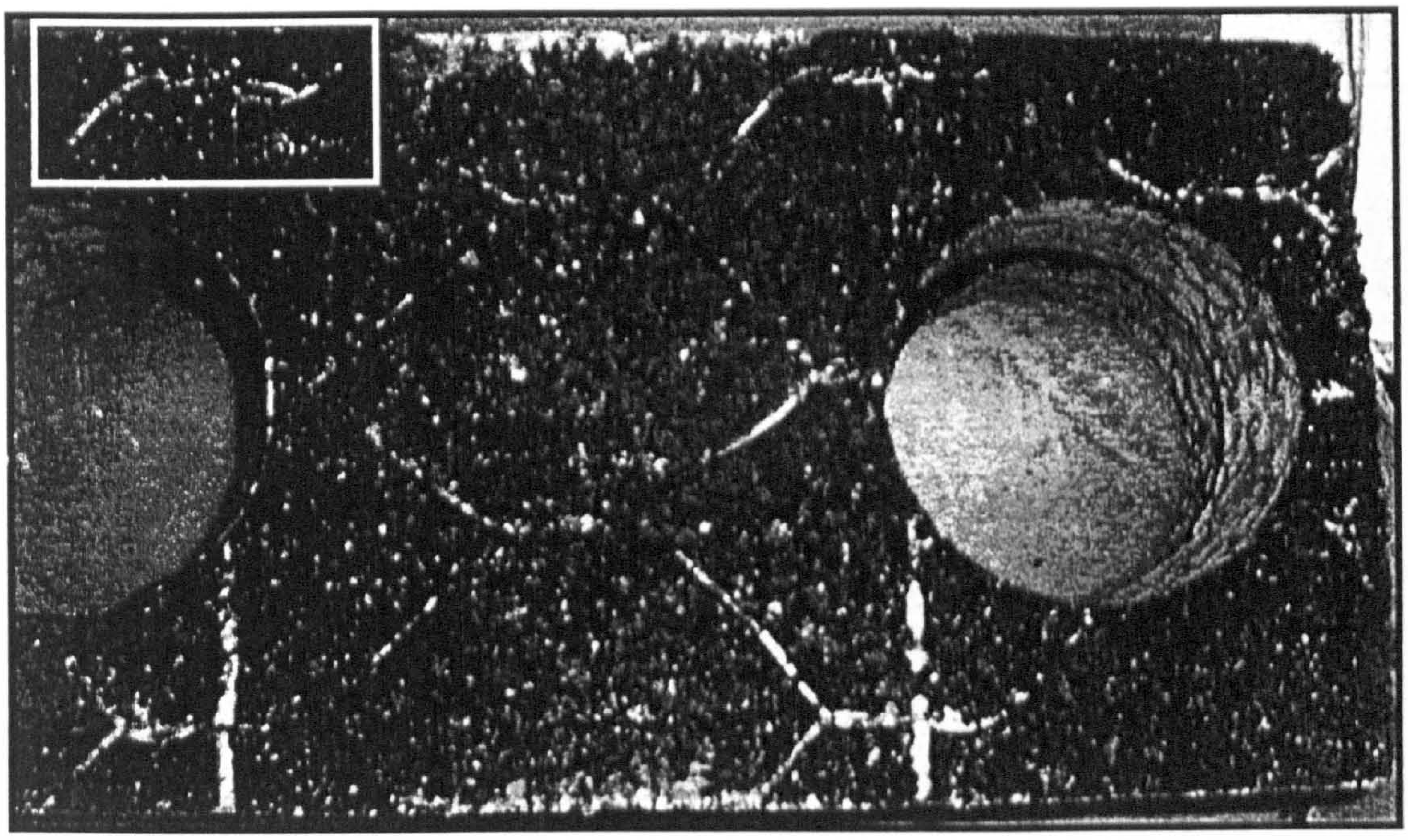


FIGURE 7C.1
POST-TESTING BEAM INTERFACE: BEAM S3 – UPPER LAYER

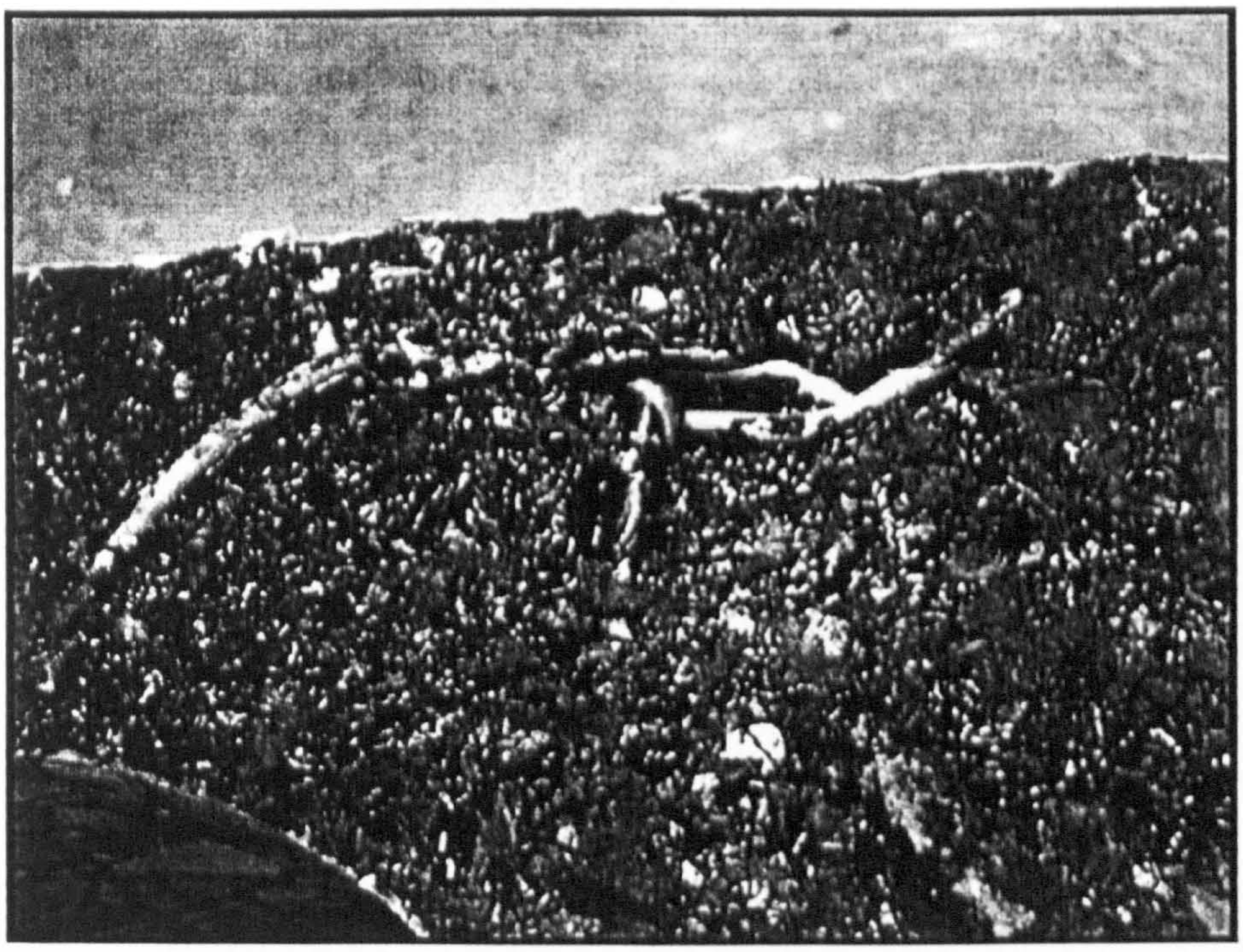


FIGURE 7C.2
POST-TESTING BEAM INTERFACE: BEAM S3 – VOIDS ADJACENT TO WIRE' NODE'

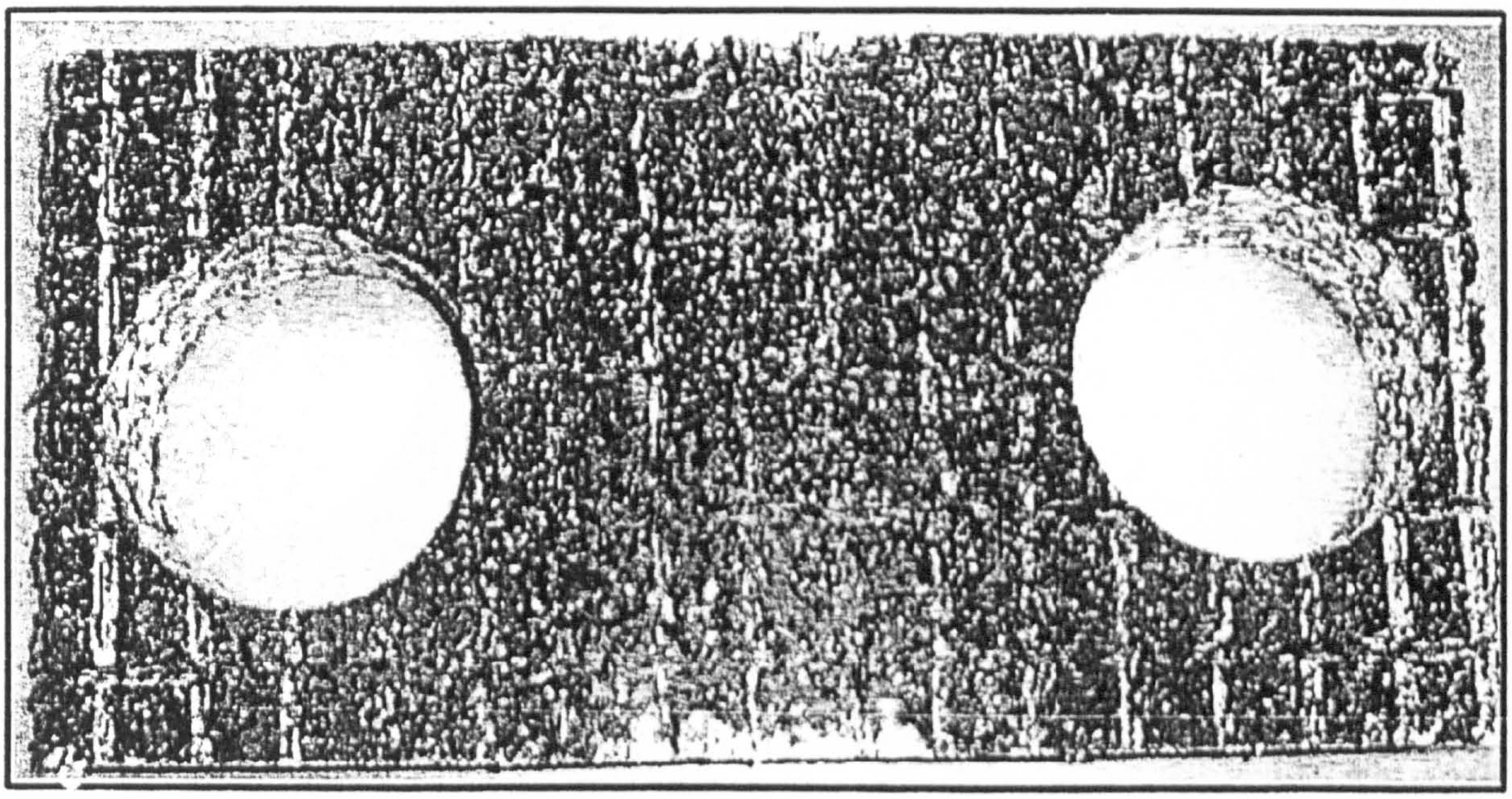


FIGURE 7C.3
POST-TESTING BEAM INTERFACE: BEAM GG2–
UPPER LAYER

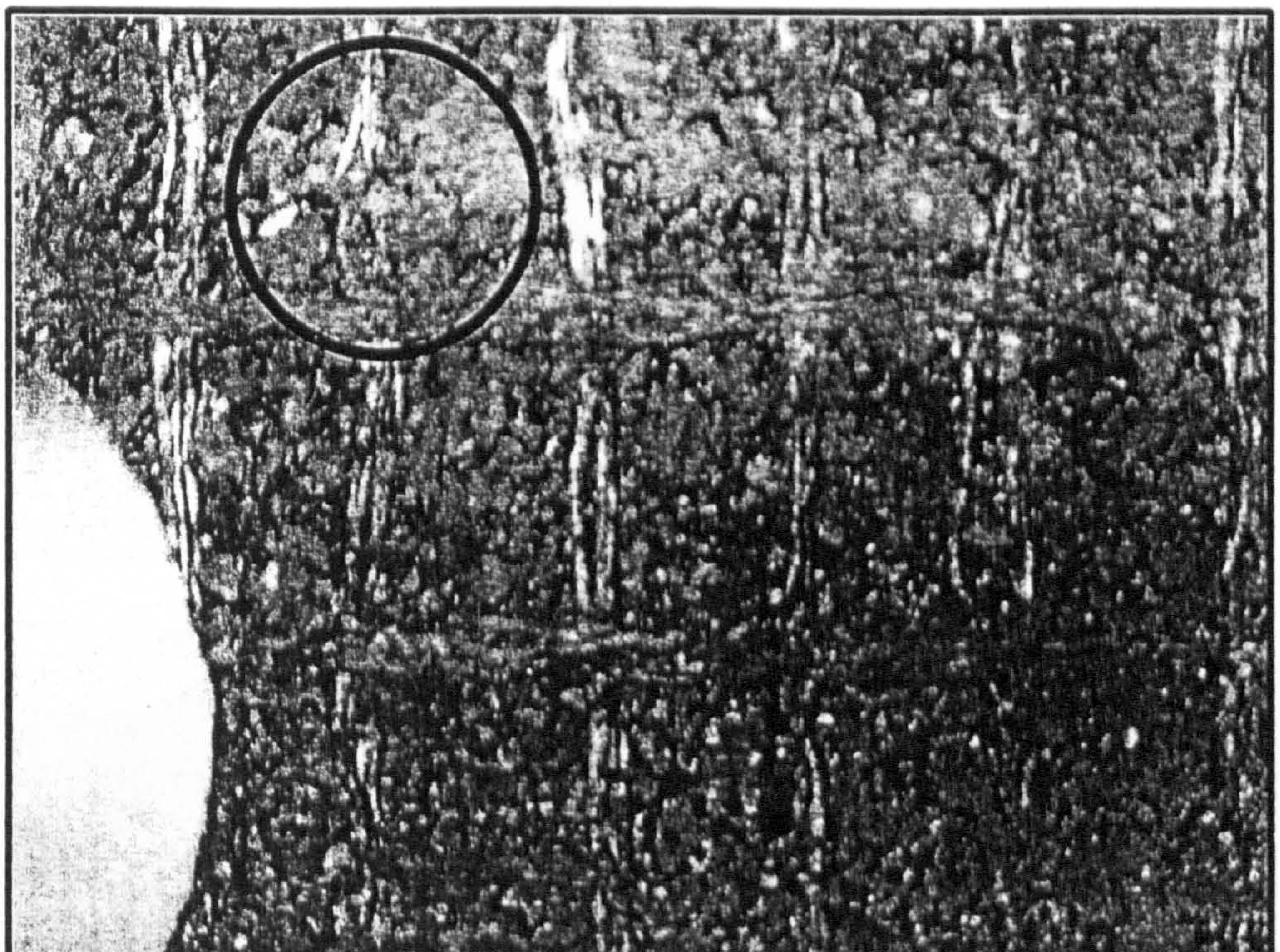


FIGURE 7C.4
POST-TESTING BEAM INTERFACE: BEAM GG2–
DETAIL OF BROKEN STRANDS

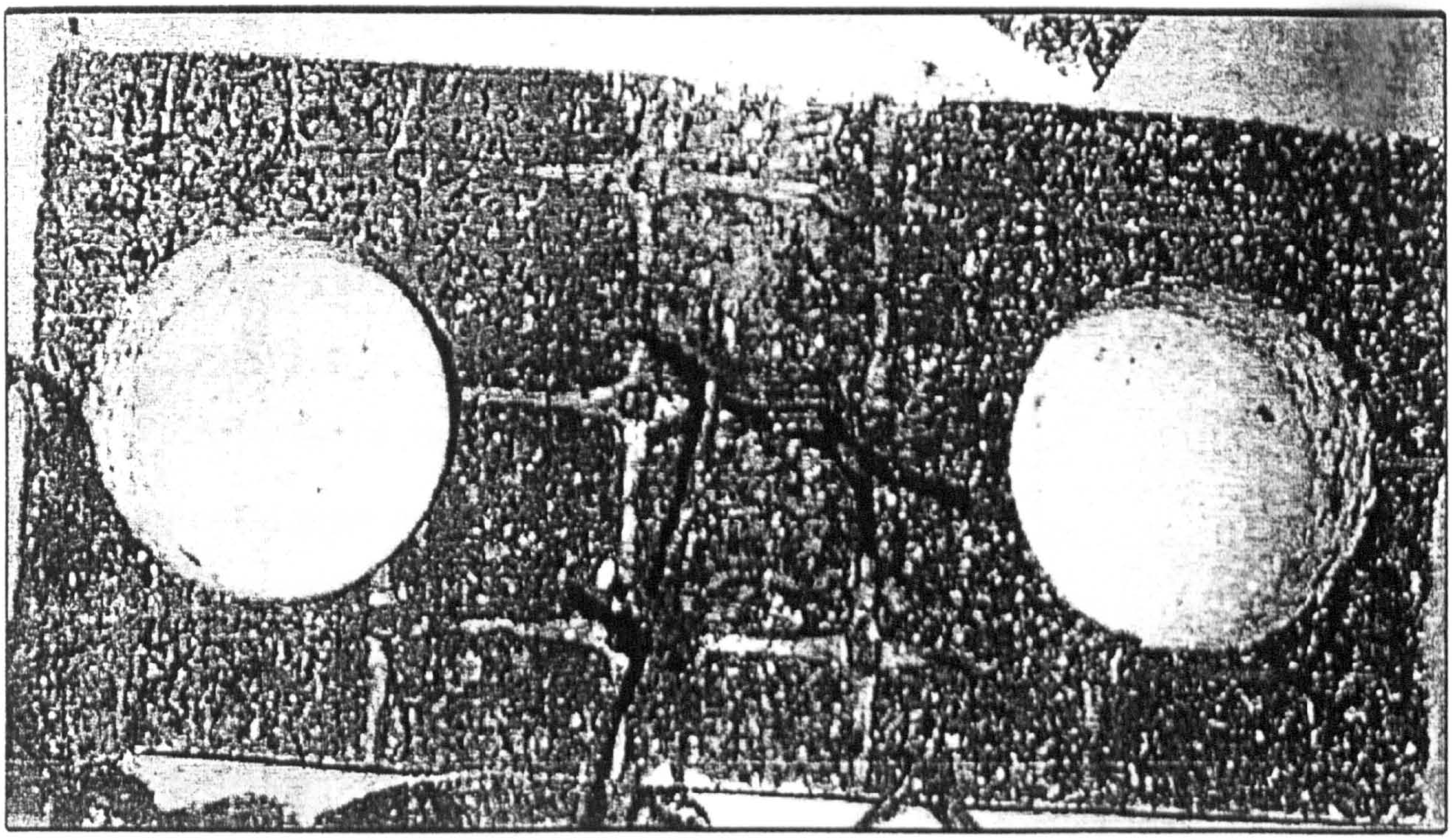


FIGURE 7C.5
POST-TESTING BEAM INTERFACE: BEAM PC2–
UPPER LAYER

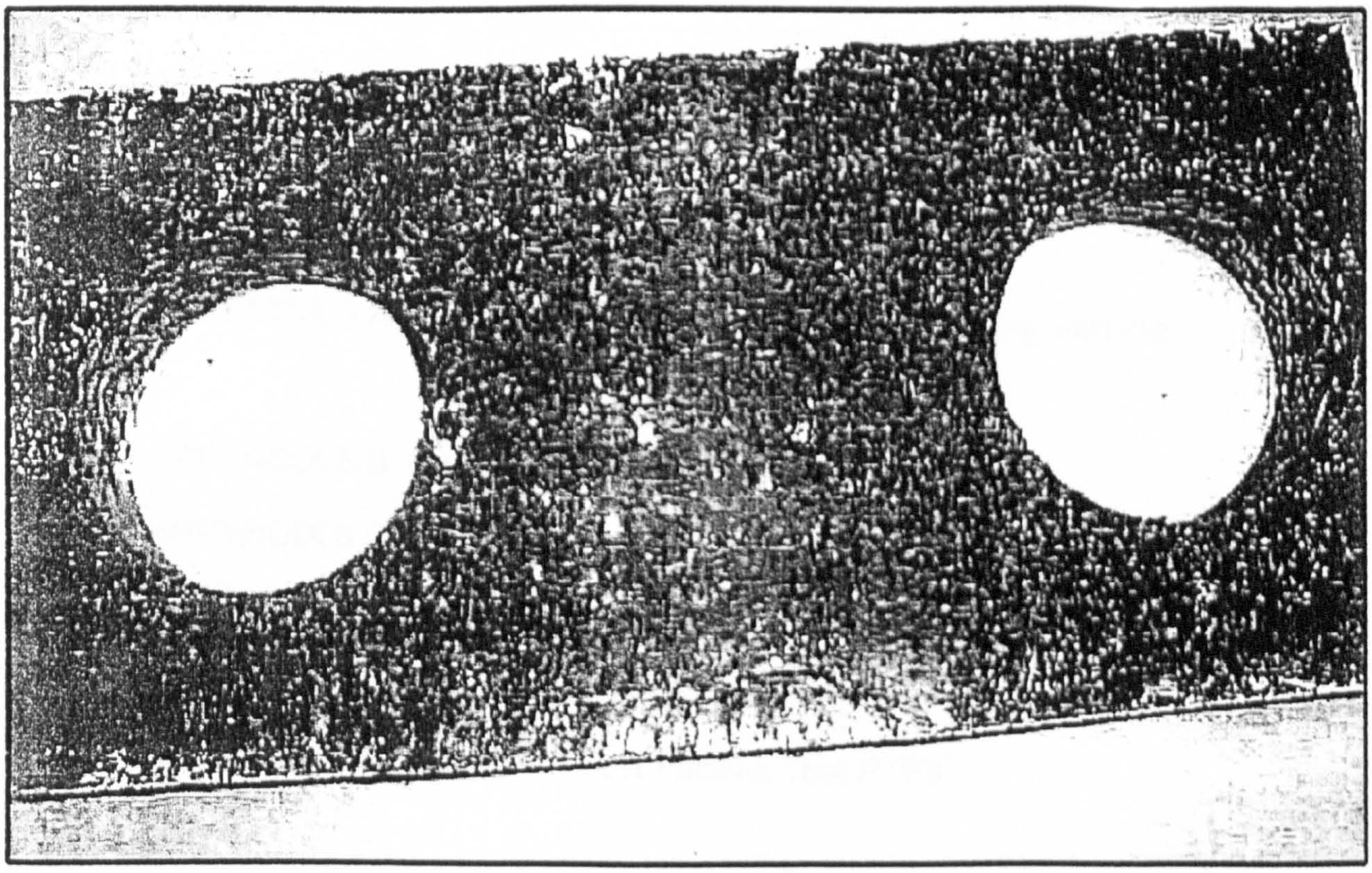


FIGURE 7C.6
POST-TESTING BEAM INTERFACE: BEAM F1

CHAPTER 8

PAVEMENT TEST FACILITY (PTF)

CONTENTS1

CHAPTER 8 - PAVEMENT TEST FACILITY (PTF)

8.1 Introduction

Whereas it is important to define the engineering characteristics of component materials through laboratory testing, it is difficult to estimate the performance of a reinforced pavement without testing a relatively large sample of material. To do this there are a number of options, including:

(1) Building a pavement and monitoring performance under real traffic,

(2) Building trial sections as part of a new road or as part of a maintenance treatment, and monitoring performance under real traffic.

(3) Building a trial section in the field for use with accelerated trafficking, or

(4) Building a trial section in a laboratory for use with accelerated loading.

Option (1) is an approach that should only be used if the consequences of unsatisfactory performance of a treatment or new construction type are not too costly, and/or there is great confidence in the product or techniques being used. In reality there is limited scope for this approach as the risks are often too great for a contractor or client to bear. Also, the time taken to failure (or at least for "meaningful results") can be prohibitive. This implies that either (a) if the treatment behaves poorly, valuable time is wasted in finding a more appropriate solution, or (b) if the treatment is successful, years of successful implementation on other road schemes could be wasted due to the time taken with "real" trafficking to obtain a reliable result. In addition, with "real" traffic and without accurate monitoring, it is difficult to know what traffic loading the test section has experienced, and therefore, reliable analysis of performance is not straightforward. In addition, as with any field trial, the effect of the environment, (especially temperature, moisture and elements that "age" bituminous materials), is difficult to assess and almost impossible to control. This option was obviously not viable for this investigation.

Option (2) has been used on many occasions, often in situations where the consequences of failure are small such as on secondary roads where the consequences of poor performance are small and/or alternative routes exist. As for option (1), the problem of knowing traffic loading and the environmental effects presents problems with analysis of test results. This option was considered, but through logistical challenges and time-related problems (i.e. the relatively short duration of the project), this option was also discounted.

The common feature of Options (3) and (4) is the accelerated wheel loading applied to test sections, whether they be *in situ* or in a laboratory. This has been used for pavement evaluation with good effect for 40 years or more since the ASSHTO trials were carried out in America. The main benefit of accelerated testing of pavements is that, before large sums of money are spent on building or maintaining pavements, the resistance of the structure or treatment to wheel loading can be assessed within weeks or months. This can save an agency's funds and benefit the public through better use of budgets and reduced maintenance (and hence user costs). With normal trafficking the time taken for the assessment of new treatments could be several years, by which time, (if the construction type or treatment was not suitable and was used for long lengths of pavement), a great deal of money and material resources may have been wasted. Table 8.1 summarises the main advantages and disadvantages of accelerated testing.

Table 8.1 Factors to Consider with Accelerated Testing

Factor	Advantage	Disadvantage
Traffic	Controlled	Could be channeled and unrealistic
Environment	Controlled (if in a laboratory)	The variations in field conditions might be the controlling factor in pavement behaviour
Construction	Large scale (realistic)	Can be difficult to control density and layer thicknesses accurately
Cost	In the long-term well-planned experiments can save expenditure by more cost effective pavement designs	Large scale testing is expensive, and experiments that are not well-planned and carried out properly may not be economical

Option (3) has been used with great effect in various countries, and particularly in South Africa [8.1], where Heavy Vehicle Simulator (HVS) test machines have been used to determine the performance of different pavement structures and maintenance treatments ranging from block paving and cemented materials to bituminous and granular-base pavements [8.2, 8.3 and 8.4]. The HVS is a 60-ton mobile machine designed to test existing roads. The device and associated instrumentation (for measurement of deflections at various layer interfaces) is particularly useful for the measurement of the effectiveness of different rehabilitation options. Due to the unavailibility of these machines in Britain, and the high cost of testing, this option was also discounted.

Option (4) (Accelerated testing in a laboratory), was chosen largely because the apparatus (the Pavement Test Facility - PTF) had been used for similar testing before [8.5], was affordable and available. The PTF is an accelerated testing facility capable of applying wheel loads of up to 15kN over a 2.2 x 8m section of pavement up to speeds of 16km/h. The test pit is 1.4m deep and contains a clay subgrade of approximately 1.1m over which the rest of the pavement structure was built. The facility is capable of applying unidirectional or bidirectional wheel loading which can be channelled or distributed laterally using electronic control. Test sections can be instrumented to enable transient stresses and deflections or strains to be measured, and at selected intervals during a test, measurements of surface deformation may be taken. A more detailed description of the apparatus has been given by Brown and Brodrick [8.6], and it is shown schematically in Figure 8.1.

8.2 Design of PTF Test Structures

The primary objective of the PTF tests was to measure the effect of different interface materials on reflective crack propagation. It was therefore necessary to consider ways of inducing regular reflection cracking to enable testing of different interface materials to be carried out similarly, and at the same time, to avoid excessive permanent deformation. This was necessary, as under normal conditions, and particularly with temperatures of 20°C and above, a relatively slow speed of loading and high contact stresses tends to induce large amounts of permanent deformation. Earlier PTF tests [8.5] on reinforced asphalt showed this to be the case.

Therefore, to crack the asphalt, three main possibilities were considered i.e. (a) placing a bituminous layer and cracking it when cold, before overlaying it with reinforced asphalt, (b) placing reinforced asphalt on concrete panels laid on the sub-base over the full width of the PTF test bed, and (c) overlaying square concrete paving slabs placed on the sub-base.

Option (a) was discounted on the basis that it would be difficult to crack the bituminous layer with sufficient precision to place instrumentation in appropriate positions. Also, it was felt it would be difficult to create regular cracks whose faces would have similar roughness and hence interlock which could affect the interpretation of test results.

Option (b) was discounted because it was not possible to find regular precast concrete panels of suitable dimensions, and it would have been too time-consuming and expensive to construct panels by hand.

Option (c) was adopted for five main reasons;
(1) The 600 x 600 x 60mm concrete slabs would potentially cause both longitudinal and transverse cracking (which would not have been the case with Option (b)).
(2) With regular-shaped slabs, the reflection cracks were likely to occur at well-defined locations, hence allowing easier positioning of instrumentation.
(3) The dimensions of the slabs suited the PTF test bed dimensions,
(4) Placing of the slabs was potentially simple and quick, and
(5) Slabs were both affordable and available.

Accordingly, all PTF test structures were constructed on paving slabs.

Notwithstanding the considerations taken during the design of the test structures, excessive deformation occurred in some test sections in PTF2, (see Section 8.5) which made the interpretation of test results difficult, due to lateral deformation of the asphalt partially masking cracks.

For PTF test 3, therefore, a different test configuration was required to reduce the effect of permanent deformation and to increase the incidence and speed of reflective cracking. Earlier PTF tests [8.5] on reinforced asphalt showed that reinforced sections reduced rutting by around 60% as compared to "control" sections and also that a soft pavement support helped to encourage the development of "bottom-up" cracks under wheel loading.

Taking the above into account, a simple modelling exercise using multi-layer linear elastic theory and a short experimental investigation was carried out (see Appendix 8.A) to investigate the possibility of using rubber sheeting within the test structure.

The simple modelling exercise was carried out using the computer program ELSYM5 [8.7] with details of the structure modelled as given in Table 8.2. Although it is noted that accurate modelling of the jointed slabs is beyond the capabilities of Multi-layer Linear Elastic programs, it was felt that ELSYM5 would indicate whether or not the presence of a rubber sheet could make a significant difference to deflections.

Table 8.2 Structure used for modelling the PTF

Layer	Thickness	Stifffness	Poisson"s Ratio
Asphalt	60	1300	0.35
Paving Slab	60	300[1]	0.15
Rubber sheeting	0, 5 & 15	2	0.5
Sub-base	150	150	0.15
Subgrade	-	50	0.4

Note 1 The value of stiffness takes into account the effects of gaps between individual slabs.

A single load of 12kN, and 400kPa was applied to the structure. Tensile strain at the bottom of the asphalt and surface deflections were calculated, as shown in Table 8.3.

Table 8.3. Results of modelling the PTF structure

Rubber thickness (mm)	Asphalt tensile strain (microstrain)
Zero	203
5	257
15	275

Although the thicker sheet gave higher predictions of tensile strain, it was judged that the additional cost of the 15mm sheet over and above the 5mm sheet outweighed the possible benefit of a potential increase in tensile strain (and hence quicker cracking).

It was anticipated that the soft layer beneath the rigid slabs would help to concentrate movement at the joints (and therefore induce reflective cracking quicker), and thus reduce the opportunity for rutting to occur. The general structure adopted for tests PTF2 and PTF3 is shown in Figure 8.2. In addition to the inclusion of layers of rubber sheeting and grout, the test temperature of test PTF3 was reduced to 13°C to help reduce permanent deformation.

Testing asphalt with two types of grid, two types of composite reinforcement and an unreinforced pavement over the entire test pit would have meant five tests which, (taking the time for excavation and construction into account) would have excessively lengthened the test program. The test area was therefore was divided into six smaller sections, as shown in Figure 8.3, allowing three sections to be tested simultaneously, and, by including control sections, making direct comparison with reinforced sections possible.

8.3 Test Program and Construction

Test Program

Test pavement PTF1 was primarily used as a construction trial to ensure that layer thicknesses were adequate for placing grids, and thus that subsequent tests could be constructed and trafficked properly. After trafficking PTF1, Test pavements PTF2 and PTF3 were then constructed with six test sections each. Over the two test pavements, two sections incorporating each type of reinforcement (and four unreinforced sections) were built and tested. Test PTF3 was carried out using the same combination of grids, composites and control sections as PTF2, but with the addition of a 5mm grout screed and a 5mm rubber sheet between the paving slabs and the sub-base to provide a more uniform construction surface and support for the slabs.

The final test program is given in Table 8.4.

Table 8.4. PTF Test Program

PTF test	Test Date
1	November 1997
2	February-March 1998
3	July –October 1998

8.3.1 Construction

The two test sections of PTF1 were built as an addition to an existing bridge joint test. The construction consisted of one control section and the other, reinforced with a Tensar AR1 grid, as shown in plan in Figure 8.4.

Layer thicknesses of all Pavement Structures are given in Table 8.5.

Table 8.5 PTF Layer Thicknesses.

Construction	PTF1	PTF2	PTF3
Top asphalt	35	35	35
Lower asphalt	25	25	25
Paving slab	None	60	60
Sand	None	10	10
Rubber sheet	None	None	5
Grout Screed	None	None	5
Sub-base	180	175	175
Subgrade	1000	1000	1000

The instrumentation installed to measure relative (vertical) slab movement, vertical pressure in the subgrade, and surface strains are described in Appendix 8.B.

8.3.2 Construction procedure

Initially, after the trial PTF1 test was completed, the existing test structure was removed to expose the clay subgrade. The clay was then trimmed, and an equilibrium moisture content established by covering the test pit with a plastic sheet. Before the pavement structure was constructed, the condition of the subgrade was assessed using the Dynamic Cone Penetrometer (DCP) and a soil penetrometer. A description of these tests is given in Appendix A8.C

180mm of Type 1 sub-base was then placed in three layers and compacted with a vibrating pedestrian roller. Density readings were taken in the sub-base using a nuclear density meter (see Appendix 8.C) and are given in Table 8.6.

Table 8.6 Nuclear Density Readings on PTF2 Sub-base

Test Position (paving slab number-see Figure 8.3)	**Reinforcement type**	**Test Reading** (kg/m^3)	**Moisture Content** (%)	**Corrected (dry) Reading** (kg/m^3)
6	AR-G	2260	5.8	2134
10	AR-G	2240	5.9	2113
14-18	Control	2291	6.1	2158
22	AR1	2302	5.9	2172
26	AR1	2313	6.4	2187
7	Rotaflex	2259	6.2	2127
11	Rotaflex	2270	6.0	2130
15-19	Control	2323	6.0	2190
23	Roadmesh	2345	5.7	2217
27	Roadmesh	2213	6.1	2179

A Clegg Hammer [8.8] was also used to measure sub-base consistency across the test section. Detailed results from this test are given in Appendix 8.C and a summary is shown in Figure 8.5. The plotted points show that densities fall within a fairly typical range found in the field (i.e. 2000-2200kg/m^3), and (b) that density is fairly consistent within the test area, although there is a tendency for densities to be lower in the composite reinforcement sections and higher in the grid-reinforced sections.

The construction sequence for the test pavement PTF3 was as follows:

Approximately 175mm of Type 1 sub-base was first placed and compacted. Quality control was achieved using the Clegg Hammer, and taking density readings with a nuclear density meter. At this stage of construction, tests were carried out to investigate whether the inclusion of a rubber sheet under the concrete slabs would actually make a significant difference to deflections. To do this, slab and plate-jacking tests were carried out using the PTF as a reaction frame. These tests are described in Appendix 8.A and show that the rubber sheet was expected to increase slab deflections significantly, hence increasing the speed of cracking and thus reducing the opportunity for rutting to occur.

After the sub-base was compacted and tested, a 5mm cement grout screed was placed to provide a uniform surface on which to place the rubber sheet. Concrete slabs with dimensions of 600 x 600 x 60mm were then placed, followed by a 25mm layer of 14mm

DBM which was left overnight to cool. The next day, grids and composite materials were applied following the manufacturers' recommendations, before laying 35mm of 14mm DBM surfacing.

Instrumentation
A summary of the instrumentation used is given in Table 8.7.

Table 8.7. PTF Instrumentation

Instrumentation device	Property measured
LVDT	Vertical deflection measurement
Strain coils	Lateral movement of concrete slabs
Nottingham pressure cell	Vertical earth pressure
Demec Gauge	Lateral asphalt surface movement (and hence strain).

To measure relative slab movements, rutting, surface asphalt strain and vertical pressure on the subgrade, LVDTs, strain coils, a rut profiler, DEMEC gauge and Nottingham pressure cells were used. These instruments are described in Appendix 8.B.

Vertical slab movements were measured using LVDTs mounted on one side of a slab joint and measuring the displacement of a "target" on the other as shown in Figure 8.6.

Strain coils were mounted on the concrete slabs to measure horizontal slab displacements, but did not operate properly due to their proximity to the steel in the PTF wheel.

Rutting was measured using a profile template attached to a datum bar as shown in Figure 8.7. The shape and magnitude of the rut across the wheelpath was measured and traced onto paper from which maximum values have been taken.

An attempt was made to investigate if surface cracking could be predicted through the measurement of asphalt surface strains using a DEMEC gauge and "pips". Figure 8.8 shows the general arrangement of the DEMEC pips and LVDTs over the position of a "typical" joint, and Figure 8.9 gives a schematic representation of the DEMEC gauge with the LVDT. The gauge had to be fitted with an LVDT to take the place of a dial gauge to measure the dynamic output to be recorded "remotely" due to the proximity of the PTF wheel to the gauges. Although the principle adopted to measure surface strains was thought sensible, in practice it was found that (generally) cracks did not form between the Demec "pips" and strains in the asphalt were too low for accurate readings with the LVDT used.

If, in future surface strains are to be correlated with crack growth, additional positions should be selected and more appropriate monitoring equipment used

Two earth pressure cells were placed under the unreinforced (control) sections to measure changes in vertical stress in the subgrade. The cells are described in Appendix 8.B.

8.4 PTF Trafficking

8.4.1 General procedure

Except for test PTF1, trafficking of the PTF sections was carried out using a 12kN wheel load, which had a tyre inflated to 500kPa. PTF1 was tested with a 15kN wheel load as this was the load used for the bridge deck test located between the two pavement sections.

Before trafficking began, initial readings of transverse profile, strain coil measurements, earth pressure and relative deflection were carried out. Trafficking was then periodically interrupted (typically once per day) to take readings and carry out detailed inspections of the pavement.

From the readings, plots of deflection and permanent deformation were produced to record pavement performance.

Failure of the test sections was taken to have occurred when cracks on the surface started to become "active", i.e. when cracks in positions close to the joints in the concrete slabs could be seen (with the aid of a magnifying glass) to open and close as the wheel passed.

A summary of the main findings of the tests is now given, with more details of the tests provided in Appendices D, E and F.

8.4.2 Test PTF1

As earlier stated, this test was carried out primarily to test the feasibility of using concrete slabs to induce regular reflection cracks and to investigate whether relative deflection of the joint could be measured using a modification of existing apparatus.

Two paths were trafficked to investigate the effects of wheel loads adjacent to longitudinal joints, and across transverse joints. The positions of the wheelpaths are shown in Figure 8.4.

Details of the test is given in Appendix 8.D.

8.4.3 Overall findings from PTF Test 1

Reflection Cracking

Although what appeared to be reflection cracks were visible they were not as definite as were anticipated. This was partly attributed to the effects of permanent deformation, which, with the high wheel load, was considered excessive. The grid did appear to have some effect, however, and visually, the reinforced sections appeared less-cracked than the unreinforced sections. The deflection measurement procedure was considered cumbersome and awkward to interpret, and led to improved instrumentation for the following tests.

Permanent deformation

As mentioned above, overall, the level of permanent deformation was considered excessive and with the following tests (PTF2, 3 and 4) a lower wheel load was used. However, a distinct reduction in permanent deformation was noticed with the reinforced sections in both the transverse and longitudinal wheel tracks.

Instrumentation

It was considered that the approach used to measure of the vertical component of slab deflection was adequate, but an attempt to measure the horizontal component of slab movement should also be made. The possibility of using strain coils was therefore investigated in test PTF2.

Construction

Use of the paving slabs seemed to fulfil requirements, i.e. they were simple to install, did not break under relatively high PTF and compaction loads, and, most importantly, they seemed to help induce cracks through the asphalt.

The poor quality of the asphalt appeared to be due to the difficulty of placing relatively small amounts of the materials over small areas (i.e. either side of the bridge joint.

8.4.4 PTF tests 2 and 3

The overall objective of PTF tests 2 and 3 was to compare the performance of sections reinforced with different materials under wheel loading. To measure performance, the number of wheel passes taken before cracking was compared, as were measurements of permanent deformation and transient relative deflections. The following sections describe PTF tests 2 and 3, and are followed by an overall analysis of test results.

8.4.5 Test PTF2

General

During this test four reinforced pavement sections and two unreinforced ("control") sections were trafficked. The AR1, control and AR-G sections were the first to be trafficked, followed by the Rotaflex, Control and Roadmesh sections on the other side of the test pit. The wheel paths followed are shown in Figure 8.10. The test is described in more detail in Appendix 8.E.

The number of repetitions to "failure", i.e. active cracking, are given in Table 8.7.

Table 8.7. PTF 2 test results.

Test section	Number of 12kN repetitions to failure
AR1	55 000
AR-G	54 000
Control (AR1-AR-G wheelpath)	20 000
Rotaflex	20 000
Roadmesh	44 000
Control (ROAD MESH-ROTAFLEX wheelpath)	20 000

Figures 8.E-1 to 8.E-4 in Appendix 8.E show how changes in relative deflections developed with trafficking and Figure 8.11 shows the resultant crack patterns.

It is noted that the relative slab movements causing reflection cracking were not the same between the sections, and a simple comparison of the repetitions to cause failure was not thought to be a meaningful representation of the relative behaviour of the sections. A simple method of calculating a single equivalent deflection for each test was therefore used to help compare performance of the sections. This is now described.

Figure 8.12 indicates how the technique is used. The approximate area under the graph is calculated by summing the contribution of each reading i.e. $\Sigma(\delta_i \times N_i)$. Then, to derive an overall representative deflection, the multiple of average deflections and repetitions is divided by the sum of the repetitions -

$$\delta_{equivalent} = \left[\frac{\sum(\delta_i N_i)}{\sum N_i}\right]$$

AR1-Control-AR-G Wheelpath

Relative Deflections-trafficking

Representative deflections are plotted in Figure 8.13, and show how the AR1 and AR-G reinforced sections withstand around two-and-a-half times the loads of the unreinforced sections.

Cracking

Transverse cracking in theAR1-wheelpath was seen to occur in the control areas and particularly at the junction of AR1-reinforced and unreinforced areas. Longitudinal cracking was quite evenly spread over the entire length of the wheelpath and appeared to be linked to the incidence of "rut shoulders".

Permanent Deformation

Deformation measurements are shown in Figure 8.14 and show both the grid-reinforced and composite-reinforced sections to develop around 50% of the rutting of the control section. This is consistent with tests carried out in 1985 [8.1] with AR1 grids. The reason for the divergence of the AR1 and AR-G lines in Figure 8.14 cannot be properly explained, but is thought due to the nature of the interlayer bonding, i.e. grid interlock for the grid, and a mixture of interlock and bitumen adhesion with the AR-G composite. It is considered that the interlock of the AR1 and asphalt does not change significantly with load repetitions, whereas the lower interface of the AR-G reinforcement (which relies on a bitumen bond) may have degraded at around 46000 repetitions or so.

Rotaflex-Control-Roadmesh Wheelpath

Relative Deflections-trafficking

Similar to the performance of the AR1 and AR-G-reinforced sections, the Road-Mesh reinforced section took around twice as long to crack as did the unreinforced section. The glass-reinforced composite (Rotaflex) section, however, only carried a similar number of repetitions before cracking as the unreinforced section, although the Rotaflex section was subjected to larger relative deflections. If the larger deflections in the Rotaflex-reinforced section are taken into account by plotting normalised deflections versus traffic to cracking (Figure 8.13), the Rotaflex performance seems consistent with the performance of other reinforced sections.

Further evidence of the benefits of the presence of reinforcement in the asphalt is indicated by the general level of relative transverse deflection measured in the sections, as indicated in Table 8.8. It is evident that both reinforced asphalt sections were subjected to greater deflections than were the unreinforced sections, but cracking occurred at the same time or later than the unreinforced section. Table 8.8 also shows how relative deflections in reinforced sections increased at a slower rate than deflections in unreinforced sections.

Table 8.8 Relative Deflections Measured Across Transverse Joints: PTF2

Load Repetitions	Roadmesh-reinforced section		Rotaflex-reinforced section		Unreinforced section
A	B(mm)	Ratio C=B/F	D(mm)	Ratio E=D/F	F (mm)
0	0.528	1.27	0.775	1.86	0.416
5 000	1.096	1.22	1.570	1.74	0.900
10 000	1.430	1.14	1.800	1.44	1.250
20 000	1.570	1.31	1.960	1.63	1.200
40 000	1.540	1.03	1.480	0.99	1.500

Cracking

Both transverse and longitudinal cracking was encountered in each section and as for the other wheelpath, longitudinal cracking was almost entirely found on rut shoulders. Transverse cracks were found in each section and although shorter than in the other wheelpath, particularly distinct in the unreinforced section and the section reinforced with Rotaflex.

Permanent Deformation

The relationship between permanent deformation and wheel repetitions measured during the test is shown in Figure 8.14. First of all it is noted that permanent deformation measured on the Rotaflex section was considerably higher than that for the Roadmesh section and, up to 30000 repetitions, not greatly different to that of the unreinforced section. Also, the unreinforced sections for both wheeltracks show quite similar permanent deformation behaviour whereas both sections reinforced with grids and the AR-G composite, showed considerably less rutting than the Rotaflex-reinforced section.

A reason for the different performance of reinforcement materials with regard rutting was sought, and as significant amounts of horizontal movement, i.e. 'shoving', was evident from the shape of the rutted profile, a link with the interlayer bond was thought likely. Also, the significant amount of horizontal deformation was thought to be partly due to the asphalt being placed on rigid concrete slabs, which help prevent vertical deformation occurring.

A link between the 'roughness' (or profile) of reinforcement and the size of the grid apertures with rutting was suspected. Accordingly, these properties were summarised as seen in Table 8.9.

Table 8.9. Details of reinforcement materials used in PTF2.

Material	Aperture size (mm)	Ratio of Aperture size to maximum stone size	Depth of grid profile (mm)
AR-1	65 x 65	4.6	4.4 (node) and 0.8 (rib)
AR-G	65 x 65	4.6	4.4 (node) and 0.8 (rib)
Roadmesh	80[1]	5.7	6 (at junction of strands)
Rotaflex	40 x 26	2.9 and 1.86	1.5 (rib)

Note 1. This dimension is taken across the middle of the hexagon mesh, i.e. at the narrowest position.

The profile of the grids (and the grid part of the composite materials) is seen to be quite different, with Rotaflex having a much shallower profile. Figures 8.15 and 8.16 show the relationship between rutting and aperture size, and rutting and grid profile and the trend of increasing profile depth with reducing rut. It is noted that in Figure 8.16 the lowest rut depth does not correspond to the largest grid profile. This could be due to the different type of reinforcement materials and the way that the grids are connected. In particular, the differences between the twisted wires of the Road-Mesh and the fully bonded polypropylene strands could have an effect on the compaction of the bituminous material, and thus on it's resistance to deformation. Measurements of density given in Table 8.10 may suggest that Road-Mesh tends to impede compaction in the top layer of material, possibly due to the relatively large nodes, whereas the large apertures facilitate additional compaction of the bottom layer during compaction of the top layer.

Table 8.10. Densities of Bituminous Material Taken from PTF2 (expressed as percent of target density)

Type of Reinforcement[1]	Top Layer	Type of Reinforcement[2]	Bottom layer
Road-Mesh	96	Road-Mesh	93
CONTROL	93	Road-Mesh	92
CONTROL	93	Rotaflex	90
Rotaflex	92	Road-Mesh	90
Road-Mesh	91	Road-Mesh	90
AR1	91	AR1	90
CONTROL	91	Rotaflex	88
Rotaflex	91	Rotaflex	87
Rotaflex	91	Rotaflex	86
Rotaflex	90		
AR-G	90		
Road-Mesh	89		
Road-Mesh	83		

Notes 1: Data sorted by values of percent of target density in the top layer.
2: Data sorted by values of percent of target density in the bottom layer.

In general it seems that reinforcement with larger apertures and relatively high profiles performs better than that with small apertures and low profiles, which, considering the properties of the glass-reinforcement seems to over-ride strength considerations. Table 8.11 shows a ranking of the performance of each reinforcement type.

Table 8.11. Ranking of Reinforcement by Rut-Reduction Properties: Test PTF2

Rank	N=20 000	N=30 000	N=40 000
1(smallest rut)	AR1	AR1	AR-G
2	AR-G	AR-G	AR1
3	Road-Mesh	Road-Mesh	Road-Mesh
4	Rotaflex	Rotaflex	Rotaflex

The relationship between aperture size and rut shows a trend of rut reducing with larger apertures. Intuitively, it is readily appreciated how apertures that are small relative to the aggregate would hinder the development of an effective interlock mechanism. Hozayen et al [8.5] also identified the ratio between the aperture and the aggregate size as being important, and found that a ratio of around 3 to 4 gave the best performance.

For a 14mm DBM mixture, this corresponds to an aperture size around 40 to 60mm, i.e. roughly that of the AR1 grids. In addition, if the lateral "shoving" mode of permanent deformation is linked to shear resistance, the shearbox test results in Chapter 6 also showed the glass-reinforced composites to have a lower shear resistance than the polypropylene and steel reinforcement.

8.4.6 Summary of PTF2 Test Results

Deflection

Initial deflections measured across the joints of the concrete slabs tended to show deflections were lower on control sections. This may imply that the inclusion of a reinforced interface layer tended to reduce interlayer bonding, which, perhaps, led to the asphalt layer behaving like two thinner layers. Alternatively, support under concrete slabs in the unreinforced sections may have been better than in the reinforced sections. Clegg Hammer test results show that sub-base support generally appears best under the control area and the area reinforced with grids. Although this seems to agree with deflection behaviour, because test results on the sub-base and subgrade were carried out before placing concrete slabs and the asphalt, it is difficult to estimate relative conditions under wheel loading.

With trafficking, deflections measured across tranverse joints were generally found to be higher than deflections across longitudinal joints. Also, deflections in the AR-G, Control and AR1 wheelpath were generally lower than deflections in the wheelpath across the Rotaflex, Control and Road-Mesh sections.

Cracking

More cracks tended to be found over transverse joints, and particularly in, and at the edges of the control areas. Cracks in the longitudinal direction at the edges of the wheelpath were thought to be largely due to the pronounced "shoulders" caused by permanent deformation. Overall reinforced sections tended to perform better than the unreinforced sections.

Permanent Deformation

Sections reinforced with grids were found to have less rutting than unreinforced sections. Also, as the AR-G reinforced section behaved similarly to the AR1 section in the same wheelpath, it may suggest that the magnitude of the grid profile may have some influence in the mechanism of permanent deformation.

8.4.7 Test PTF3

General

Due to the large ruts measured during test PTF2 and the consequent difficulty in observing cracking, the pavement structure was modified for test PTF3, and the test temperature reduced to 13°C. The pavement was modified by placing a 5mm thick rubber sheet of 2MPa stiffness beneath the concrete slabs to increase the relative deflection between the slabs, which in turn was intended to increase the speed of cracking through the asphalt. With less time (and therefore wheel passes) for cracking to occur, it was reasoned that less rutting would be caused. Also with a reduced test temperature, asphalt mixtures become stiffer, and thus help to reduce rutting, although the relative deflections would also be smaller.

Six test sections were trafficked during this test, as in test PTF2. The test layout is shown in Figure 8.17.

The pavement was cooled by reducing the air temperature, and was monitored using thermocouples placed in the asphalt surface along the centreline of the pavement. Trafficking was carried out with a 12kN wheel load as before.

Figures 8.F-1 to 8.F-3 in Appendix F show how changes in relative deflections developed with trafficking. The number of repetitions to "failure" i.e. active cracking, are given in Table 8.12 for both wheelpaths.

Table 8.12. PTF 3 test results.

Test section	**Number of 12kN repetitions to failure**	**Equivalent Deflection at 20°C.(mm)**
AR1	26000	1.37
AR-G	44300	1.05
Control	23200	0.94
Rotaflex	19000	2.65
Road-Mesh	27000	1.65
Control	10000	0.99

Absolute Deflection Measurement

In addition to relative deflection measurements across transverse and longitudinal joints, 'absolute' measures of deflection were taken during test PTF3 to help interpretation of deflection measurements. In particular, the measurements were taken to see if absolute deflections increased when relative deflections decreased, (as happened during the PTF2 test towards the end of the test). Absolute deflections were measured by attaching an LVDT to the side of the test pit and measuring deflections on a target adjacent to the wheelpath.

AR1-AR-G Wheelpath

Deflections-Trafficking

Figures 8.F-1 to 8.F-3 in Appendix F show all deflections measured on this wheelpath to be fairly similar, and, like test PTF2, relative deflections tend to reduce toward the end of the test. 'Absolute' deflections, on the other hand continue to increase suggesting that movement on both sides of the slab increases simultaneously, lending weight to the proposed explanation that towards the end of the test slabs tilt more or less together. This could occur if material under the edge of the slabs compacts more than in the centre of the slabs.

Cracking

With the reduction in rutting (and associated "shoulders"), little longitudinal cracking was noted, except on the control sections, and the crack pattern across transverse joints was more definite (see Figure 8.19). The majority of the cracking was present within the control section or over the joints between the control and AR1 or AR-G sections. Cracking at the junction of control and reinforced sections suggests that

(a) the reinforced interface influences performance, and
(b) if small areas of reinforced asphalt were to be used (to deal with localised cracking for instance), there is a danger that cracks will be induced at the edges of the reinforced area.

Intuitively, it follows that where a "patch" or "strip" repair is to be carried out, reinforcement used should be made or cut in a manner that reduces sharp changes between reinforced and unreinforced sections. This may be difficult to effect in practice, and could lead to additional time and cost. It is also reasoned that where a stiff (relative to asphalt) interface material is used, these 'edge effects' are more likely to occur.

The number of repetitions at which "active" surface cracking occurred was noted and a "representative" deflection (i.e. a value that takes into account all deflections prior to active surface cracking) calculated. In addition to the procedure adopted for results from Test PTF2, the reduced temperature at which Test PTF3 was carried out meant that deflections required correction. Although the accuracy of the magnitude of the correction applied is questionable, the approach is logical, and, appears to give reasonable agreement with results from PTF2. The technique used for correction of deflections for temperature is now described.

- Representative values of DBM stiffness at 13°C. and 20°C. were taken from relationships developed at Nottingham.
- Multi Layer Linear Elastic Analysis (MLLET) was used to model the PTF structure (ELSYM5 in this case), using the two different values of asphalt stiffness, and assuming that asphalt is the only material whose properties change significantly due to the temperature difference.
- The ratio of deflections was calculated for the two cases modelled, and taken as being applicable to deflections obtained from the tests.
- Deflections were then corrected using this ratio.

Permanent Deformation

The magnitude of rutting was found to be similar to the values obtained from PTF2, but the rut "shoulders" were smaller than the shoulders measured in PTF2. Also, with less pronounced shoulders their curvature was reduced, tensile strains were smaller resulting in less longitudinal cracking adjacent to the shoulders. Figure 8.18 shows how rutting increased with trafficking, and also that the relative performance of the reinforced sections improved as trafficking progressed. Rutting in the sections reinforced with AR1 and AR-G materials is seen to reduce to around 60% of that of the control section. It is interesting to note that although the AR-G is a composite material, the effect on rutting is similar to that of the AR1 grid. As both the grid and the composite have the same profile on the top interface, it appears that this factor has an important influence on rutting, particularly on horizontal "shoving" movements of the asphalt.

Material quality

Cores were taken from the pavement from all sections and used for determination of density and for visual inspection. The results in Table 8.13 show relatively low densities and high void contents, which are thought due to a combination of the thin layers and speed of construction. Unless air temperatures are high and construction is quick, thin layers cool quickly, leading to the binder becoming stiffer and resulting in a material more difficult to compact.

Table 8.13. PTF3 asphalt – density and void content

	Above Interface		Below Interface	
	Density (Mg/m^3)	Voids (%)	Density (Mg/m^3)	Voids (%)
Maximum	2.26	17.19	2.31	16.47
Average	2.19	15.04	2.24	12.98
Minimum	2.13	12.03	2.15	10.17

The main points derived from test results are summarised at the end of this section together with the main points drawn from test results from the Road-Mesh and Rotaflex wheelpath.

Rotaflex, Control and AR-G Wheelpath

The number of repetitions to "failure" i.e. active cracking, under a nominal 12kN wheel load is shown in Table 8.12.

Deflections-Trafficking

Plots of deflection versus wheel passes are shown in Figures 8.F-1 to 8.F-3 in Appendix 8.F. The most noticeable feature of the behaviour of deflections shown in Figures 8.F-1 and 8.F-2 is the relatively high deflections in the Rotaflex section.

Although failure of the unreinforced sections was expected to occur before that of the reinforced sections, the number of wheel passes measured on the section between the Rotaflex and Road-Mesh sections (10000) appears uncharacteristically low. In this case, failure occurred first at the junction between the Rotaflex and Control sections, and would not be typical of a large installation of reinforced asphalt. This point is therefore not included in the regression seen in Figure 8.20.

Rutting

Measures of permanent deformation plotted against load repetitions are shown in Figure 8.18.

A significant feature of Figure 8.18 is the rutting measured on the Rotaflex section, being greater than values from the Control section. The reason for this could be linked to the shallow profile of the grid and fabric, which may provide less resistance to horizontal movement than for unreinforced material. This was discussed in Section 8.4.5 and 8.4.7.

As for test PTF2, the ability of interface materials to help reduce rutting can be ranked by correlating grid profile height or the ratio of grid aperture to aggregate size with the number of wheel passes to achieve different rut depths. The rankings for this test are given in Table 8.14 and are discussed further in Section 8.5.

Table 8.14. Ranking of Reinforcement materials according to rut-reduction: Test PTF3.

Performance	Material N=20,000	Material N=30,000	Material N=40,000
1(Best)	RoadMesh	RoadMesh	RoadMesh
2	Rotaflex	AR1	ARG
3	ARG	ARG	AR1
4	AR1	Rotaflex	Rotaflex

Cracking

Longitudinal cracking was almost non-existent during this test unlike transverse cracks which were quite obvious, especially from around 30000 repetitions.

Figure 8.19 shows how the majority of the cracking was found within the Control sections, and over the joints marking the edges of the reinforced and control sections. Cracks on the junction of the Rotaflex and Control sections developed quickly once cracking began. This may be linked to the differences between the stiff glass-reinforced composite and the unreinforced interface. The quick increase in crack length with repetitions is similar to the behavior noted during tests on beams reinforced with interface products incorporating glass products. This may suggest that when a material much stiffer than asphalt is used 'within' it, some consideration should be given to helping reduce the difference in elasticity between it and the asphalt. In this regard, another glass-reinforced product, 'GlasGrid', which has a self-adhesive backing, has apparently been used in the field with success. It is considered that the adhesive backing may help accommodate the differences in elasticity between the glass and the asphalt.

Material Quality
Cores were taken from each section in the pavement and used for visual inspection, density, stiffness and interface bonding determination. The relatively low values for interface shear, density and stiffness are discussed in Section 8.5.3, and seem to be in line with the cracking and deformation results.

8.5 Overall Analysis of PTF results

8.5.1 Deflection and cracking
To obtain characteristic deflections, representing the period from the start of trafficking up to cracking, the procedure given in section 8.4 has been used. A simplified measure of the energy or "work-done" by the pavement is used to derive a single 'equivalent' deflection. Also, to compare test results from test PTF2 to test PTF3 (carried out at a different temperature), deflections have been corrected using linear elastic theory and "typical" values of asphalt stiffness at the different temperatures. The procedure is also given in Section 8.4.

For each section, the number of traffic passes to failure has been plotted against "equivalent deflection" in Figure 8.20, which shows two distinct groups of points: control sections and reinforced sections. Data from reinforced sections are reasonably well-represented by the regression line shown in the figure, apart from the data point at 2.65mm deflection and 19000 repetitions, (representing the Rotaflex section in test PTF3).

From Figure 20, it appears approximately, that reinforced sections withstand around two-and–a-half times the number of repetitions taken by control sections before active cracking occurs.

8.5.2 Permanent Deformation
Most reinforced sections were found to have significantly reduced permanent deformation when compared to unreinforced sections. However, it is difficult to directly compare results of PTF2 and PTF3 as the tests were carried out at different temperatures. To help compare products, therefore, measures of permanent deformation have been normalised by dividing data from reinforced sections by measurements taken in the adjacent unreinforced section. The relative performance is thus more obvious, as is seen in Figure 8.21.

From Figure 8.21, and other results given in this chapter, the following points are noted:

- the inclusion of reinforcement usually leads to less permanent deformation,
- different types of reinforcement influence permanent deformation to different degrees,

- the permanent deformation observed during the PTF tests was due to both horizontal movement, i.e."shoving". and vertical deformation.
- Rutting in most reinforced sections varied between 30% and 60% of those in control sections, which is significant and has promise for applications in the field,
- generally the grid-reinforced sections have less rutting than the composite-reinforced sections,

In explaining the observed rutting behaviour, the horizontal component of deformation is considered to play a key role, especially as the thin asphalt layers were underlain by rigid concrete slabs, thus reducing vertical deformation. It seems that the horizontal component of permanent deformation ('shoving') can also be reduced depending on the bond between the reinforcement and the asphalt (which may be due to either adhesion or interlock). In turn, mechanical interlock of reinforcement and asphalt is affected by the profile of the reinforcement. This explains the poorer performance of the Rotaflex-reinforced sections in both tests PTF2 and PTF3.

To investigate the relationship of grid profile and permanent deformation, rut at 20,000, 30,000 and 40,000 repetitions was plotted against grid profile, and the ratio of maximum aggregate size. Figures 8.22 and 8.23 show quite distinct trends of rut reduction with larger ratios of aperture to aggregate size and bigger grid profiles. The tentative correlations support the intuitive understanding that deeper grid profiles and grid apertures considerably larger than aggregate in the asphalt mixture help to develop more resistance to lateral movement through interlock. This has important implications in the design of reinforced asphalt, and is supported by the findings of Hozayen et al [8.5] who found that asphalt performed better when reinforced with materials with ratios of aperture to aggregate size greater than around 3 or 4.

Although tests PTF2 and PTF3 were carried out at different temperatures, similar trends are obtained. This suggests that the effect of the temperature difference on bituminous interlayer bonds was less significant than the effect of mechanical interlock. Also, because sections reinforced with AR-G behaved similarly to those reinforced with AR1, it suggests that the profile on the upper interface is important, and in these tests, appears to be dominant in influencing behaviour.

8.5.3 Asphalt Characteristics

To help interpret PTF results, and to monitor the quality of installation, 100mm diameter cores were taken from each test section. Densities obtained from the cores are shown in Figure 8.24, expressed as percentages of the target density (2.44), corresponding to a void content of 5%. There is considerable variation in results, with the Road Mesh sections giving the highest range, and samples from the unreinforced section giving (on average) the highest values. The variation in densities for the Road Mesh is perhaps understandable when the configuration of the material is considered, as some disruption to compaction is to be expected where wires are twisted at the nodes. It follows that the converse is true for the unreinforced sections.

In some cases, during coring, the layer of asphalt under the reinforcement broke up and partially remained on the underlying concrete slab. In these cases density values could not be obtained.

Some cores were taken through cracks, and were inspected to help clarify the influence of reinforcement on cracking. However, as cracks were narrow (typically 1mm or less, wide), they were difficult to trace through the layer. In some cases cracks appeared to be wider at both the top and bottom of the upper layer of asphalt than in the middle of the asphalt layer. This may suggest that that cracking could have initiated both on the top and bottom surfaces. If some of the cracks did initiate at the surface, it is considered that they may have been caused by the unusually (for a pavement) large deflections induced by the rubber layer and the soft upper layer of subgrade.

The bond between the asphalt and the concrete slab was found to be good in all the cores taken through the slab centres, but often poor adjacent to joints in the slabs. This probably indicates the effect of movement at the joints. No definite pattern was found regarding where debonding had occurred, but it was more noticeable on the grid-reinforced and control sections than the composite-reinforced sections. In an attempt to obtain a more quantitative measure of interface bonding, shear adhesion and direct tension tests were carried out, as described in the following sections.

Shear Adhesion Tests

The apparatus shown in Figure 8.25 was used to test 100mm x 100mm samples cut from the PTF3 pavement. Samples were tested at a rate of 10mm per minute, and the peak load and corresponding displacement recorded. Table 8.15 lists the results.

Table 8.15 Shear Adhesion Test Results

Sample	Peak Load (N)	Displacement (mm)	Shear Modulus (N/mm/mm^2)
Rotaflex -Slab 23 -Slab 27	389 313	0.51 0.51	0.08 0.06
AR1 -Slab 10 -Slab 6	806 1169	0.52 0.71	0.16 0.16
Road-Mesh - Slab 7	1536	1.56	0.10
AR-G -Slab 26	1611	0.92	0.18
Control -Slab 15	1517	0.67	0.23

The results in Table 8.15 and Figure 8.26 show the unreinforced sample to have the highest shear modulus, and the Rotaflex composite, the lowest. Generally the order of results appears logical, with the control having the highest shear resistance, and a composite, the lowest. However, an exception seems to be the AR-G composite, which was found to have a higher shear resistance than either the AR1 or the Road Mesh grids.

The ranking of these results is broadly compatible with those obtained from the shearbox (see Chapter 6), where composite stiffnesses were found to be lower than both control and grid-reinforced specimens. The low values of interface stiffness (as compared with the shearbox test results) are explained in part by the difference in sample size, the different quality of construction, and sample preparation.

Sample size is thought to influence results due to edge effects. The effect of relatively high edge stresses is likely to be more pronounced on the relatively small (100mm x 100mm x 60mm) blocks cut from the PTF pavement than on the larger (380mm x 200mm x 120mm) shearbox samples. Quicker initiation and propagation of bond failure is expected over the small sample area. More investigation into the effects of sample size on stress distribution and the implications to bond failure should be carried out to help resolve differences in test results.

Poorer construction techniques in the PTF are thought to be the reason for the lower density and higher air void contents in the PTF materials. Compaction in the roller-compactor is easier to control than compaction in the PTF where thin layers are prone to cool quickly, and thus become more difficult to compact.

Linked to the comments on sample size is the effect of sample preparation. Breaking out samples from the PTF and the subsequent sawing is considerably more likely to lead to more damage than for samples that are made in the roller-compactor and then sawn.

Direct Tension Tests

Tensile properties of the interface bonds were measured on cores from test PTF3 by applying tensile loads to samples through steel plates glued to either end of the samples. To obtain data on the variation of tensile resistance with loading speed, a range of test rates was used. Figure 8.27 shows results differentiated by the position of failure, i.e. at the concrete-asphalt interface or at the asphalt-reinforcement interface.

Test results indicate that Road Mesh-reinforced specimens and unreinforced specimens perform similarly, and have a higher tensile resistance than the other samples. The reason for the similarities between these results could be due to the large aperture size of the Road Mesh giving a relatively large contact area of upper and lower layers of asphalt.

65% of the samples failed on the interface between asphalt and reinforcement, implying that the asphalt generally bonded better to the concrete than to the reinforcement. There seems to be no definite pattern to either the mode of failure or the magnitude of failure stress, although unreinforced samples and the samples reinforced with grids have generally higher failure stresses than do samples reinforced with composite reinforcement.

Notwithstanding the relatively small number of samples tested, results indicate that there is little or no correlation between tensile bond strength and reflective cracking. This seems to imply that the tensile strength of the interface bond does not play a large role in the cracking mechanism in the pavement test facility test configuration.

8.6 Conclusions from the PTF Work

(i) Pavement Structure

The pavement structure was designed to initiate and propagate reflective cracks. To achieve this, 600mm x 600mm x 60mm concrete slabs were placed over a rubber sheet which in turn lay on a 150mm sub-base. Finally, the reinforced asphalt surfacing was placed over the concrete blocks. The ensuing crack pattern appears to vindicate the design as cracks appeared predominantly in the direction of tranverse joints. Where cracks in the longitudinal direction formed, they were largely masked by the rut 'shoulders' adjacent to the wheeltrack.

(ii) Construction

Tests on asphalt cored from the pavement showed high void contents, low densities and low stiffness. This is attributed to the combination of thin layers and hand-roller compaction.

The properties of both the subbase and the subgrade were found to be consistent across the test area, implying that differences in the performance of the unreinforced and reinforced sections are unlikely to be due to differences in foundation support. The subgrade had a (DCP-derived) CBR of around 3% in the uppermost 300mm, which increased to between 10 and 20% CBR at around 900mm. The density of the sub-base was between 2100 and 2220kg/m^3, which is fairly typical for a well-graded crushed limestone.

(iii) Reflection Cracking

For a thin pavement on a soft foundation, reinforced asphalt withstands around twice, to two-and–a-half times the number of wheel loads carried by unreinforced material before 'active' cracks appear.

Both grids and composite reinforcement types were found to follow the same trend line on a plot of relative vertical deflection versus traffic. Likewise, there was no differentiation between glass, polymer or steel reinforcement.

The findings relating to reflective cracking relate to a specific structure and load configuration. It follows that results cannot be extrapolated to realistic structures without considerable further analysis.

(iv) **Rutting**

Both vertical and horizontal permanent deformation was found for the pavement structure used. Permanent deformation in the horizontal direction was manifested in raised shoulders on either side of the wheel track.

Significant differences were noted between rutting in reinforced and unreinforced sections. In particular, grid-reinforced sections reduced rutting by around 60%.

The reason for the reduction appears to be linked to both the profile (or roughness) of the reinforcing material, and the aperture size (relative to the maximum aggregate size). In particular, the deeper the profile, the more reduction in permanent deformation was found.

Composite reinforcing materials with low profile were found to perform little or no better than unreinforced sections, although composite materials with a distinct profile (i.e. AR-G) performed similarly to grid-reinforced sections.

Reinforced sections were seen to have a similar proportional effect on rut reduction at both 20°C. and 14°C.

(v) **Instrumentation**

A range of equipment was used to instrument pavement performance, as indicated in Table 8.16.

Table 8.16 Instrumentation used to Measure Pavement Performance

Apparatus	Property Measured
LVDT	Vertical deflection
Demec Gauge	Horizontal (surface) movement
Earth Pressure Cell	Vertical subgrade stress
Strain Coils	Lateral movement of concrete slabs
Rut profile bar	Permanent deformation

The apparatus found to provide the most useful measures of pavement performance were the LVDTs and the rut profile bar.

Output from the Strain Coils was found to be distorted by the metal in the PTF wheel and was virtually meaningless. Readings from the Demec gauge were not particularly useful due to difficulties of anticipating positions where cracks were to form.

8.7 References

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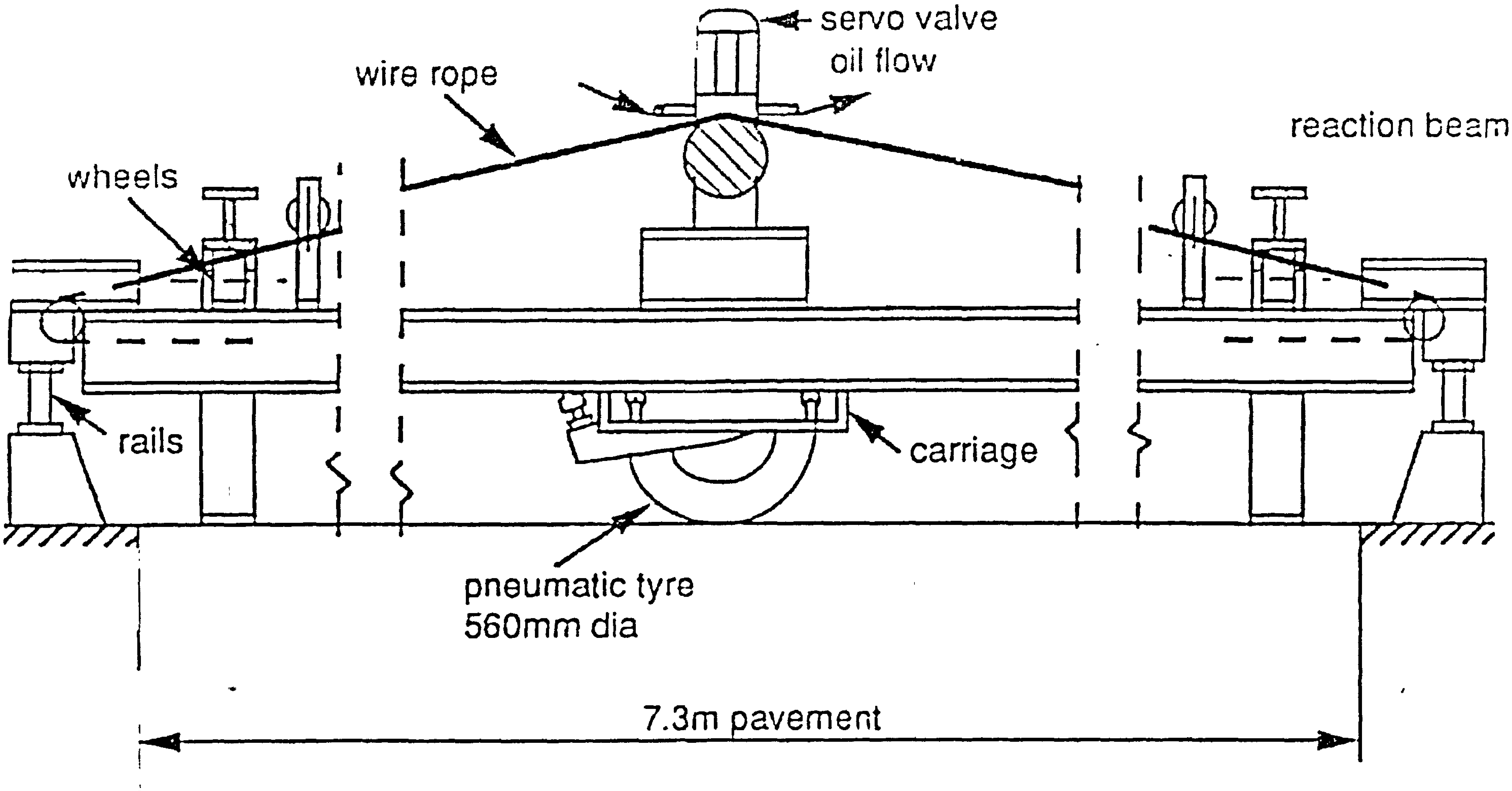


FIGURE 8.1
NOTTINGHAM PAVEMENT TEST FACILITY [8.6]

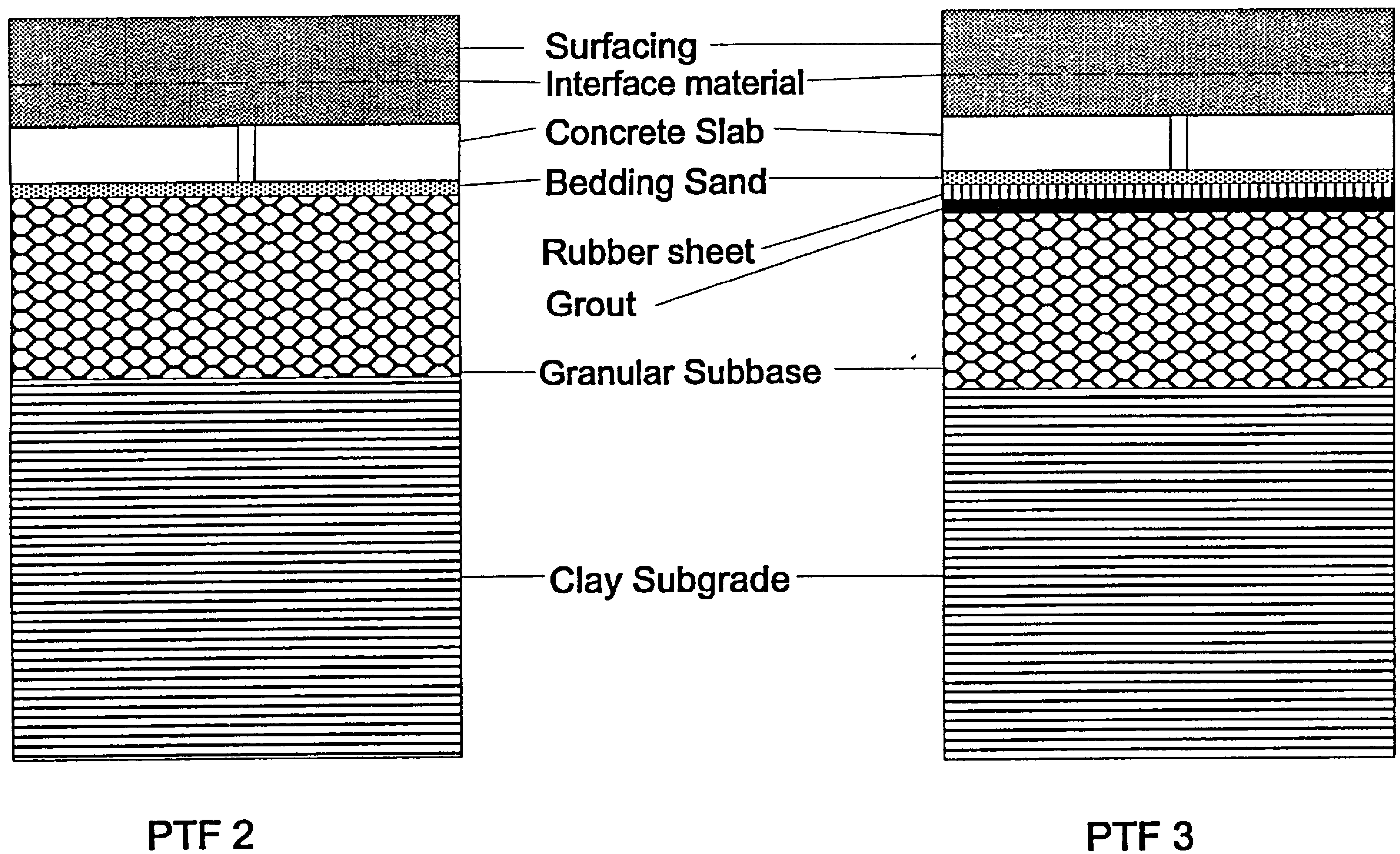


FIGURE 8.2
PTF PAVEMENT STRUCTURES

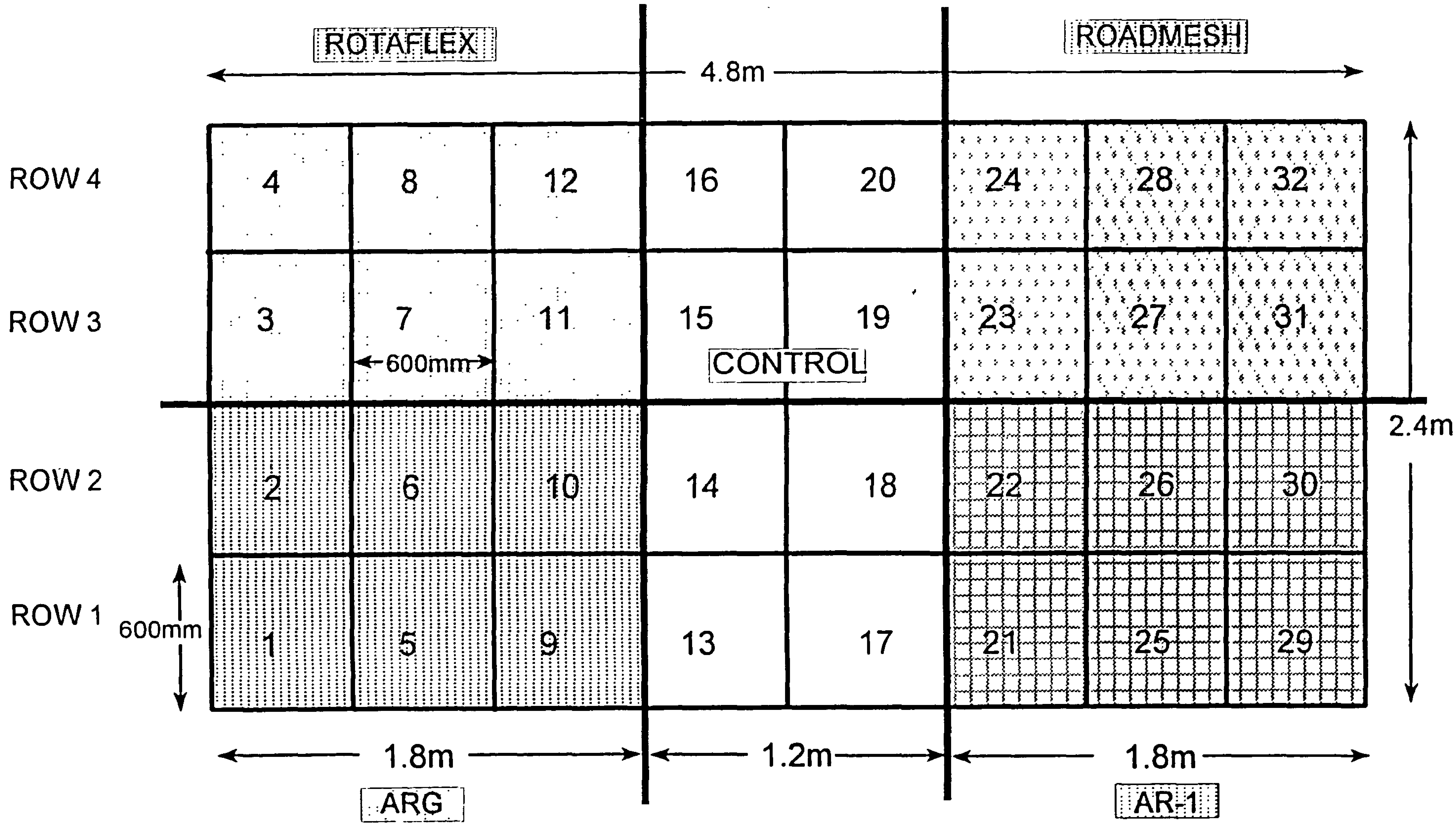


FIGURE 8.3
TEST CONFIGURATION : PTF2

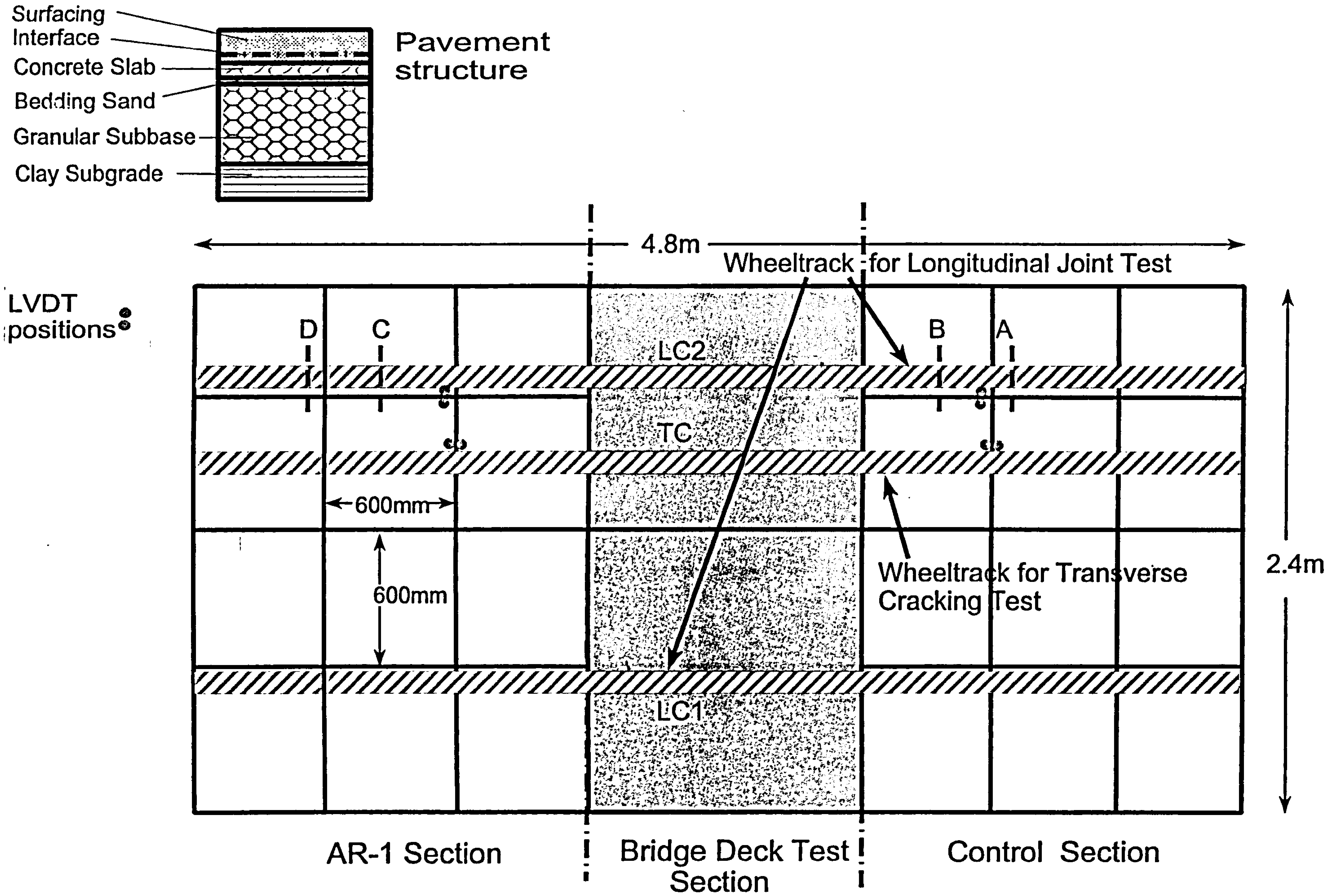


FIGURE 8.4
TEST CONFIGURATION:PTF1

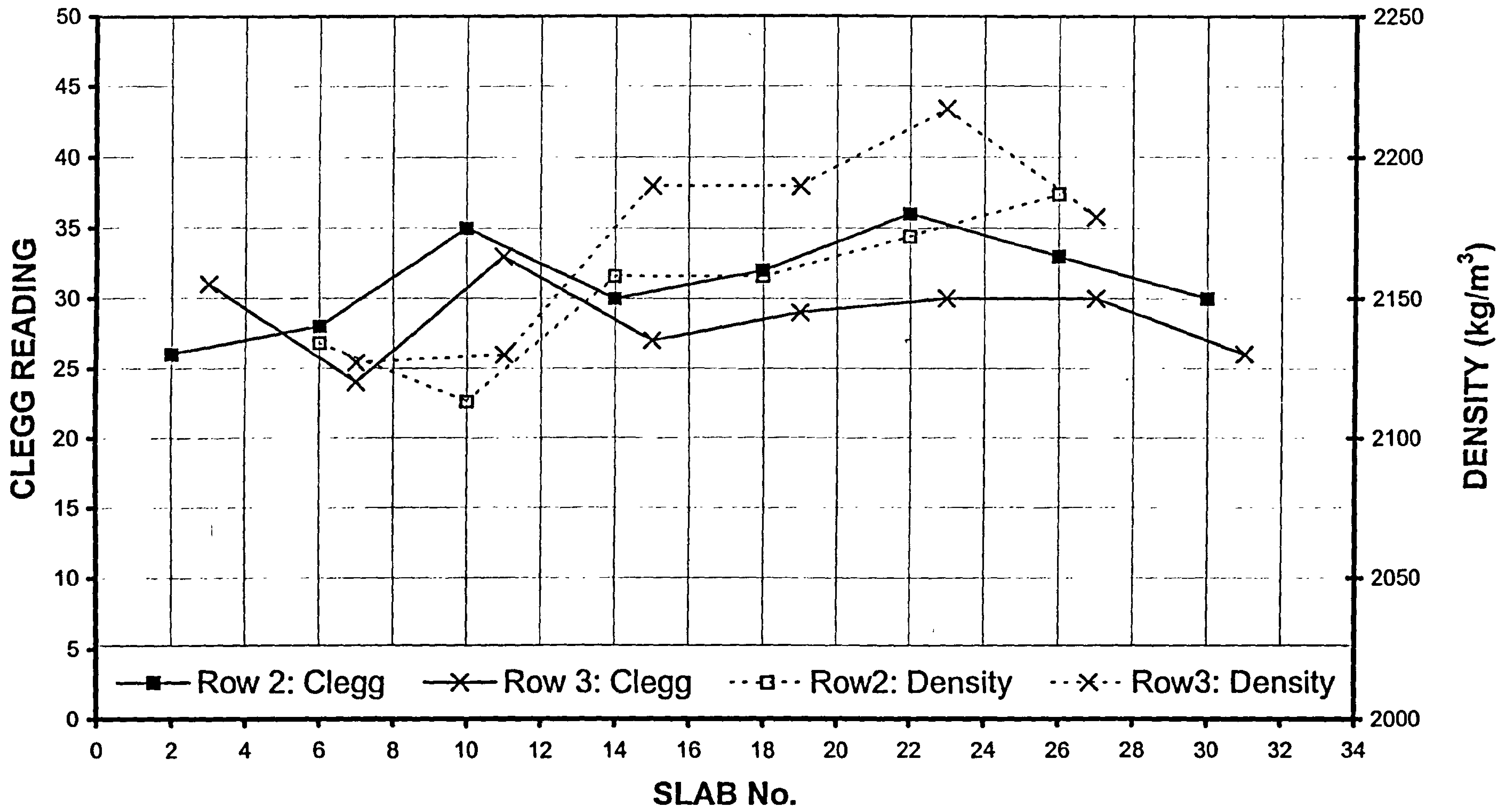


FIGURE 8.5

PTF2 SUBBASE - PRE TRAFFIC TESTING
CLEGG HAMMER AND NUCLEAR DENSITY GAUGE READINGS

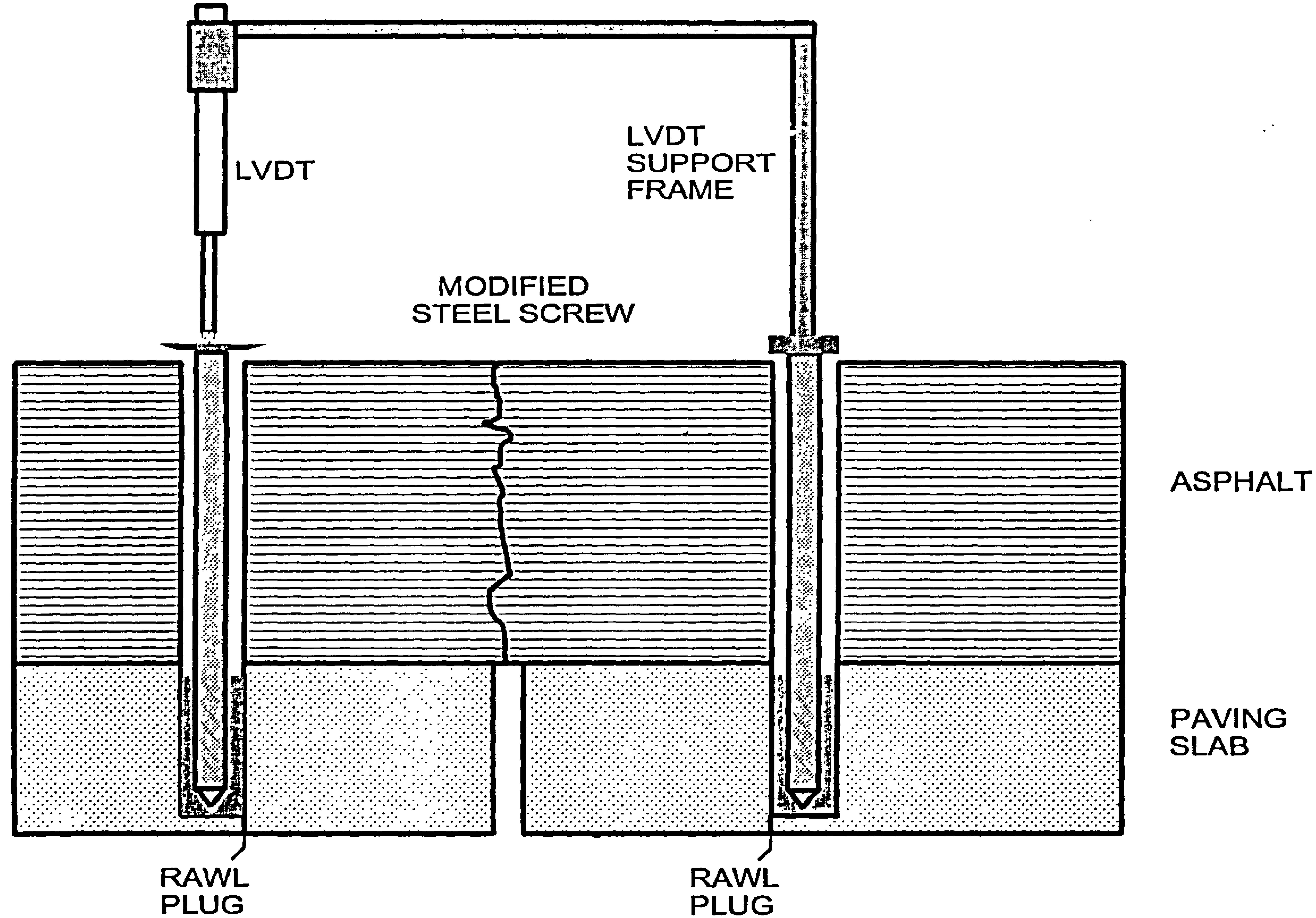


FIGURE 8.6
APPARATUS USED TO MEASURE DIFFERENTIAL DEFLECTION ACROSS CONCRETE JOINTS

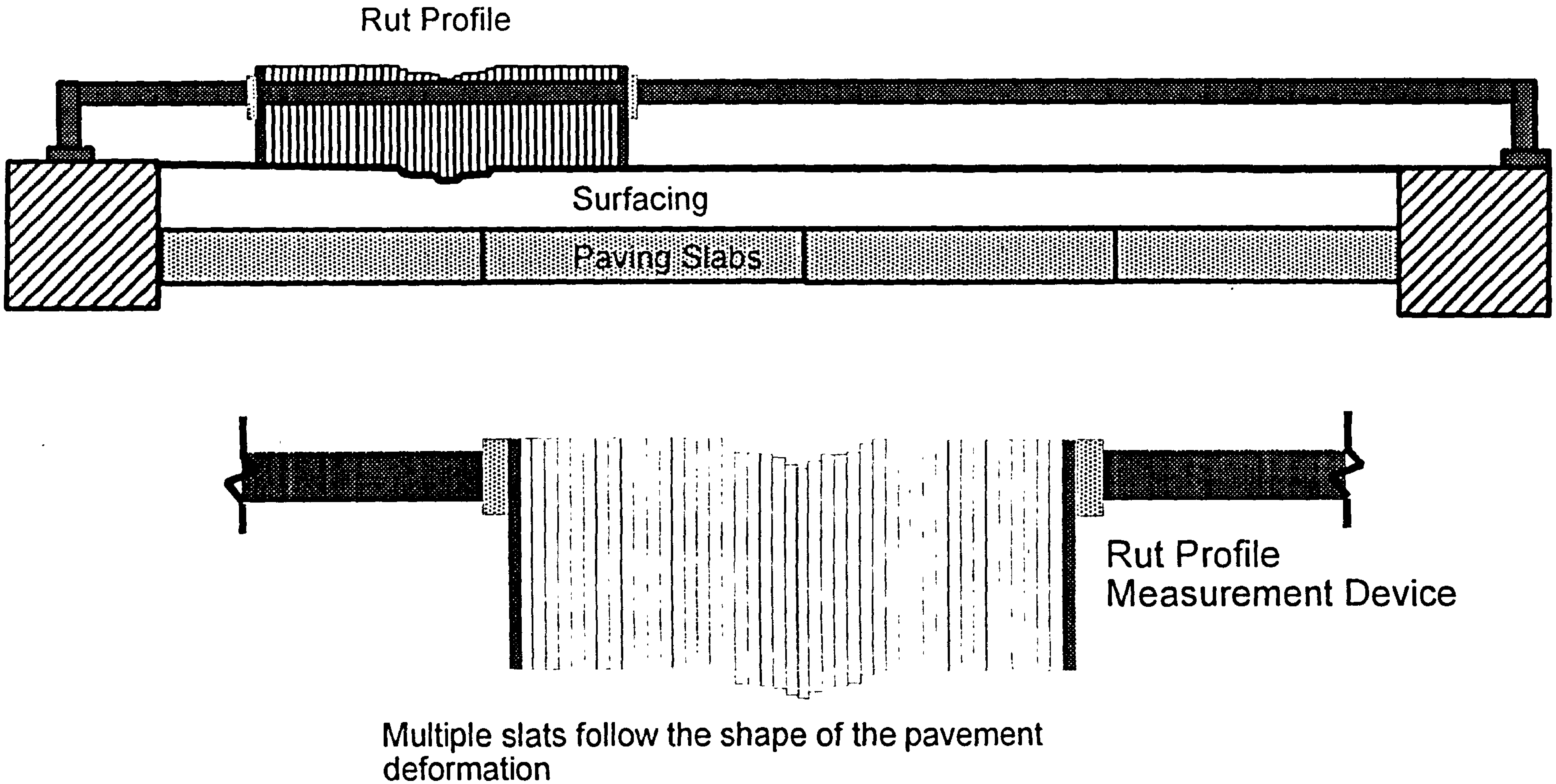


FIGURE 8.7
RUT PROFILE MEASURING DEVICE

FIGURE 8.8
GENERAL LAYOUT OF INSTRUMENTATION OVER CONCRETE SLAB JOINTS

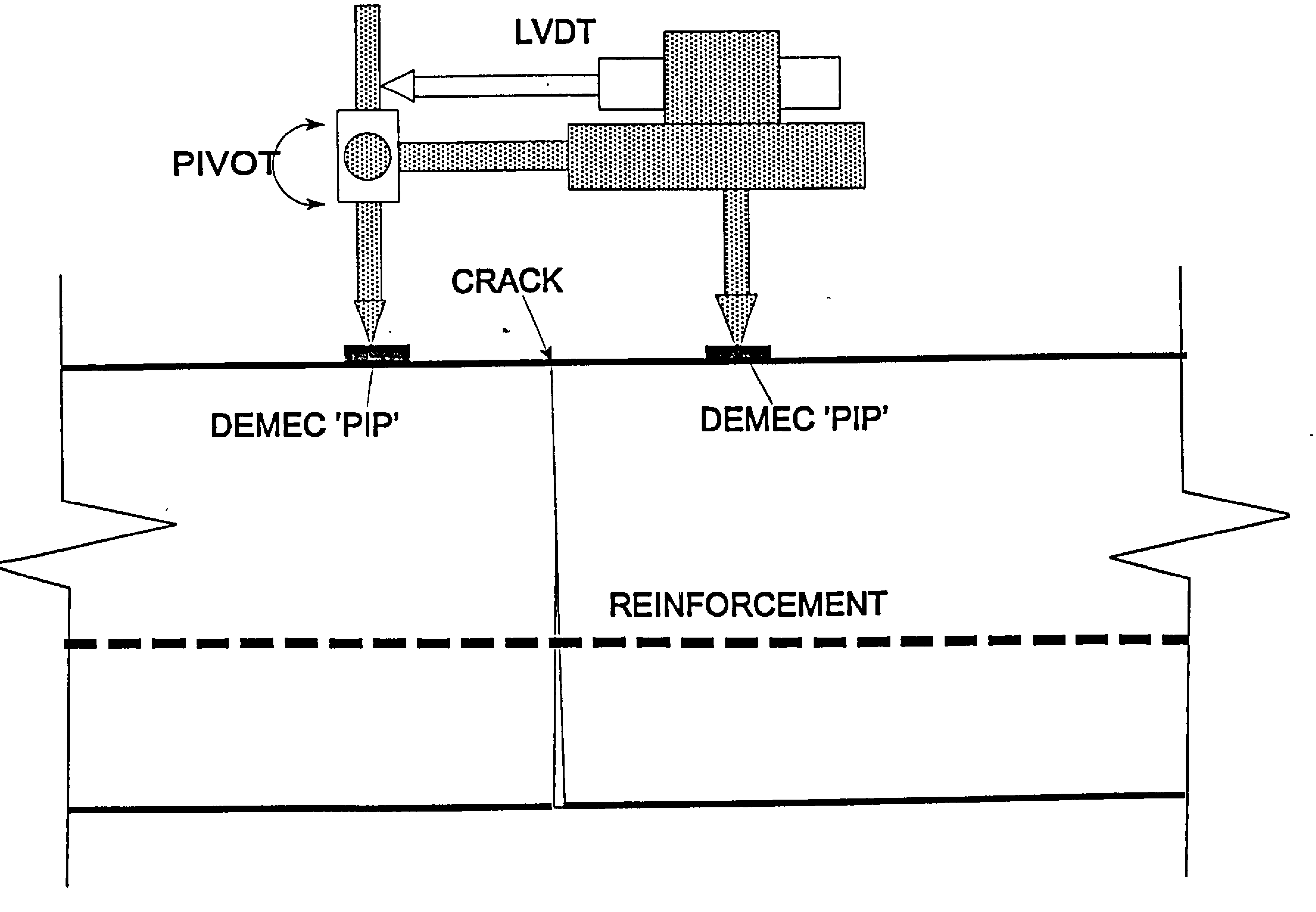


FIGURE 8.9
DEMEC-LVDT ARRANGEMENT FOR SURFACE STRAIN MEASUREMENT

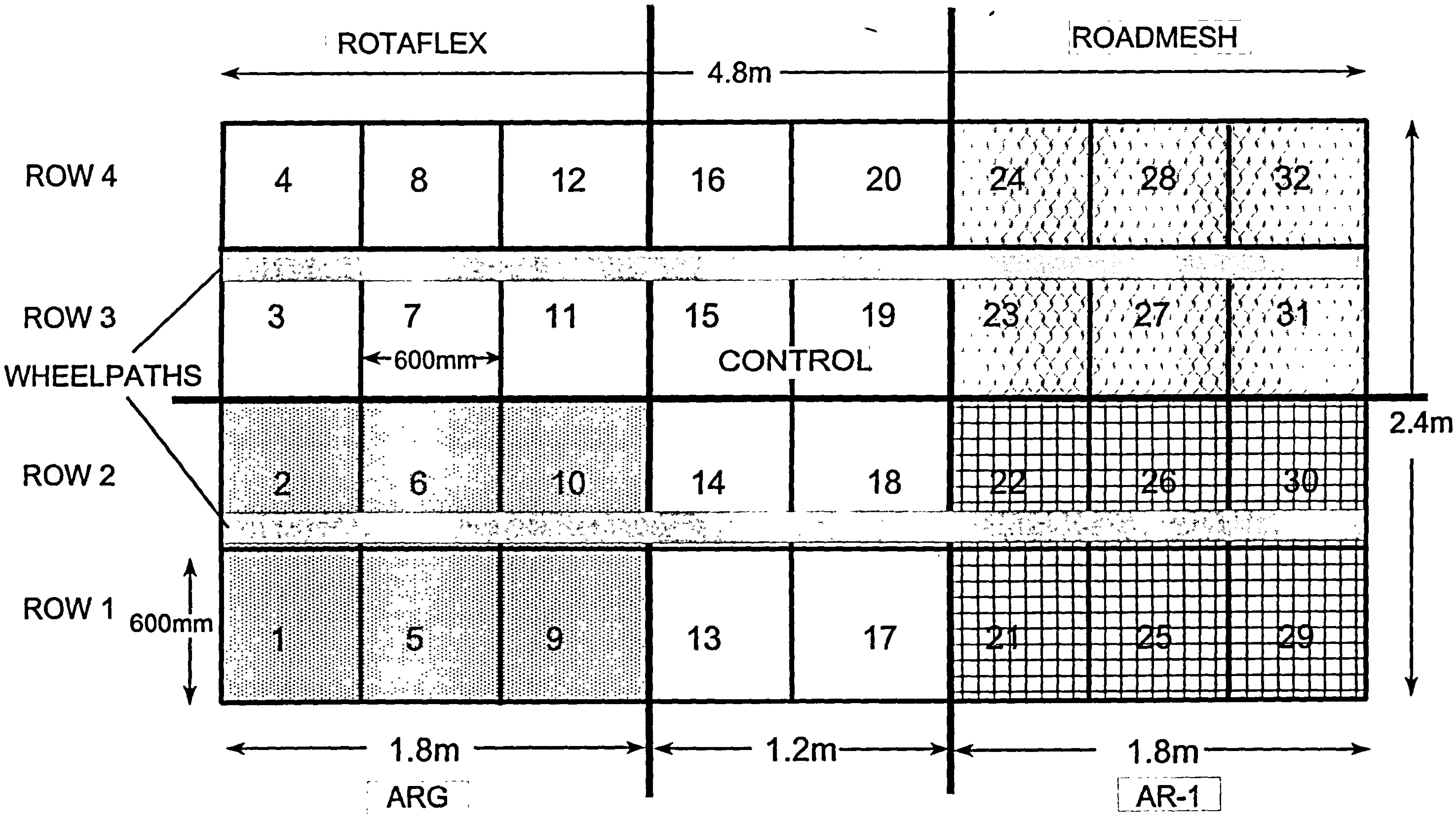


FIGURE 8.10
TEST CONFIGURATION: PTF2

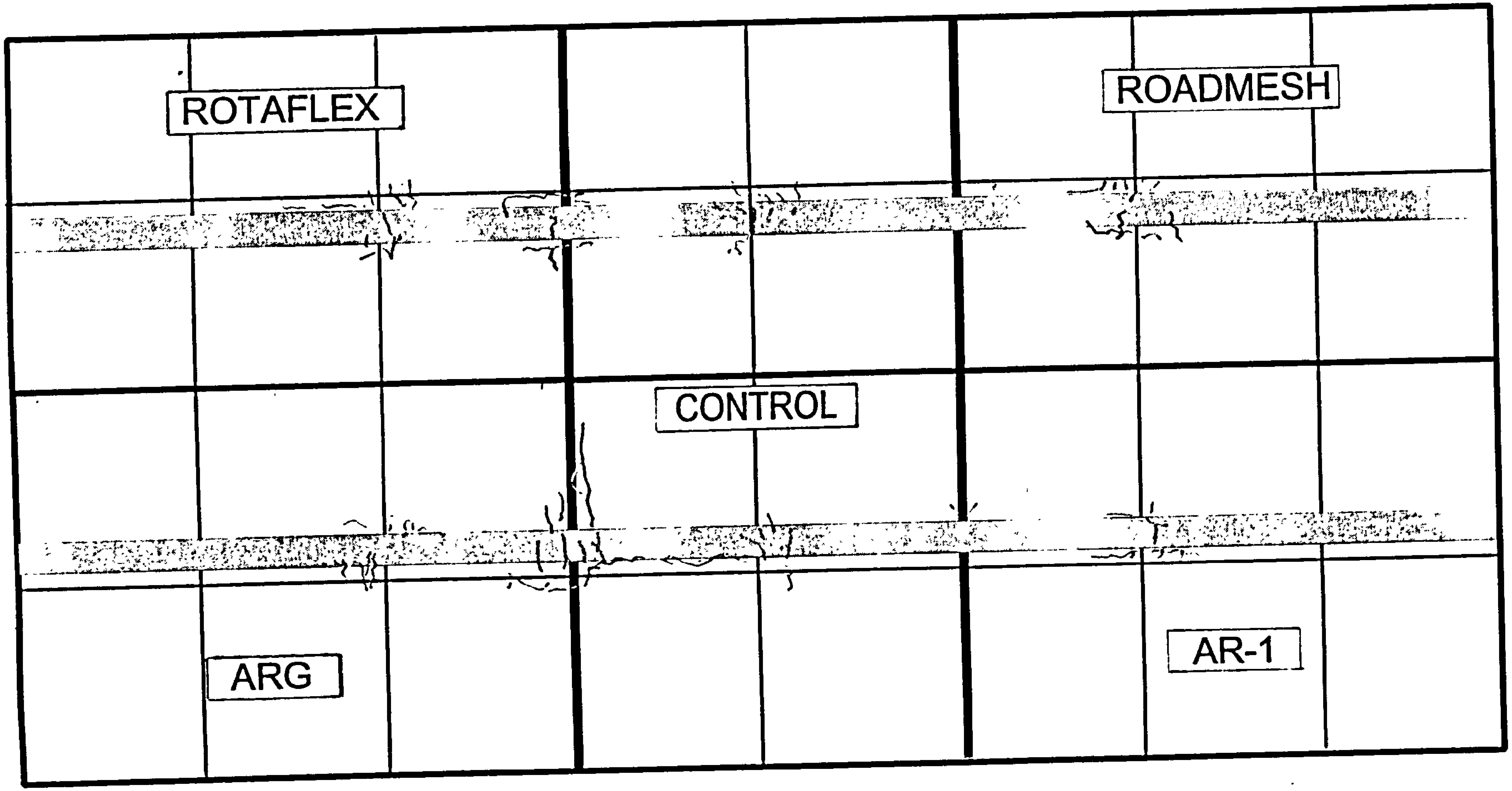
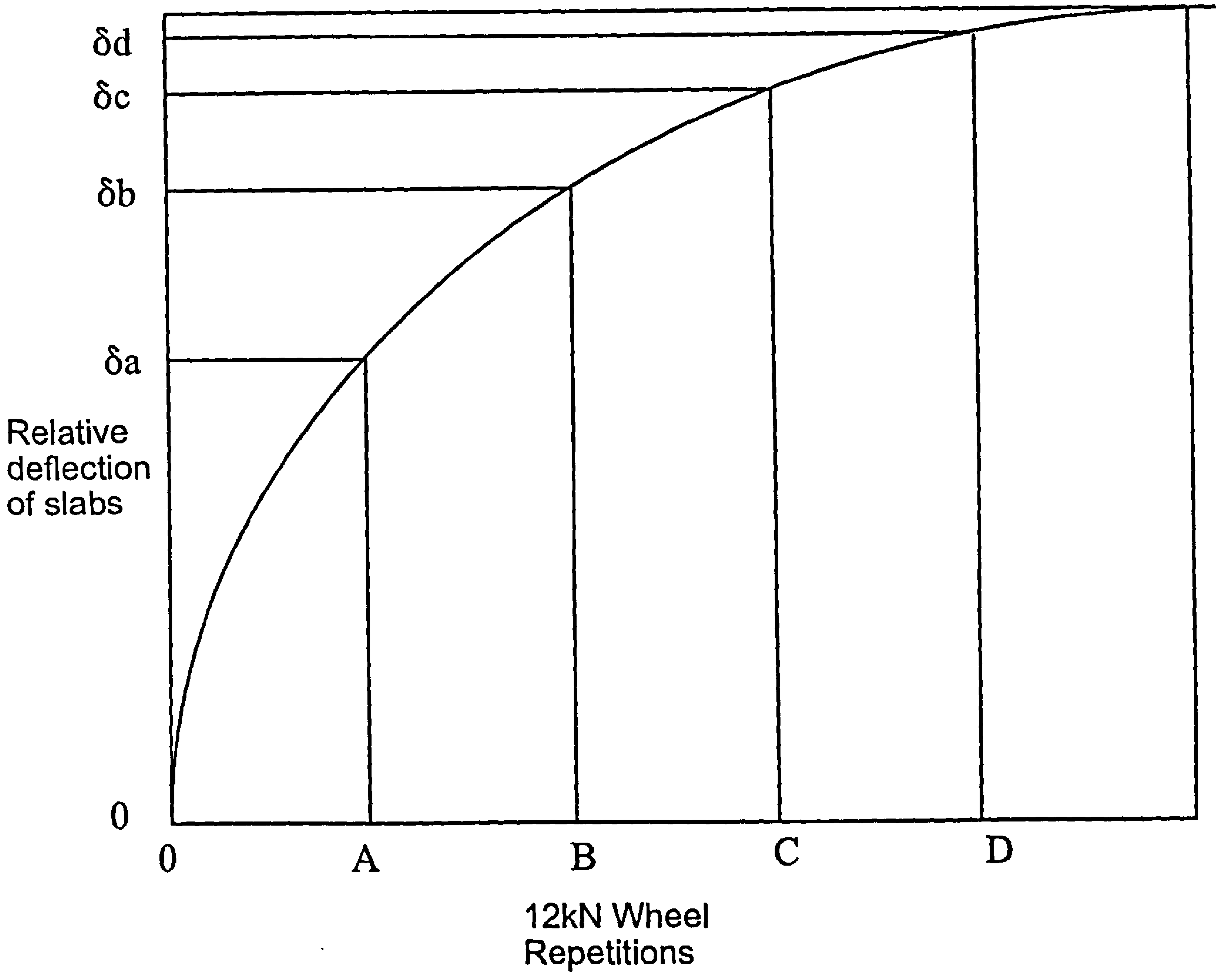


FIGURE 8.11
CRACK PATTERNS : PTF2



$$\begin{aligned}&(\delta a + 0)/2 \times (A - 0) +\\&(\delta b + \delta a)/2 \times (B - A) +\\&(\delta c + \delta b)/2 \times (C - B) +\\&\underline{(\delta d + \delta c)/2 \times (D - C)}\\&= \Sigma(\delta N)\end{aligned}$$

$$\text{Equivalent } \delta = \frac{\Sigma\delta N}{\Sigma N}$$

FIGURE 8.12
CALCULATION OF NORMALISED DEFLECTIONS

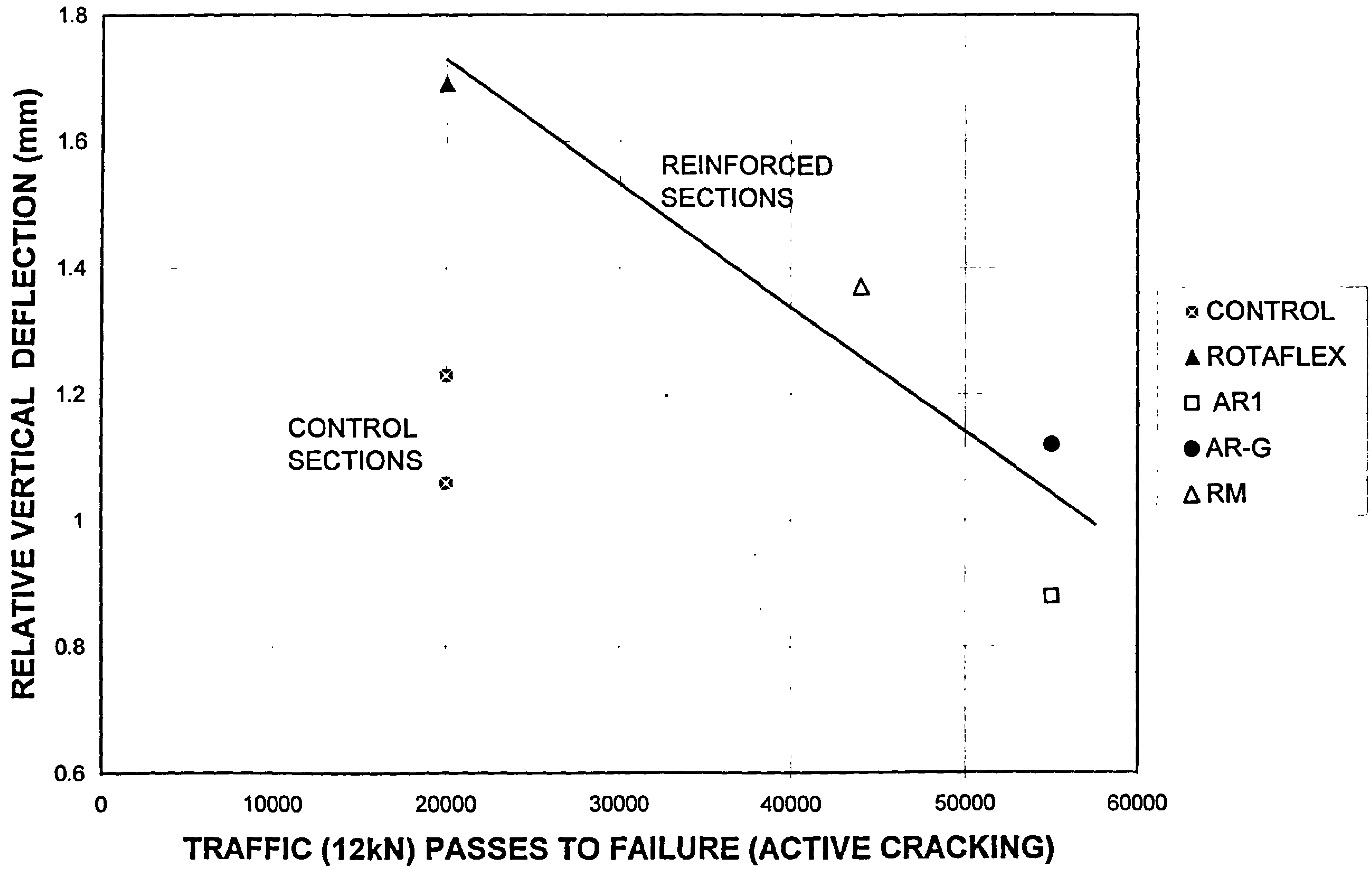


FIGURE 8.13
CRACKING versusTRAFFIC and TRANSVERSE DEFLECTION: PTF2

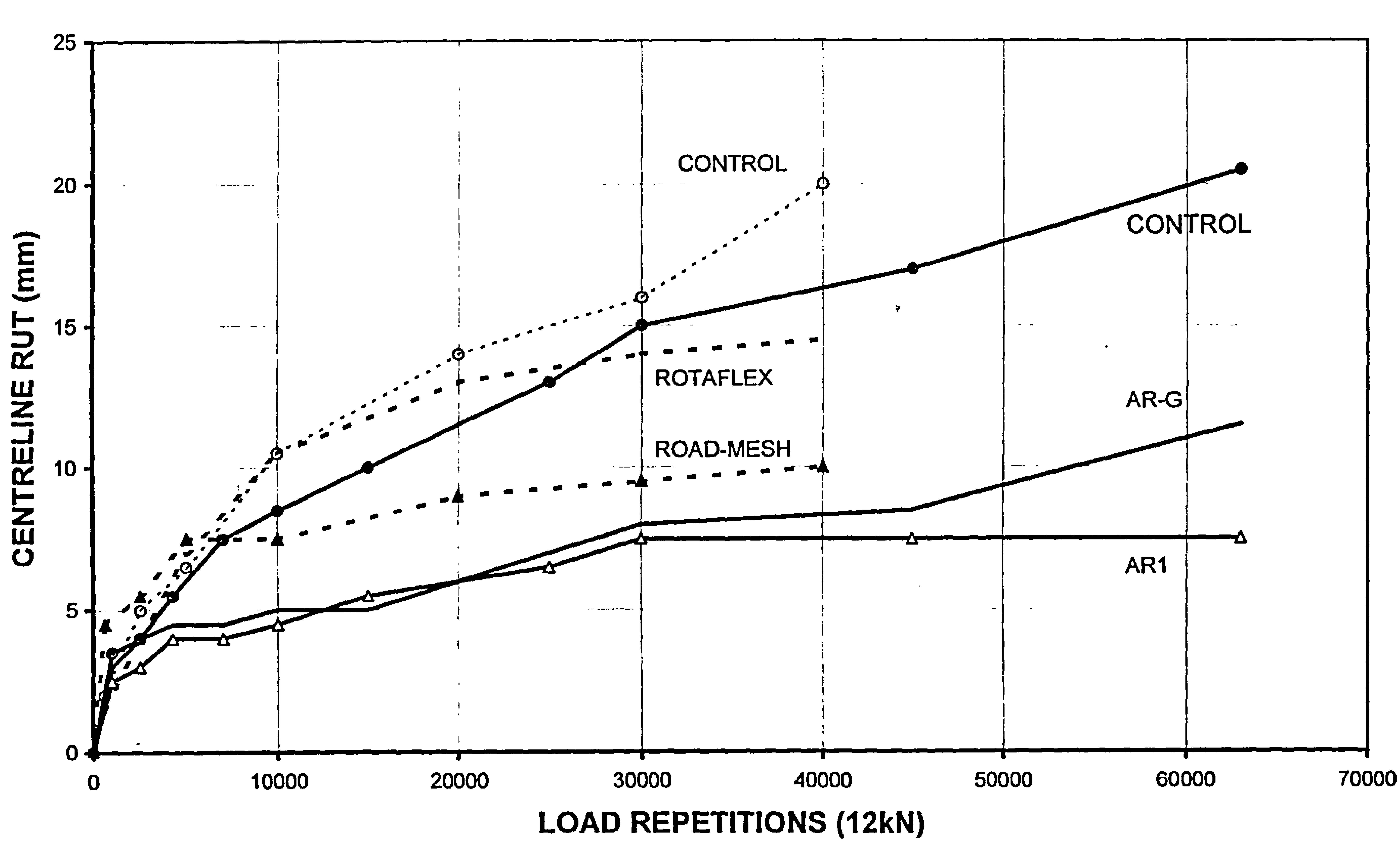


FIGURE 8.14
RUT versus LOAD REPETITIONS: PTF2

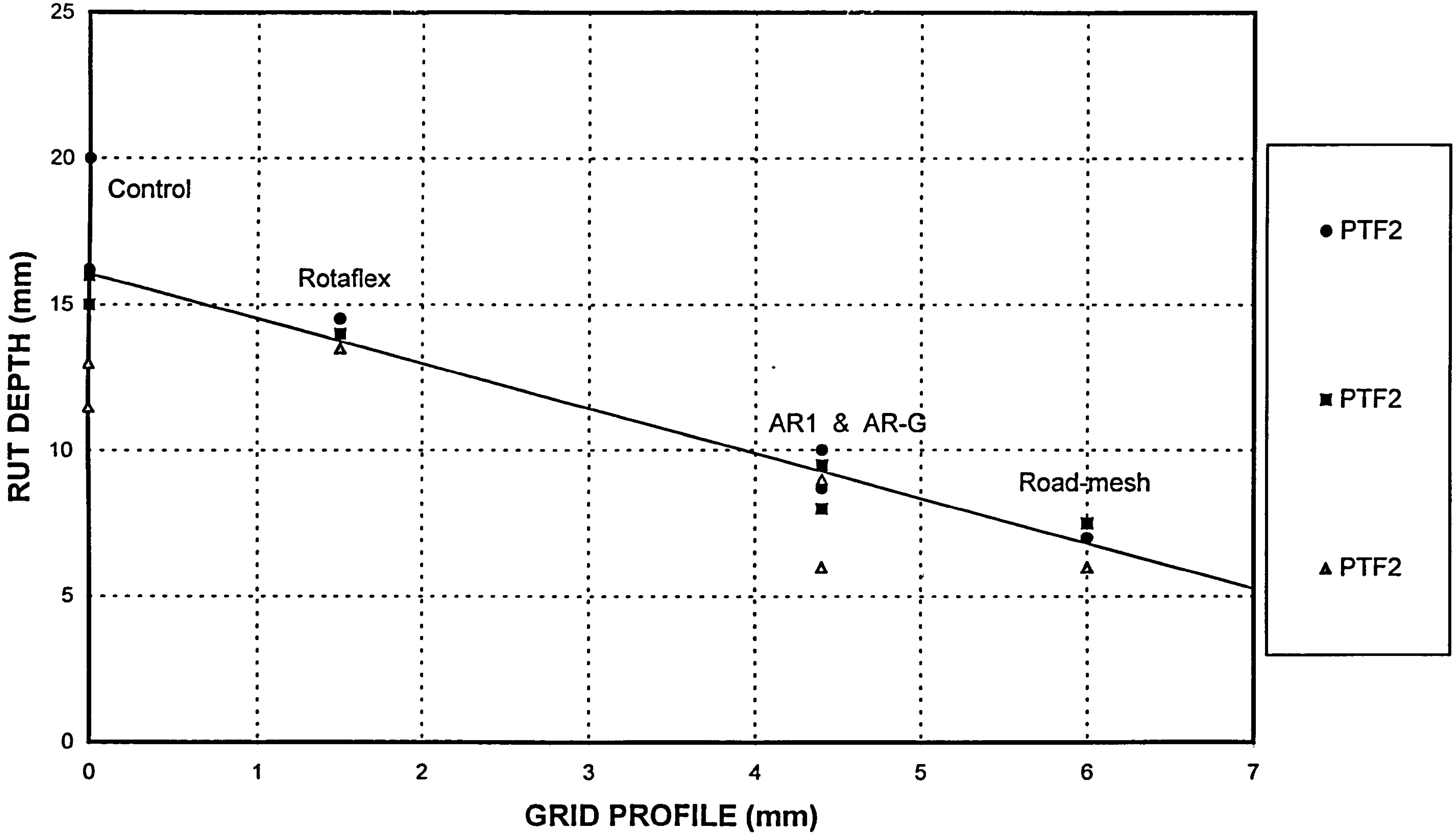


FIGURE 8.15
RUT DEPTH-GRID PROFILE: PTF2

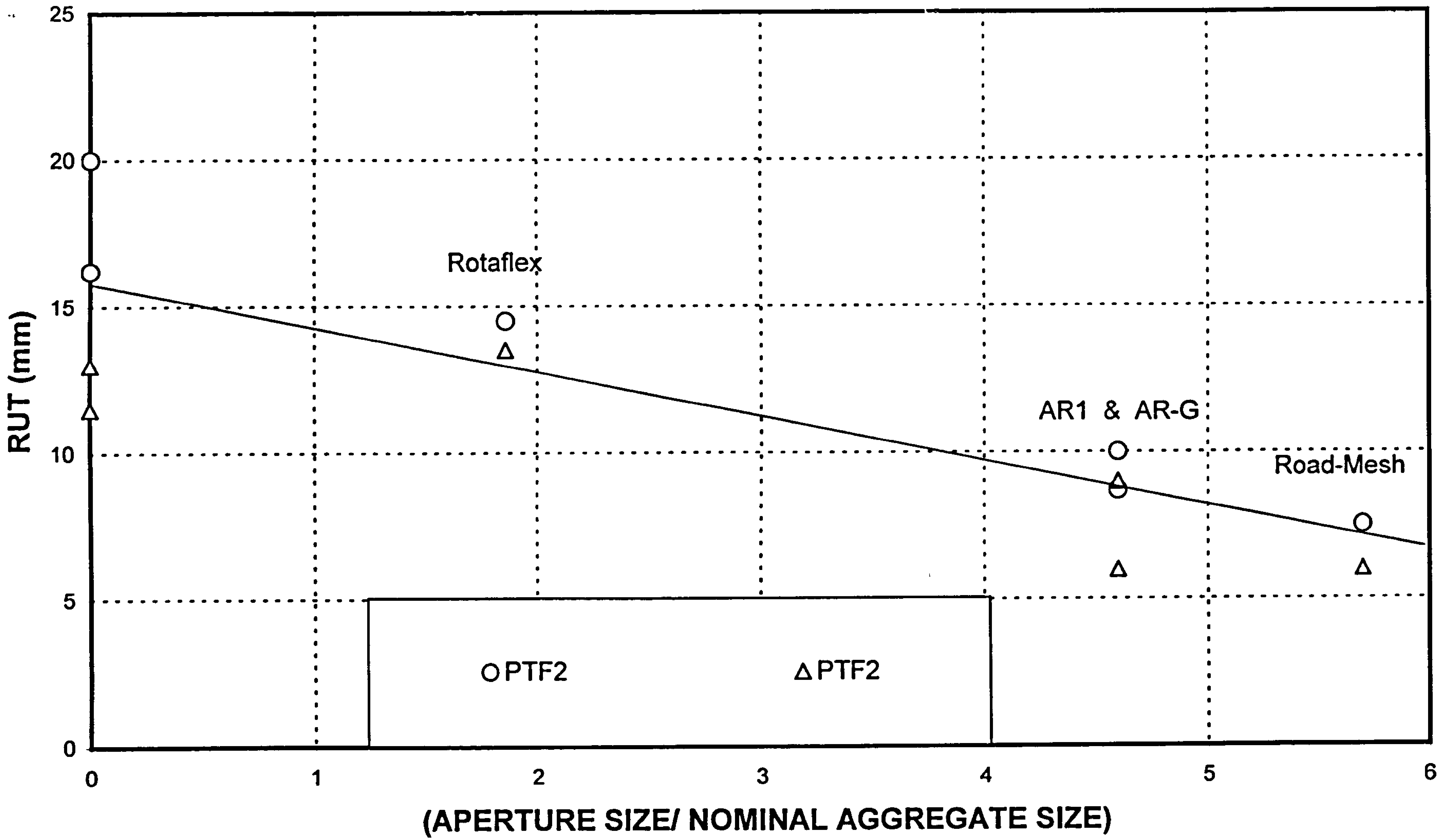


FIGURE 8.16
RUT DEPTH-APERTURE/AGGREGATE RATIO

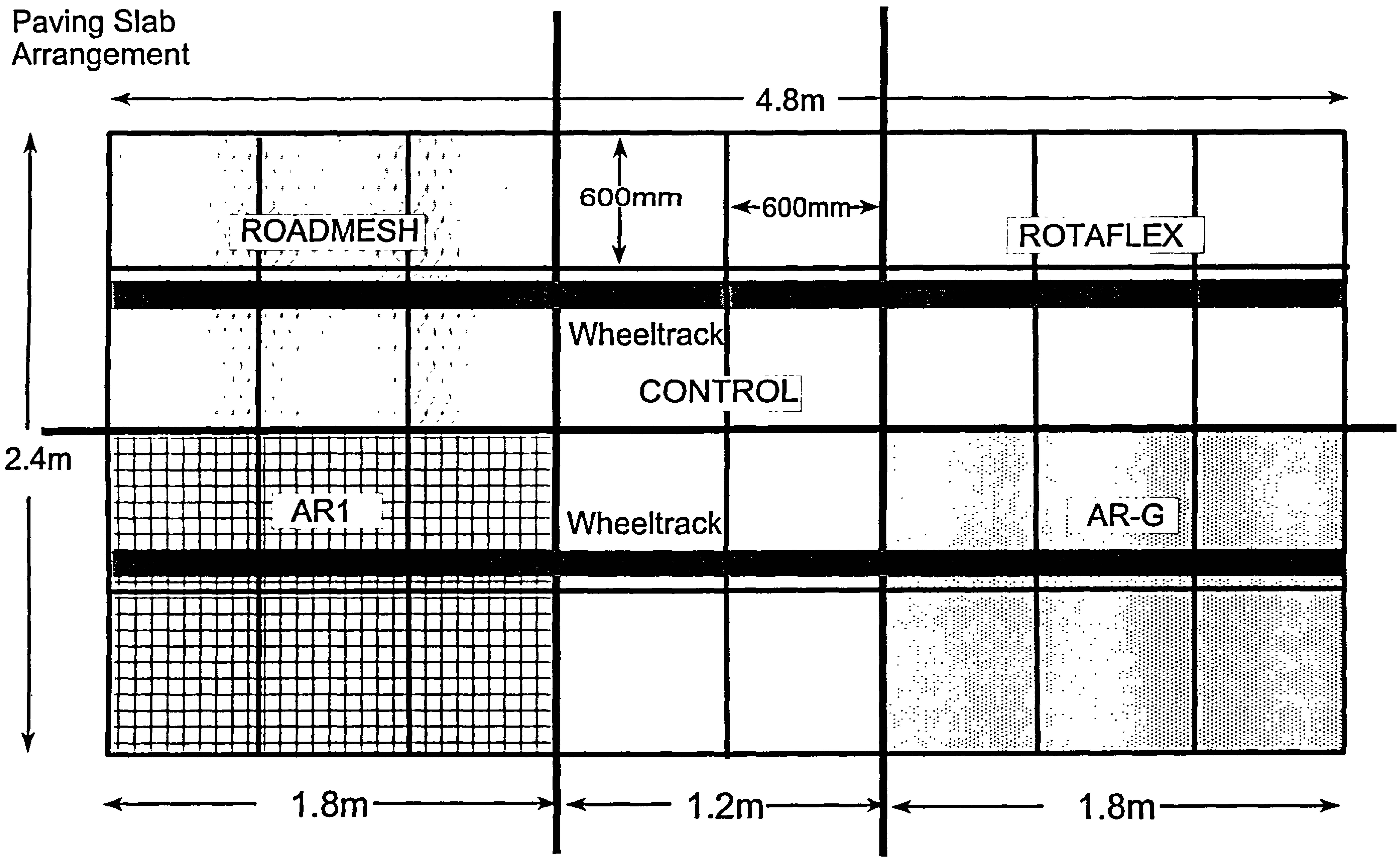


FIGURE 8.17
TEST LAYOUT: PTF3

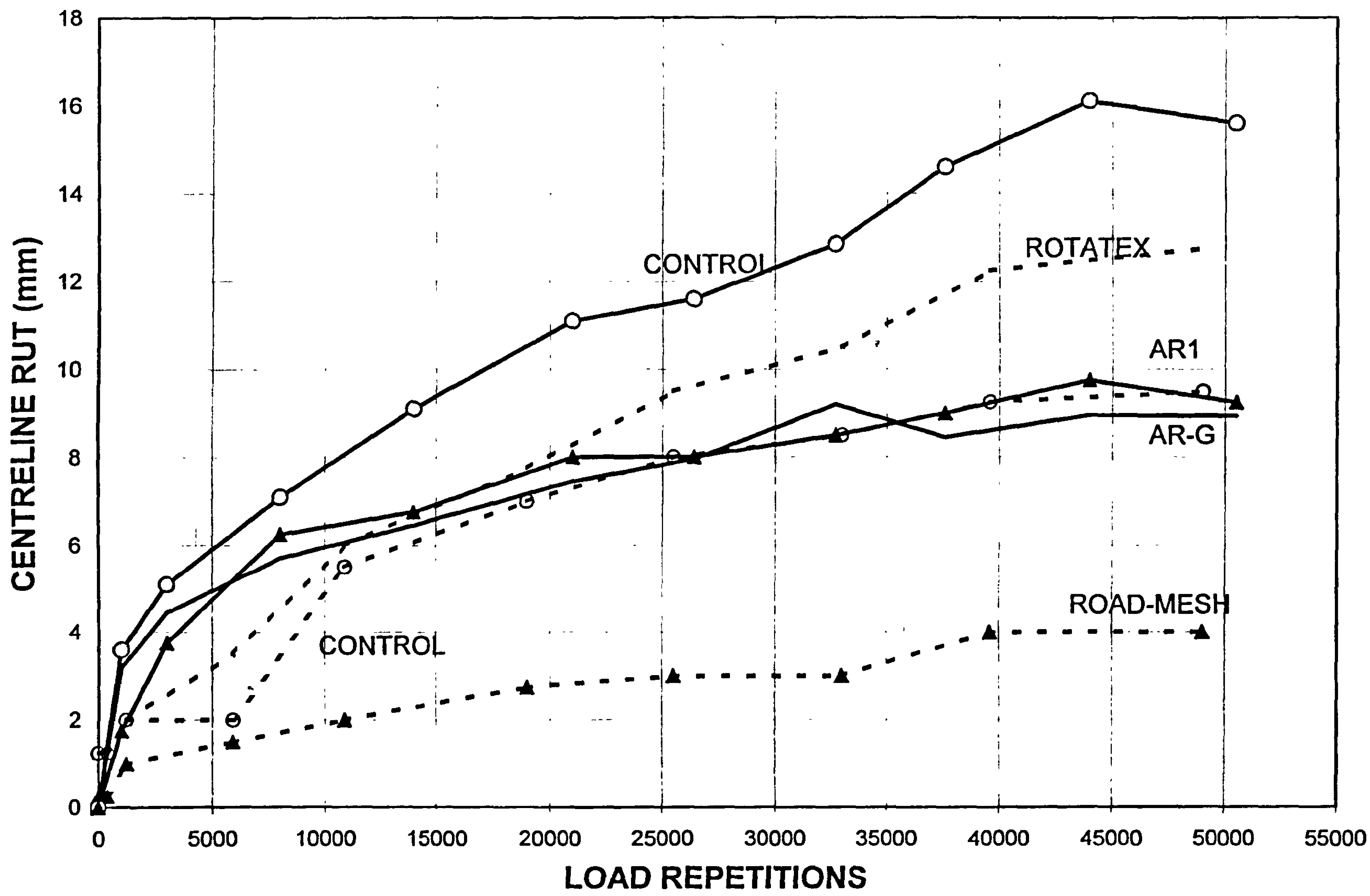


FIGURE 8.18
RUT versus LOAD REPETITIONS: PTF3

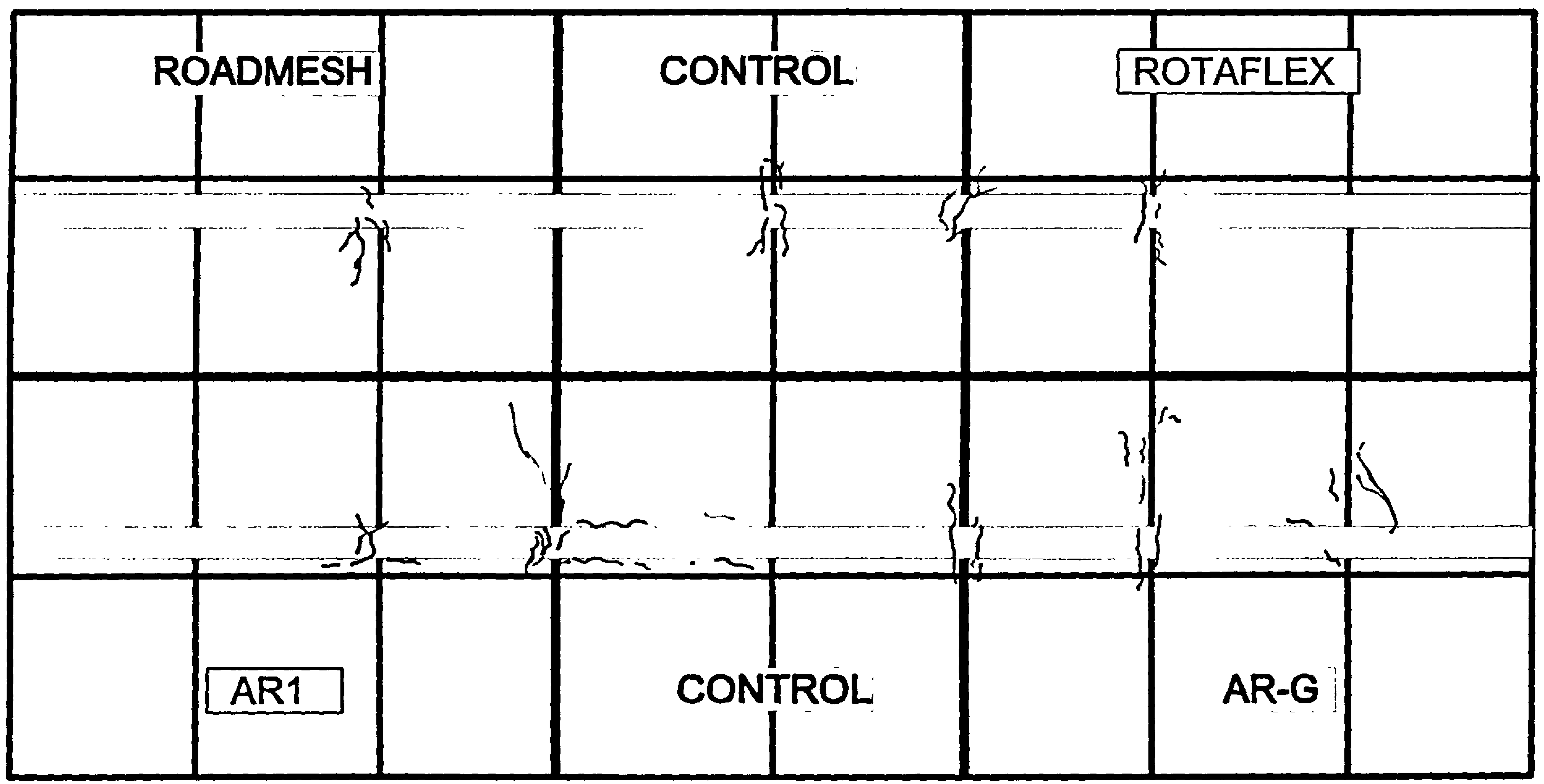


FIGURE 8.19
CRACKING PATTERNS: PTF3

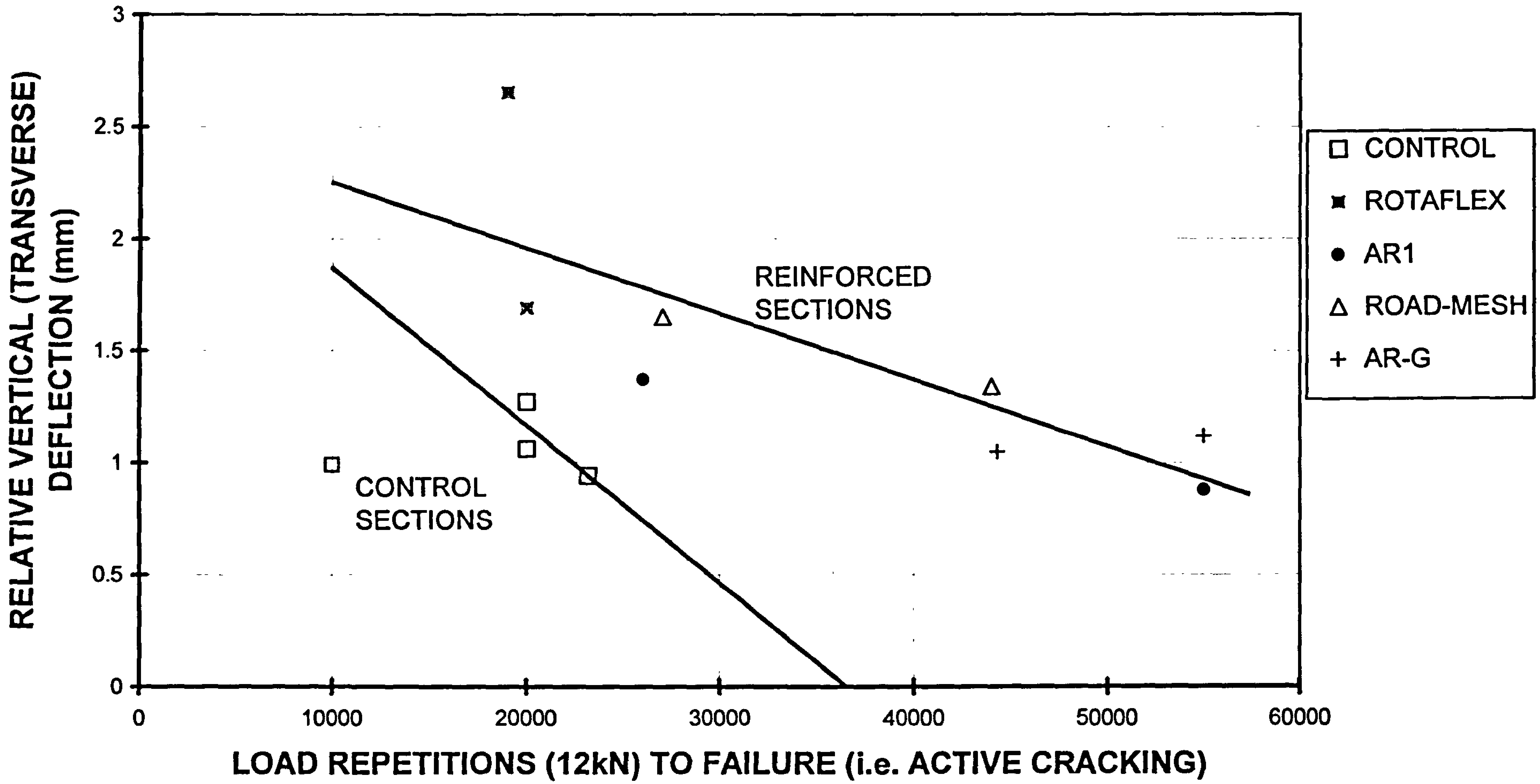


FIGURE 8.20
CRACKING AND DEFLECTION VERSUS TRAFFIC LOADING FOR PTF2 AND PTF3

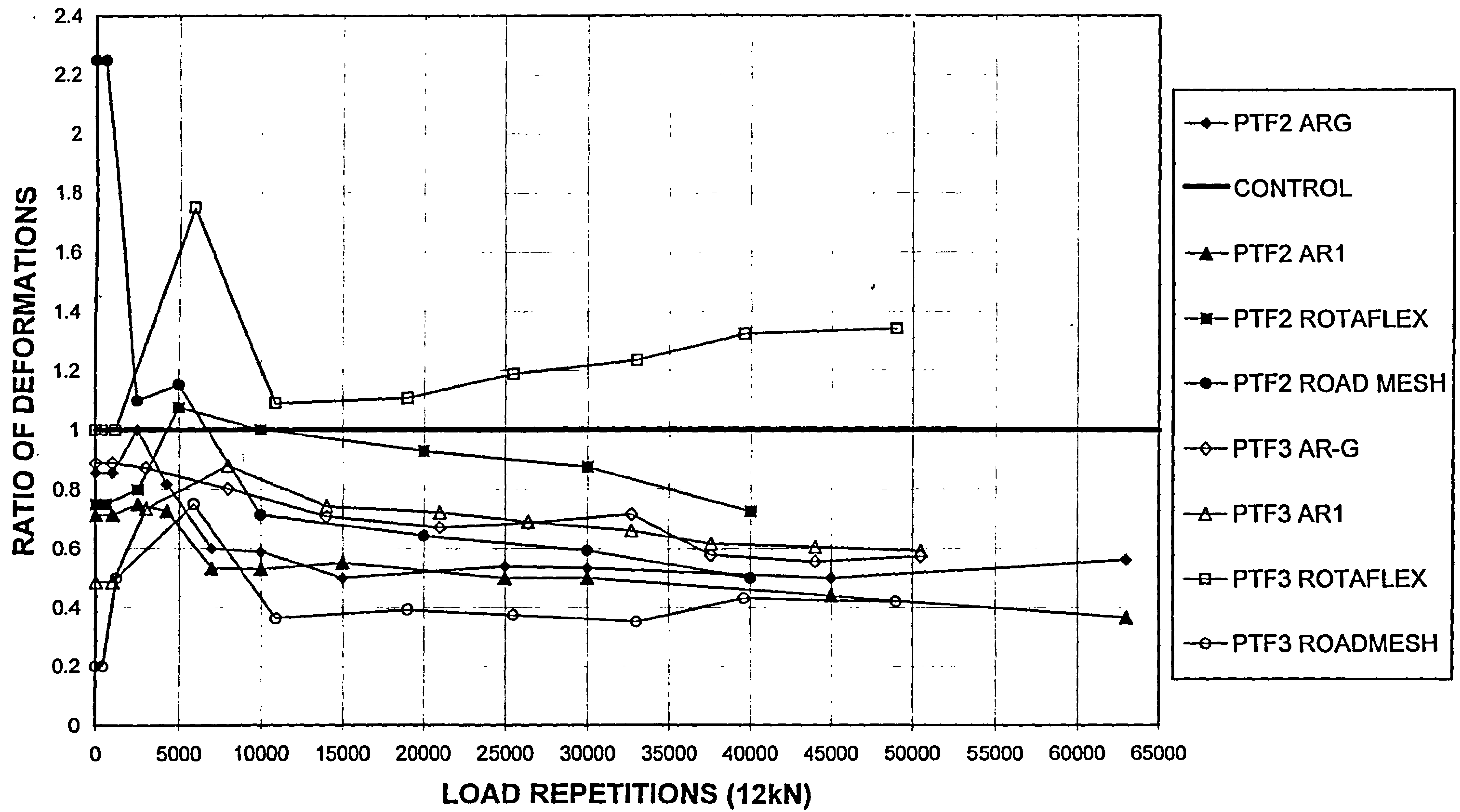


FIGURE 8.21
RATIOS OF PERMANENT DEFORMATION: PTF2 AND PTF3

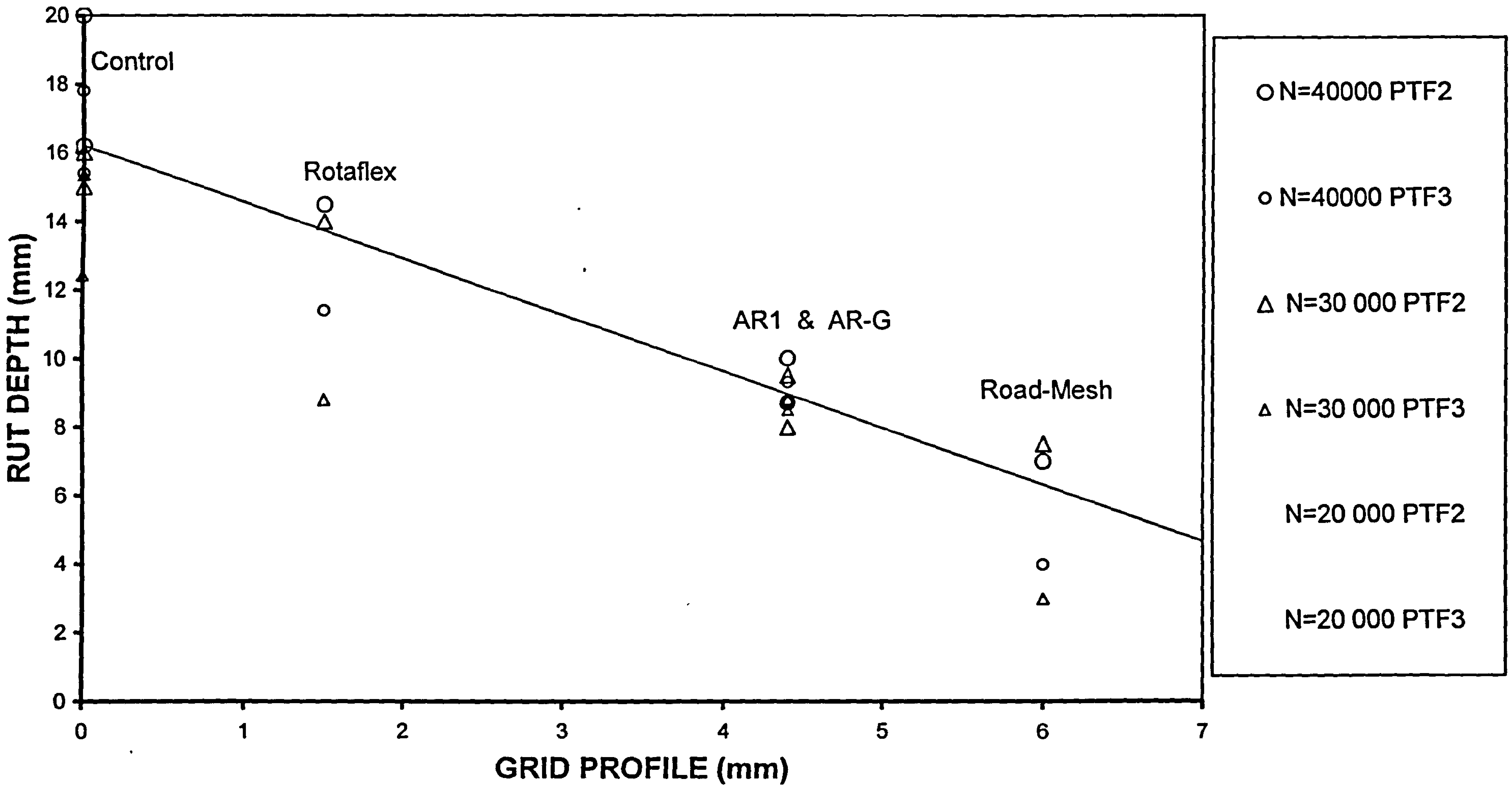


FIGURE 8.22
RUT ANALYSIS - PTF2 AND PTF3: EFFECT OF REINFORCEMENT PROFILE

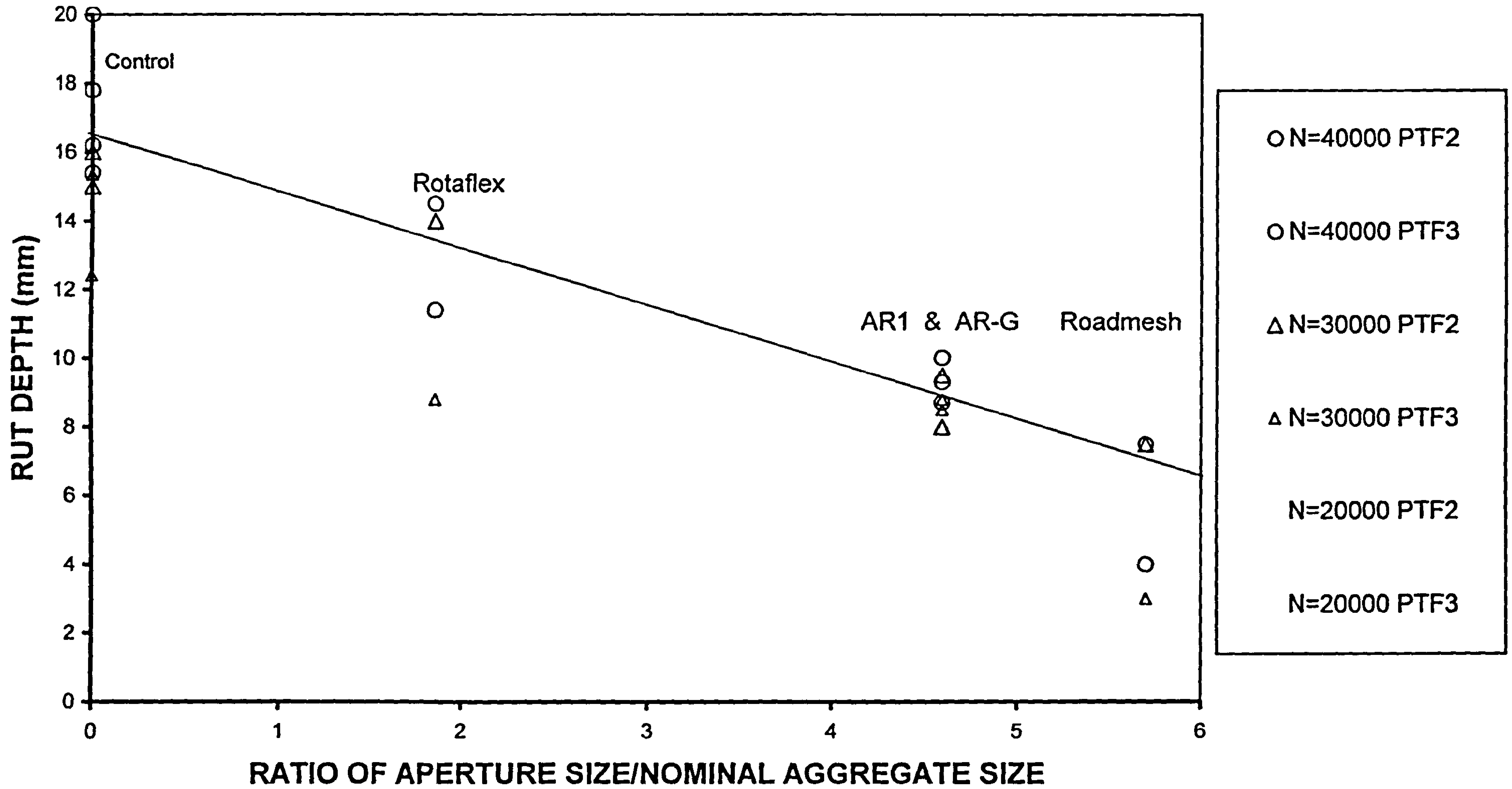
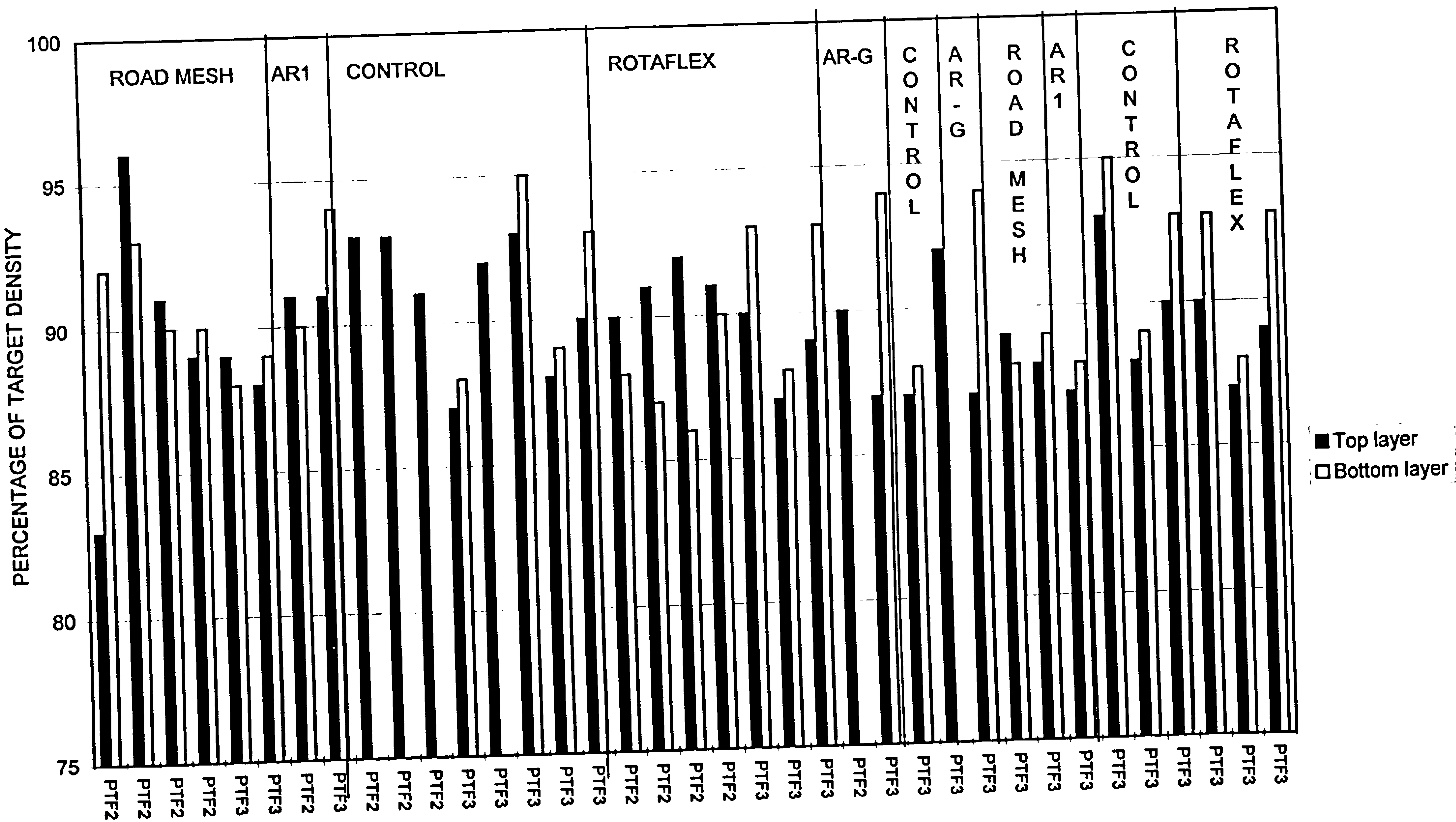


FIGURE 8.23
RUT ANALYSIS - PTF2 AND PTF3:EFFECT OF APERTURE SIZE



DENSITY MEASUREMENTS: CORES FROM PTF2 AND PTF3
FIGURE 8.24

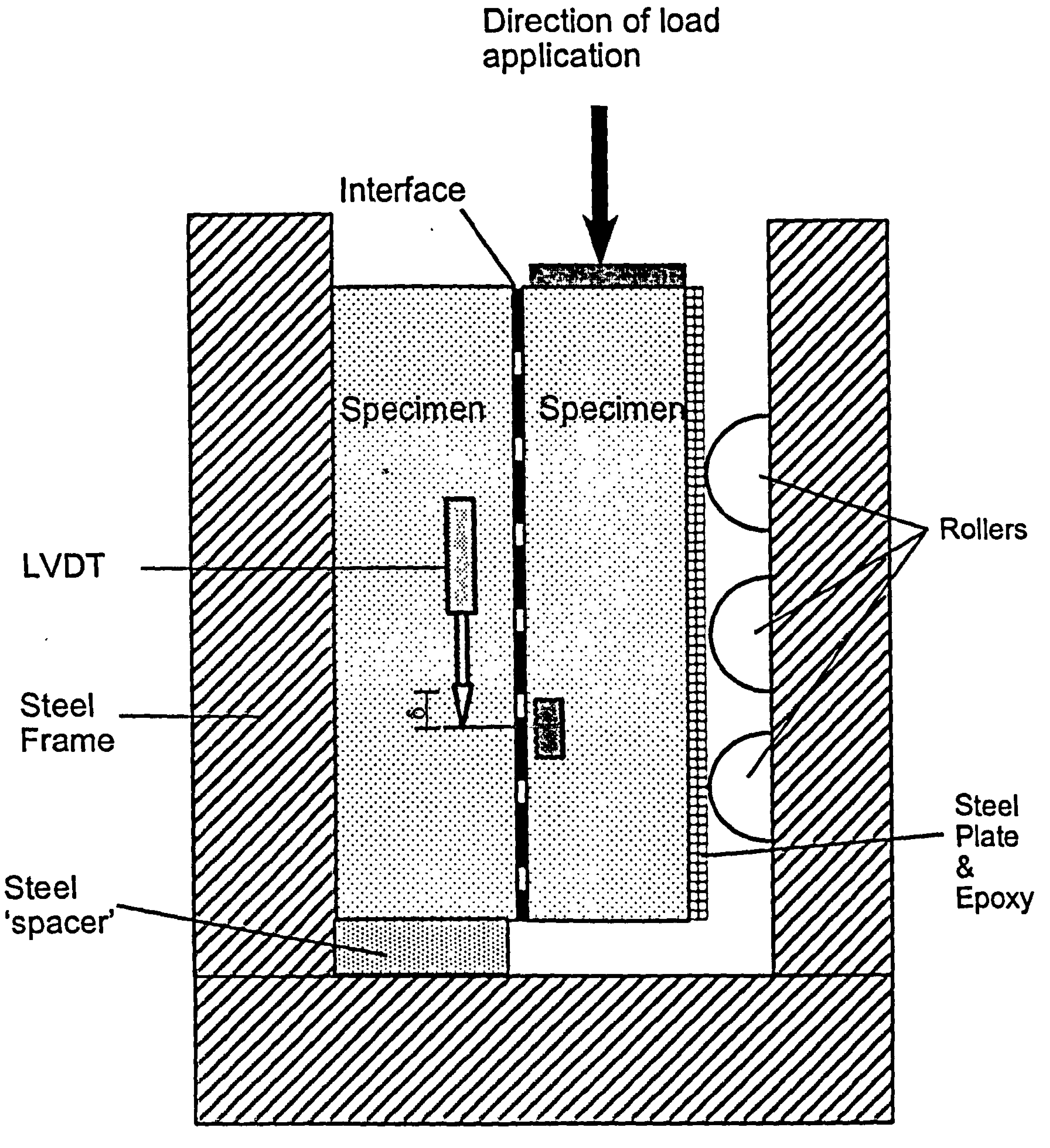


FIGURE 8.25
DIRECT SHEAR APPARATUS

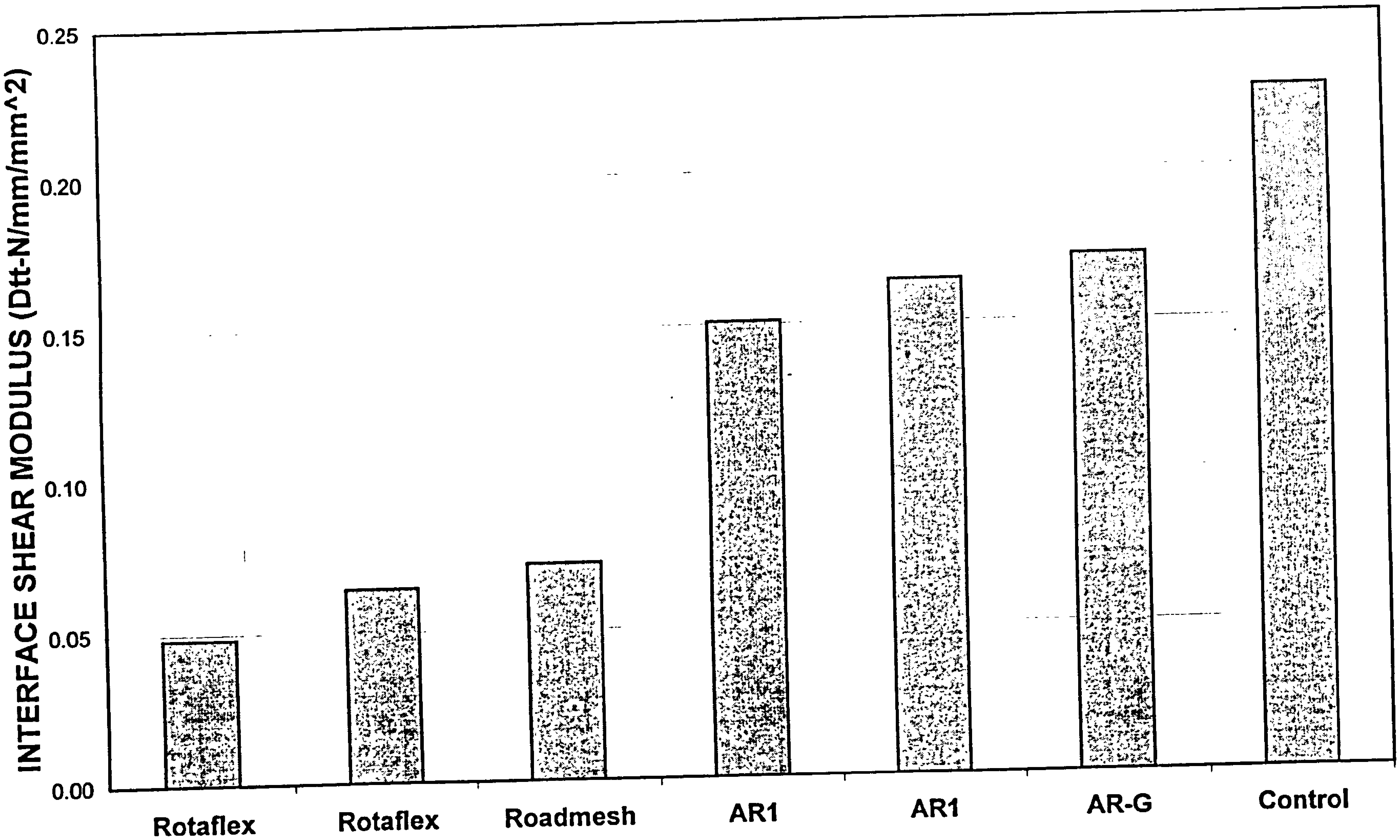


FIGURE 8.26
INTERFACE SHEAR MODULI: PTF3

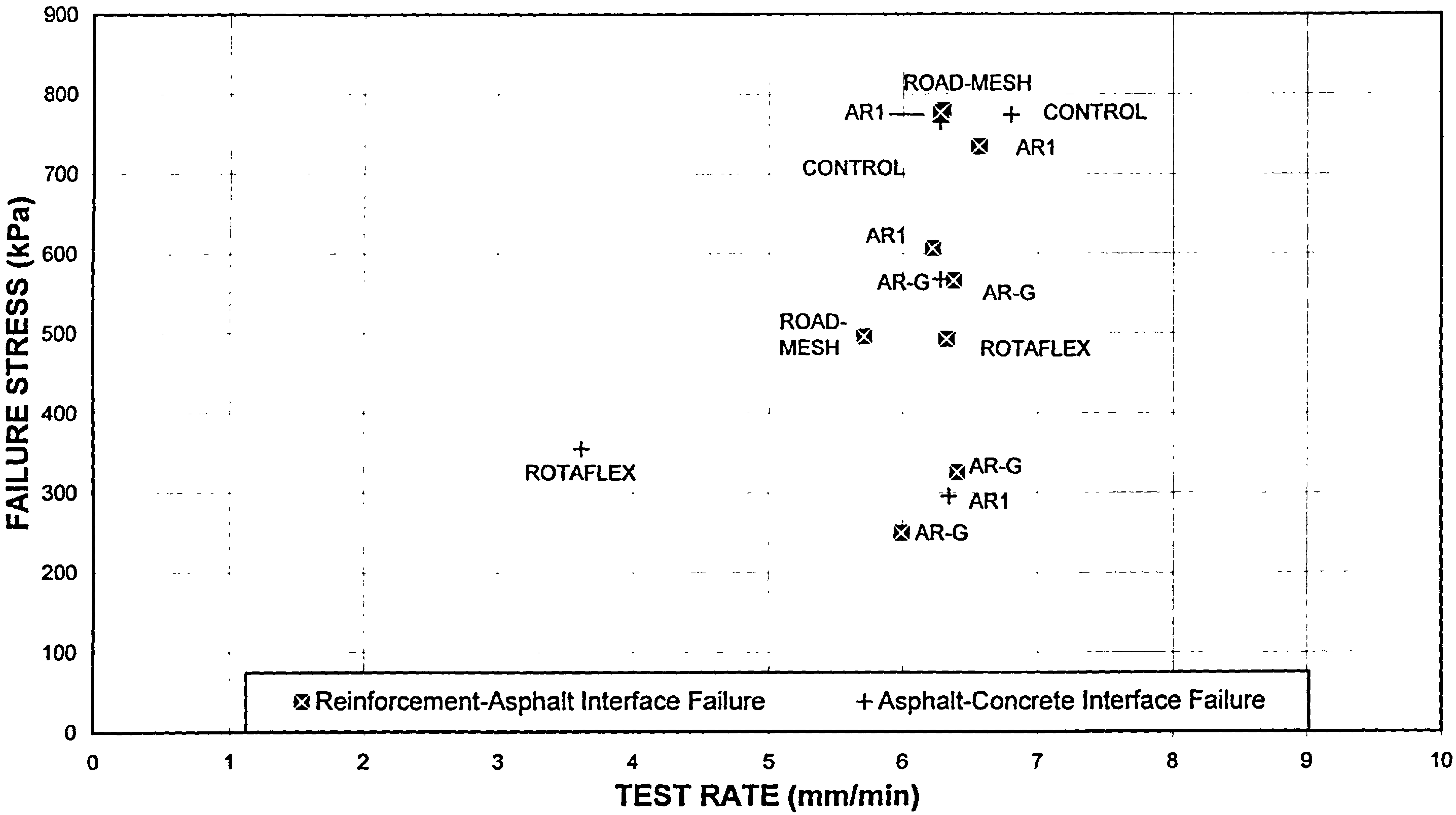


FIGURE 8.27
DIRECT TENSION TEST RESULTS

APPENDIX 8.A

Investigating the use of rubber sheeting with the PTF: Load Testing

To measure the effect of rubber sheeting on paving slab deflection in the PTF, a plate-jacking apparatus was used to test the sub-base with and without a rubber sheet. The apparatus used is shown in Figure 8.A-1 and test positions indicated in Figure 8.A-2.

Tests were carried out as follows:
Loads were applied by hand with the hydraulic jack and released by opening a valve controlling flow of oil into the piston. The loading time was in the order of 4 to 5 seconds, and the unloading within a second. The applied stresses and resultant deflections are given in Table 8.A.1.

Table 8.A.1 Plate Loading Test Results.

Test No.	Applied Stress (kPa)	Deflection (mm)	kPa/mm	Rubber Sheet (Y/N)	Average resilience (kPa/mm)	Reduced resilience due to rubber
A	11.1	0.66	16.8	Y	16.7	A:B=23%
	20	1.27	15.7			
	20	1.53	13.1			
	41.9	2.46	17.0			
B	38.6	1.9	20.3	N	20.3	
C	7.4	0.64	11.5	Y	11.5	C:D=50%
D	39.5	1.72	23.0	N	23.0	
E	36.2	3.44	10.5	Y	8.8	E:F=26%
	34.3	5.45	6.3			
	37.5	3.89	9.6			
F	36.2	3.03	11.9	N	12.0	

It is noted that in the positions where rubber sheeting was used, there was a decrease in 'resilience' of between 23 and 50%, with the largest decrease being found in the unreinforced (control) area. This suggests that rubber sheeting increases deflections and therefore will be useful for enhancing the relative deflections induced between adjacent concrete slabs.

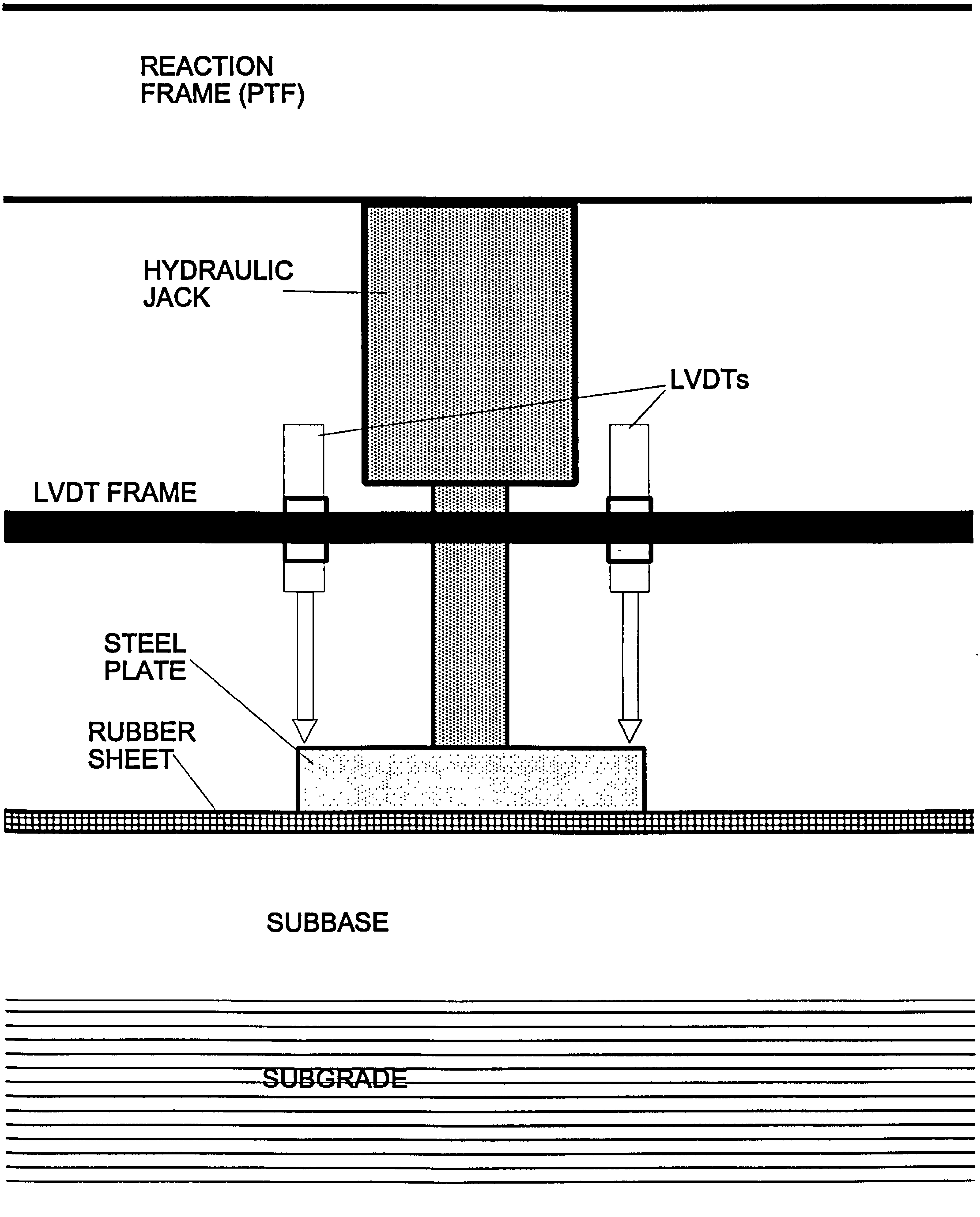


FIGURE 8.A-1
PLATE-JACK TEST CONFIGURATION

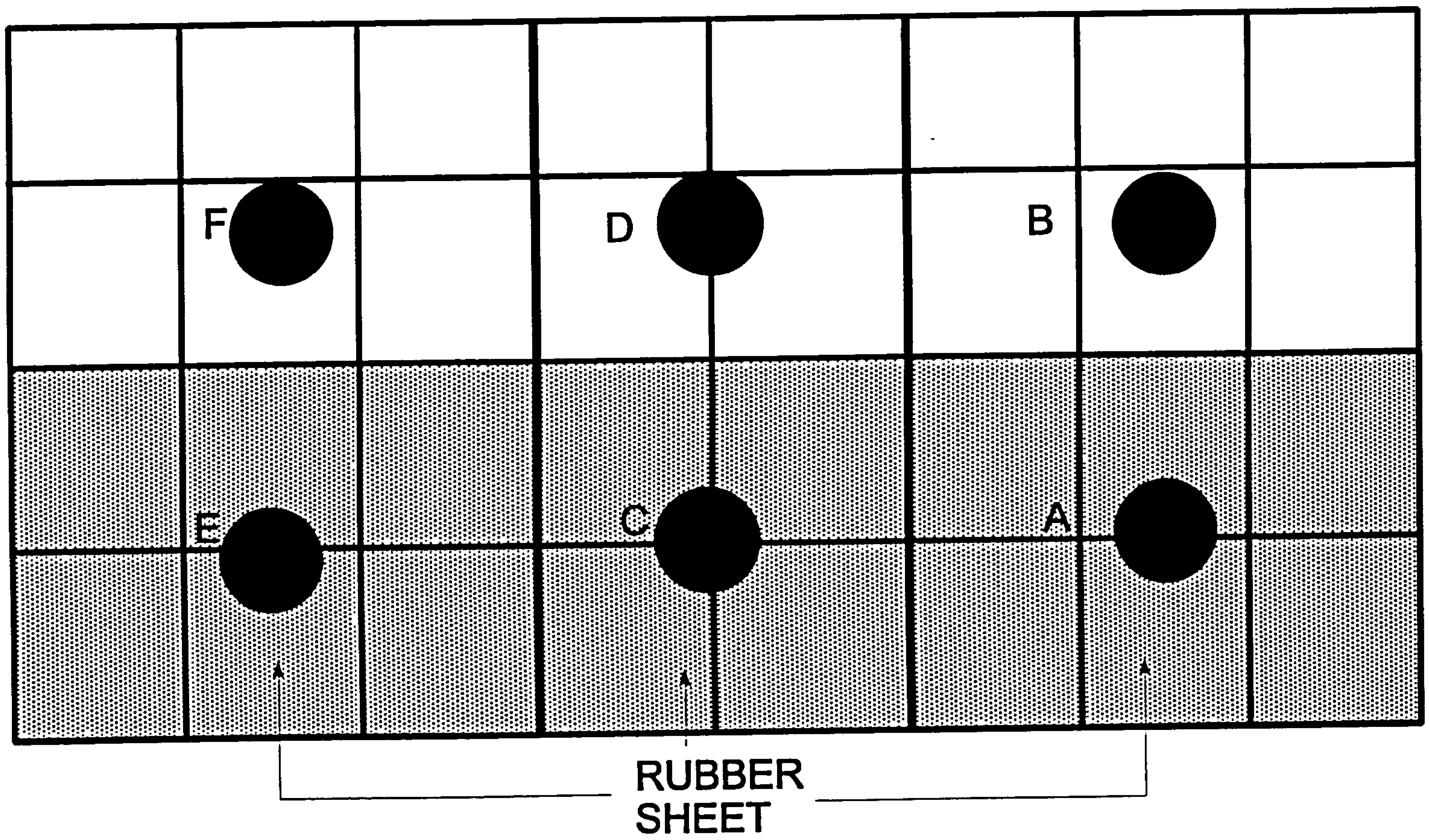


FIGURE 8.A-2
PLATE TEST LAYOUT: PTF3

APPENDIX 8.B

Instrumentation used in the PTF Tests

Instrumentation was installed within the pavement to record vertical earth pressure, vertical and horizontal movement of the concrete slabs, and horizontal strain in the DBM surfacing. Each of the instruments is now briefly described.

8.B (i) Earth pressure cells

General description
The earth pressure cells placed in the subgrade for test PTF2, are described by Brown [8.10]. The pressure cell uses a strain-gauged diaphragm to register applied stress and was calibrated using a by placing the cell between steel plates and applying water pressure through a rubber membrane.

Installation
Three pressure cells were placed in a shallow recess on the top of the subgrade with the top of the pressure cell level with the top of the subgrade at the positions shown in Figure 8.B-1. The positions chosen were in the control (unreinforced) sections to measure a 'maximum' likely stress under the pavement structure. To connect the cells to instruments used for monitoring, cables were taken across the top of the ubgrade and out of the test pit through ducting.

Performance
Readings of earth pressure were taken during trafficking in PTF2 on the ROTAFLEX/Road-Mesh wheelpath and increased from 27 to approximately 60kPa. The most noticeable pressure increase occurred between 20000 and 30000 load repetitions, which coincided with the incidence of active surface cracking in the Control section.

Pressure cells were installed in test sections PTF3 and 4, but, through malfunctioning of the equipment used to read the cells, and /or damage to the cables, they could not be used.

8.B (ii) Strain Coils.

A brief description of the strain coils used is given below. More details are given in the paper by Dawson and Little [8.11].

General description
Strains can be measured with inductive strain coils when the coils are accurately placed within a common electromagnetic field and an alternating current is applied to one of the coils, thus inducing an electromagnetic field in the other. The magnitude of the induced current is related to the distance between the coils and so, can be calibrated by moving the coils apart whilst recording the change in induced voltage. The layout of the strain coils is shown in Figures 8.B-1.

Installation
The coils were carefully placed to give readings in the middle of their range and fastened to the top of the slabs with bitumen, after which their initial readings were taken.

Performance
Dawson and Little [8.11], noted that one of the problems with using this type of device is that the coil can pick up electromagnetic noise from sources other than that of the other coil. This was found to be the case with PTF Test 2 where the electronic disturbance was attributed to metal in the PTF wheel, and induced a change in voltage much greater than that induced by slab movement, and so masked the effect of slab movement. Strain coils were therefore not used in Test PTF3.

Although in this case the strain coils were not suited to the test configuration, they have been used successfully in other situations, such as in a haul road in Scotland (Dawson and Little [8.11]) where the coils were deeper in the pavement.

8.B.(iii) DEMEC Gauges (for measurement of asphalt surface strain)

Description
To measure asphalt surface strains over the slab joints, a DEMEC gauge was used to measure movements between across small metal disks (or 'pips') stuck to the pavement in the positions shown in Figure 8.B-1. Due to the proximity of the pips to the wheel, the DEMEC gauge could not be held in position by hand, or read by eye. An LVDT was therefore used in place of a dial gauge, as seen in Figure 8.9.

Performance
Almost without exception, the DEMEC pips were positioned where either (a) no

surface movement could be detected or (b) cracks formed close to, but not between, pips and led to no definite measurements being recorded. It is recommended that for future tests of this nature, an alternative approach to measuring surface strains be used. The use of a more sensitive LVDT should be considered with more pips placed in a localised area to ensure that better measurements are obtained.

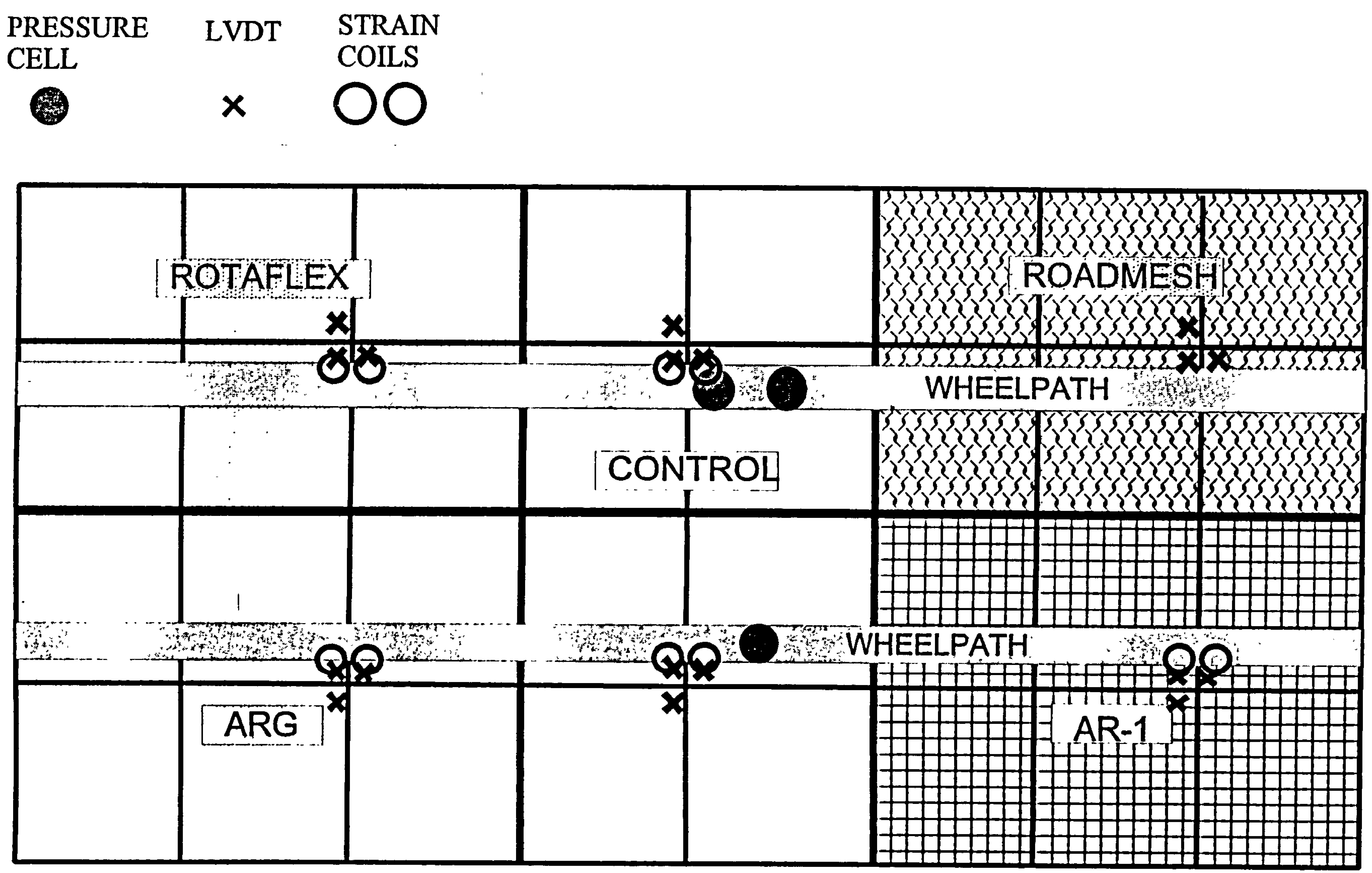


FIGURE 8.B-1
POSITION OF INSTRUMENTATION: PTF2

APPENDIX 8.C

Measurement of PTF Foundation Properties

8.C.1 Dynamic Cone Penetrometer (DCP) Testing

Apparatus

The Dynamic Cone Penetrometer (DCP) is an instrument designed for in-situ assessment of unbound materials to a depth of 800mm, although greater depths are possible with an extension. Figure 8.C-1 shows the apparatus in a typical test set-up.

The DCP can be used to assess strength by correlation with the Californian Bearing Ratio (CBR), and layer thicknesses which can be defined by penetration rates. This implies that once penetration rates are correlated with a material type (by coring or a test pit perhaps), the DCP can be used to quickly assess material across a site without having to take a large number of cores or open test pits.

Tests are relatively quick to perform, and normally, simple to analyse as typically, only the depth of penetration versus the number of hammer drops are required. When plotted against depth, these measurements help to identify the effective thickness of pavement layers which are then used in design.

Test description

To carry out a DCP test, the weight is raised and dropped onto the anvil, thus driving the cone into the ground. The depth of penetration is measured a selected number of blows, depending on the nature of the material being tested. The apparatus is kept vertical during the test which normally requires two people to carry out the test.

Test positions

The subgrade positions tested are shown in Figure 8.C-2, and are referenced to slab positions, as given in Table 8.C.1.

Table 8.C.1. DCP Test Positions

DCP Test No.	Paving slab number	PTF Test 2 section type
1	6	AR-G
2	13	Control
3	25	AR1
4	7	Rotaflex
5	15	Control
6	27	Roadmesh

Test readings & Figures

Test results are illustrated in Figure 8.C-3 and show the pavement to be fairly similar in the top 350mm or so of the subgrade over the whole test area. Also, a soft layer between 200 and 300mm depth is easily recognisable for each of the sections tested. To investigate the reason for the apparent differences in stiffness, samples of material were retrieved and used for determining moisture contents. The results are seen in Table 8.C.2. Material in the subgrade is seen to have around 2% more moisture at around 250mm depth that at 150mm and 350mm. Using the relationships developed by Cheung [8.12], the increased moisture content suggests that the subgrade would have a strength of around 3% CBR, compared to 5% CBR of the drier material. Whereas samples of material were only taken from one area, due to the 'uniform treatment' of the whole test area during construction, and the apparent homogeneity of the subgrade material (see Figure 8.C-3), it seems likely that similar conditions exist across the (relatively small) PTF test area. The reduction in CBR appears to agree with the increase in moisture content, i.e. a reduction in strength with an increase in moisture content.

Table 8.C.2 Subgrade Moisture Content.

Position (Slab no.)	Subgrade Depth (mm)	Moisture Content (%)
18 (Control section)	0.1	16
	0.22	17.8
	0.3	15.5

8.C.2 Soil Cone Penetrometer Testing

Test description

The soil cone penetrometer is a simple device used to obtain measures of soil resistance as it is pushed into the subgrade. Readings are taken of the force required for the penetration, and with correlation to other test results can be related to strength. A schematic of the apparatus is given in Figure 8.C-4.

Test results

Test results are given in Figure 8.C-5 and show the maximum force for penetration of the top 75mm of the subgrade, plotted against slab number. Similarly to the DCP test results, there is a general trend showing the subgrade to stiffen towards the higher slab numbers, which, for PTF2 corresponds to the AR1 and Roadmesh test sections, and for PTF4, the Rotaflex and AR-G sections..

Overall findings of the subgrade investigation

a) The subgrade has a surface 'crust' that overlays a softer layer between around 200 to 300mm.

b) Material below 300mm increases in strength.

c) The subgrade appears quite uniform across the PTF test area.

d) In general, the strength of the top of the subgrade increases towards the area of higher slab numbers, which after construction were the areas reinforced with AR1 and Roadmesh grids.

8.C.3 Clegg Hammer Testing

Test description

The device consists of a 4.5kg mass falling through 450mm in a 50mm diameter guide tube (see Figure 8.C-6). An internal accelerometer is mounted in the hammer and gives an output that is read from a display on the box housing the electronics. The hammer is dropped successively and the fourth reading taken and expressed as the Clegg Impact Value (CIV). This reading can be related to a CBR value and possibly a measure of stiffness. To relate values from the Clegg test to engineering material parameters with any certainty, direct correlation is required. This is due to the (typically) variable nature of road building materials and the small area of material tested. However, the Clegg hammer was used in this instance to help with checks on consistency, so an absolute value of stiffness or strength was not required.

Test positions

Results of the testing are shown in Figures 8.C-7. Positions are referenced in terms of slab positions.

8.C.4 Nuclear Density Gauge Testing.

Apparatus

The apparatus used was a Campbell Pacific MC2 Porta Probe Gauge. Calibration was carried out using the synthetic reference material supplied with the gauge. Readings were taken on a previously-used sub-base material and compared with the earlier test results to ensure consistency.

To ensure good contact with the sub-base the gauge was placed on a thin bed of fine sand.

Measured densities are given in Tables 8.C.3 and 8.C.4.

Table 8.C.3 Nuclear Density Readings on PTF2 Sub-base

Position (paving slab no. -see Figure 8.4a)	Section Type	Test Reading (kg/m^3)	Moisture Content (%)	Corrected (dry) Reading (kg/m^3)
6	AR-G	2260	5.8	2134
10	AR-G	2240	5.9	2113
14-18	Control	2291	6.1	2158
22	AR1	2302	5.9	2172
26	AR1	2313	6.4	2187
7	Rotaflex	2259	6.2	2127
11	Rotaflex	2270	6.0	2130
15-19	Control	2323	6.0	2190
23	Roadmesh	2345	5.7	2217
27	Roadmesh	2213	6.1	2179

Table 8.C.4. Nuclear Density Readings on PTF3 and4 Sub-base

Position (paving slab no. -see Figure 8.4a)	Section Type	Test Reading (kg/m^3)	Moisture Content (%)	Corrected (dry) Reading (kg/m^3)
5	AR-G	2371	6.0	2237
25	AR1	2284	5.9	2157
13	Control	2339	5.8	2209
20	Control	2328	6.0	2198
28	Roadmesh	2275	6.1	2148
7	Rotaflex	2360	5.9	2228

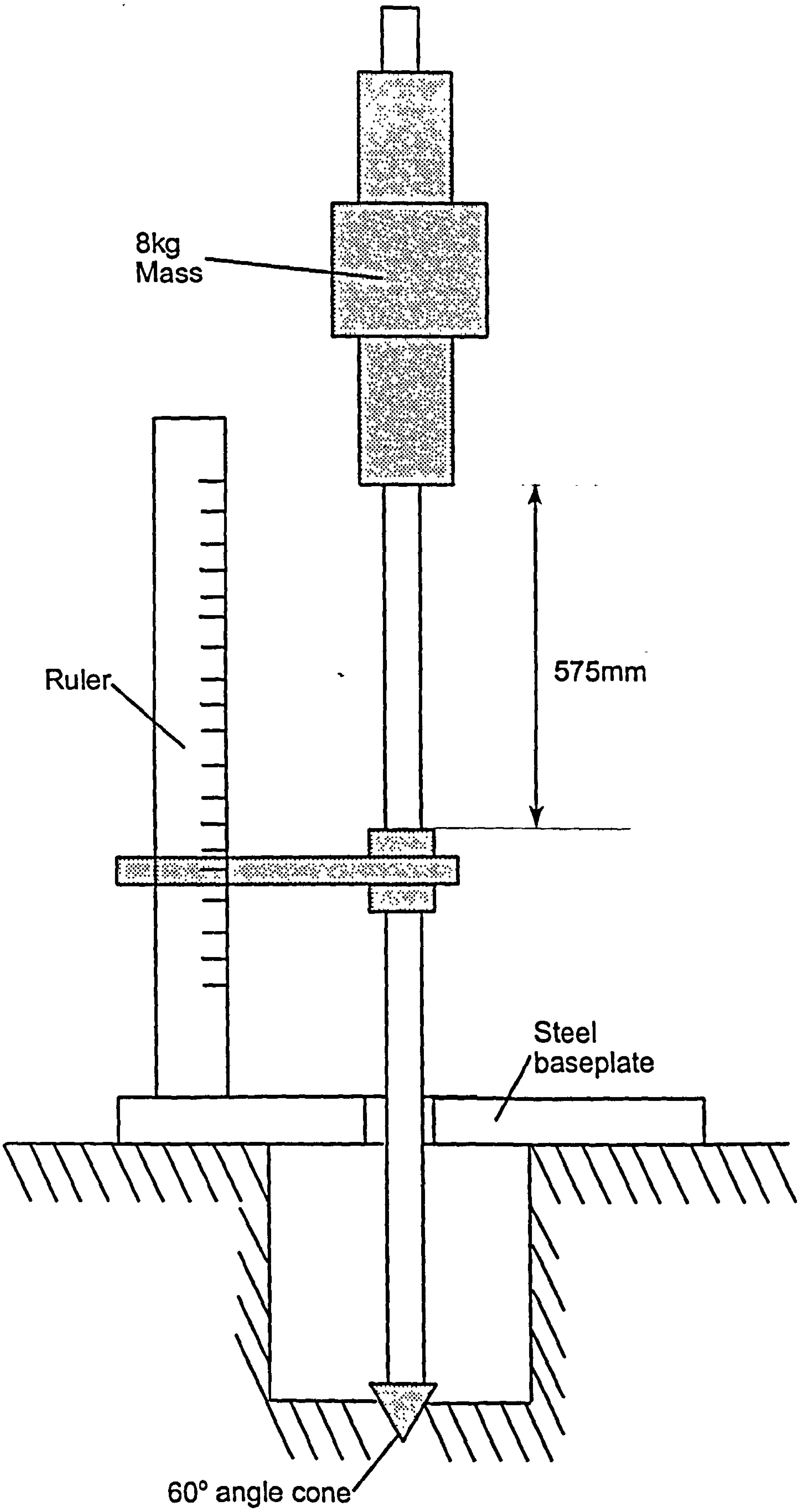


FIGURE 8.C-1
DYNAMIC CONE PENETROMETER

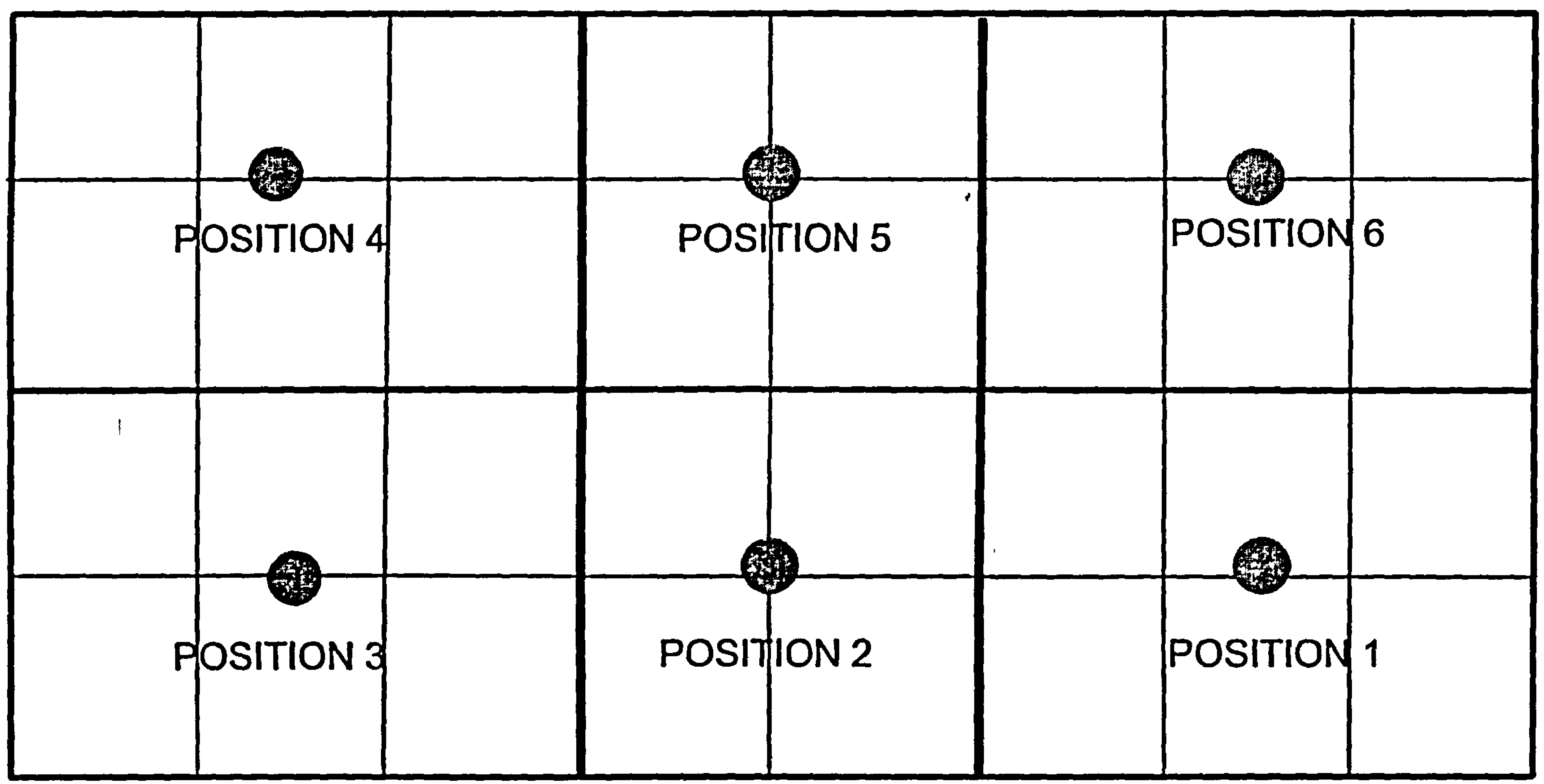


FIGURE 8.C-2
DCP TEST LAYOUT

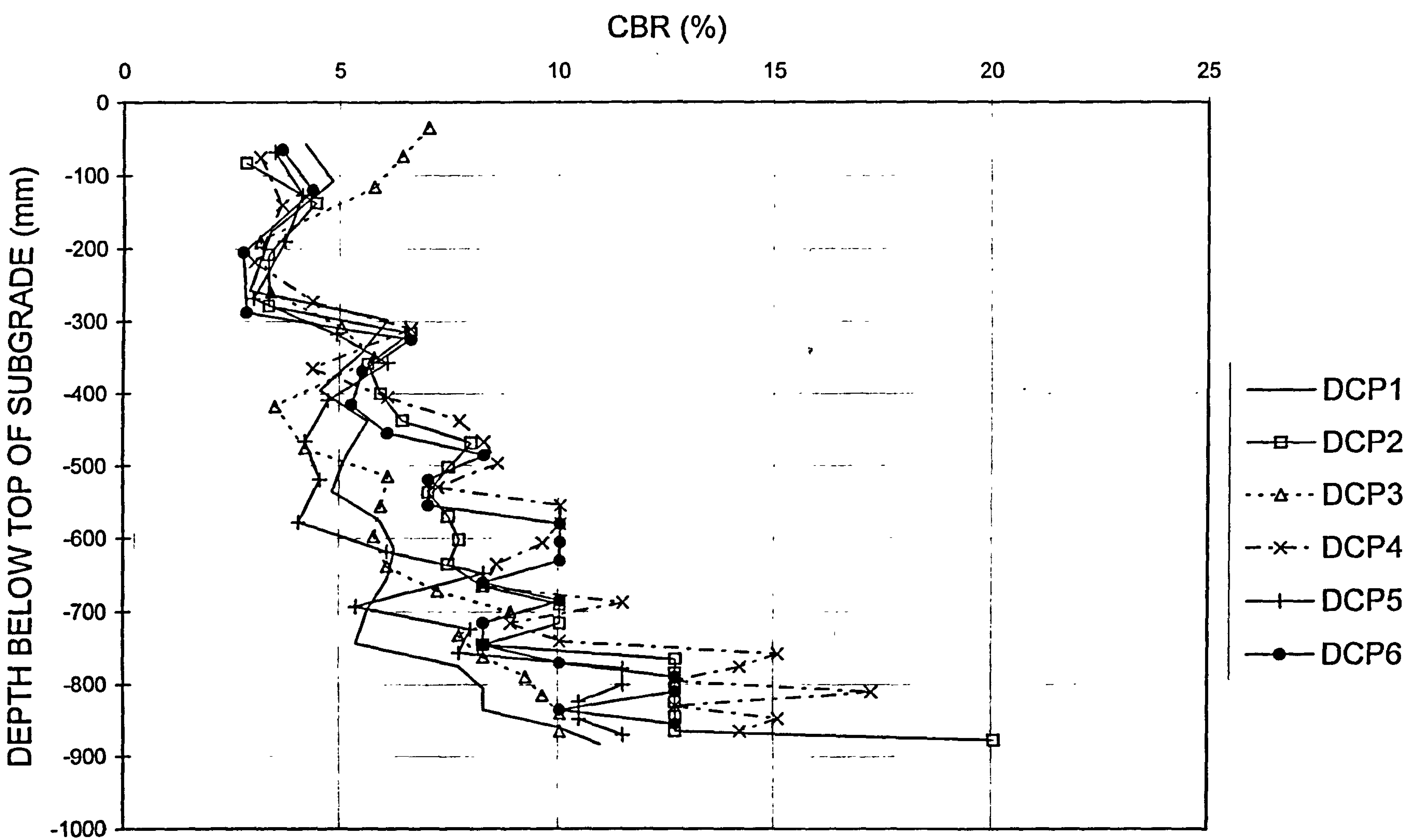


FIGURE 8.C-3
DYNAMIC CONE PENETROMETER TEST RESULTS: PTF2 SUBGRADE

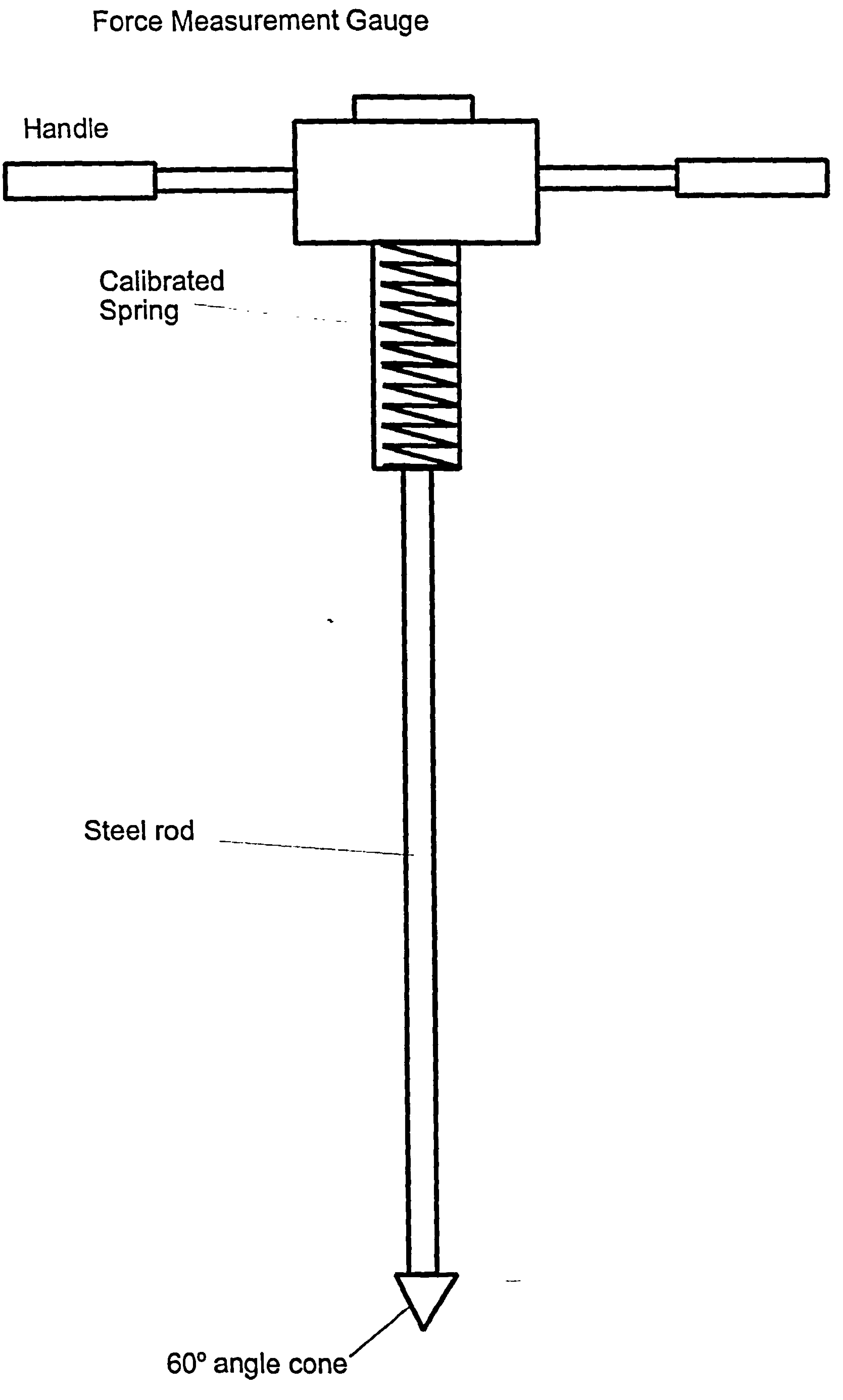


FIGURE 8.C-4
SOIL CONE PENETROMETER

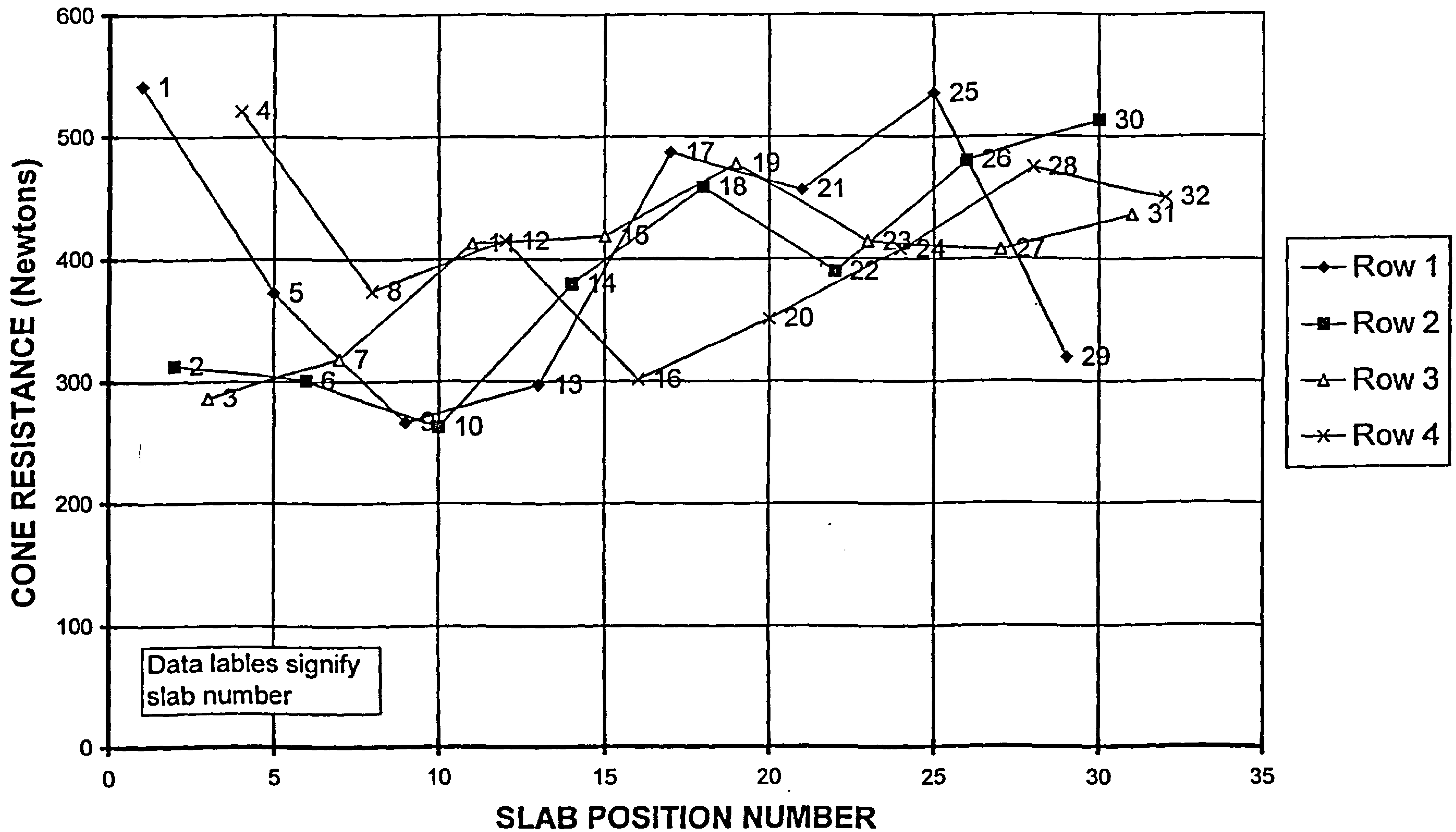


FIGURE 8.C-5
PTF2: SUBGRADE RESISTANCE VALUE versus SLAB POSITION

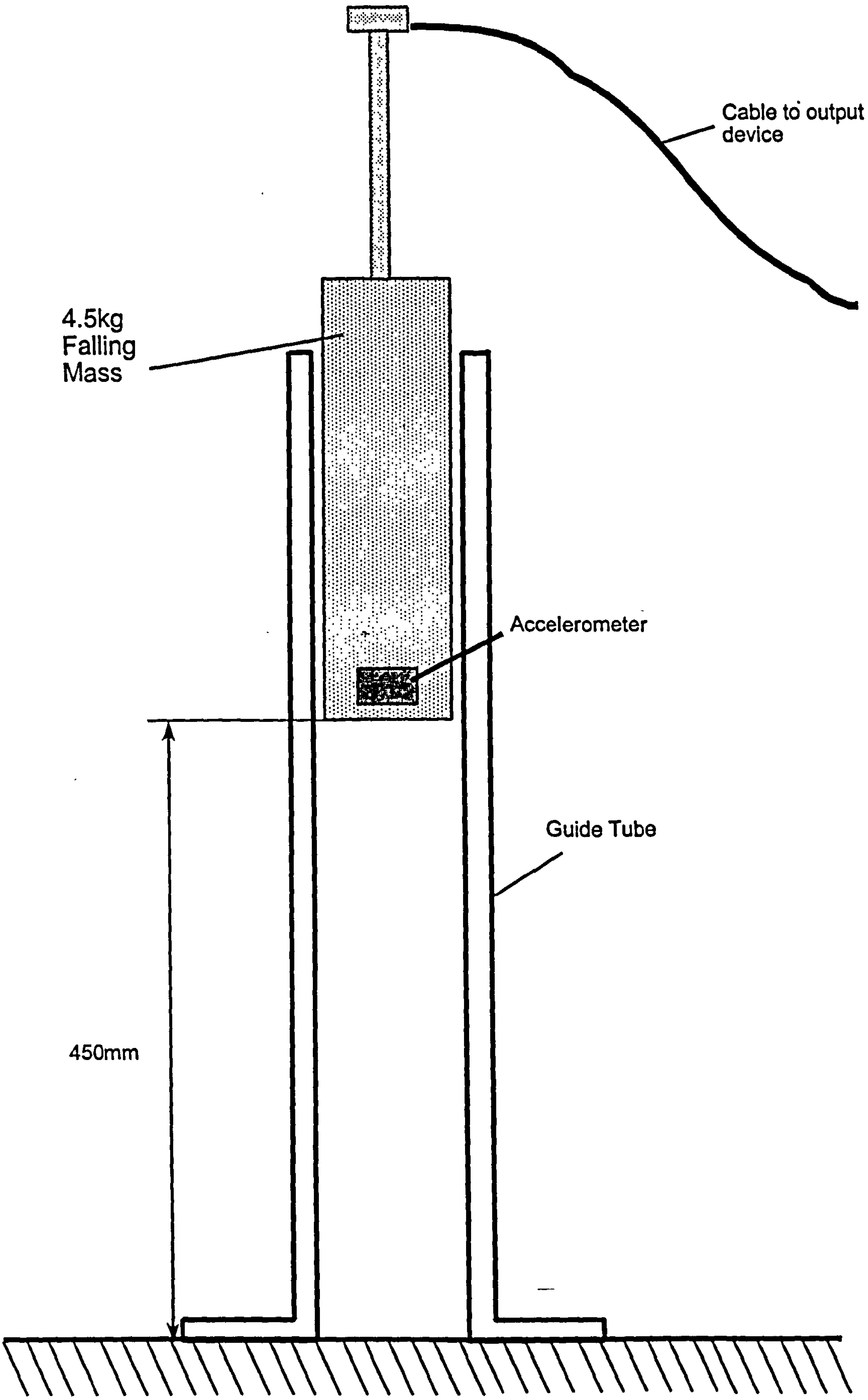


FIGURE 8.C-6
CLEGG HAMMER

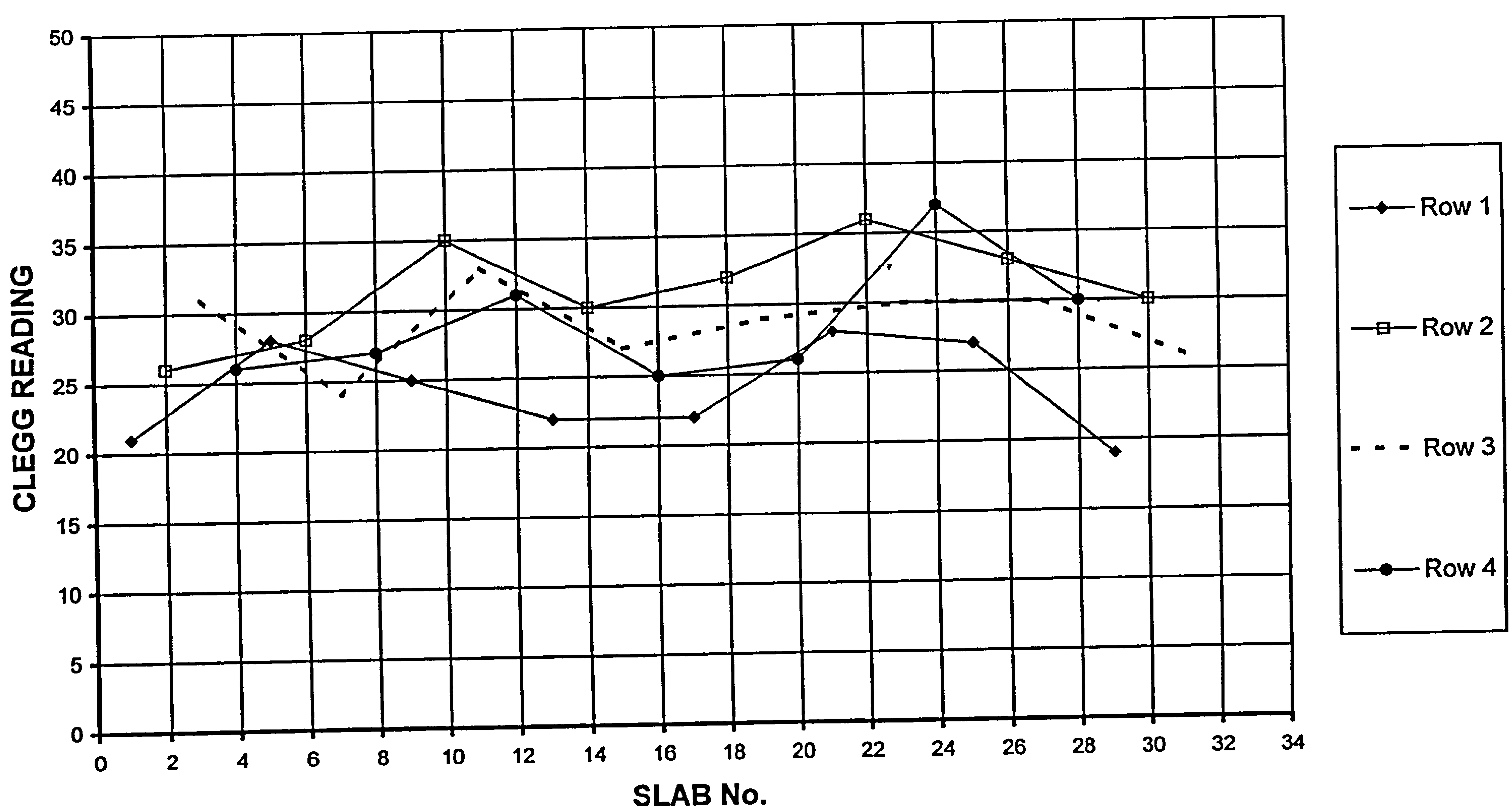


FIGURE 8.C-7
PTF2 SUB-BASE: CLEGG HAMMER READINGS

APPENDIX 8.D

Pavement Test Facility: Test PTF1

Test Results.

Transverse joint reflection cracking test

Plots of deflection and permanent deformation with repetitions are given in Figures 8.D-1 and 8.D-2 and show deflections measured across the transverse joints and permanent deformation to be larger for the control section than for the AR1-reinforced section. The reason for the large drop in relative displacements measured in the control section at around 6000 repetitions is uncertain but is thought due to the slabs deflecting more or less simultaneously rather than a 'true' reduction in deflection. This is borne out by observation of the movement of the surfacing which did not appear to reduce throughout the test, but indeed, seemed to continue to increase. Measurements of permanent deflection for the control section (Figure 8.D-2) were stopped at around 20000 repetitions, when the lateral movement of the asphalt ("shoving") caused the section to fail. Measurements on the reinforced section were however continued to around 70000 repetitions and showed rutting on the reinforced section to gradually increase with repetitions. Figure 8.D-3 shows a significant difference in transverse profiles of reinforced and unreinforced sections, and particularly a reduced deformation with the AR1 section. Surface cracks did not show a very distinct pattern over the position of the joints in either section but were more concentrated in the unreinforced section.

Longitudinal joint reflection cracking test

Figures 8.D-4 and 8.D-5 show the change in deflection measurements with load and repetitions, and the increase of rut with repetitions. The most obvious feature of the graph is that with trafficking, relative deflections measured on the AR1-reinforced section reduced whereas those for the unreinforced section increased. A possible reason for this is that initially, the asphalt laid in both sections was similarly poor i.e. with air voids having an average value of 18%. During early trafficking, deflections were in general quite similar, but with more repetitions, cracks in the unreinforced section became more concentrated and discrete than those in the reinforced section which then led to higher deflections as the layer cracked through. The reinforced material, on the other hand, behaved differently with the reinforcement helping to spread the cracks over a greater area, thus reducing their severity. Also, on excavation, the slabs in the reinforced section appeared to be better seated than those in the unreinforced section. It is, of course, not known whether the slabs became better seated during trafficking or the seating of the slabs in the unreinforced section had deteriorated quicker. Cracking parallel to the wheel track (longitudinal cracking) could not be distinguished from cracks in the rut shoulder.

Permanent Deformation on the longitudinal joint test was similar to that of the transverse joint wheelpath test for both the reinforced and unreinforced sections, i.e. the grid-reinforced sections had approximately half the rutting of the reinforced sections.

Asphalt Quality. On completion of the trafficking, core samples were taken from trafficked and untrafficked sections of the test pavement. Stiffness testing was carried out using the Nottingham Asphalt Tester (NAT) [8.13] following which, densities were measured and used to calculate air void contents. A summary of the results is given in Table 8.D.1.

Table 8.D.1 Asphalt Properties: PTF1

Property	Maximum	Average	Minimum
NAT Stiffness at 20°C (MPa)	835	607	350
Air Voids:Top layer (%)	18.8	13.9	10.8
Air Voids:Bottom layer (%)	19.8	18.2	16.3

Results show that the asphalt quality was poor with a relatively high air void content and low stiffness. These values contrast with "typical" values found for a well-compacted material of this type at similar temperatures which would have air voids of around 5% and stiffnesses of around 2000 to 3000MPa. The poor quality of the material is attributed to the time taken to place and compact the material by hand, especially as the layers were thin and cooled quickly.

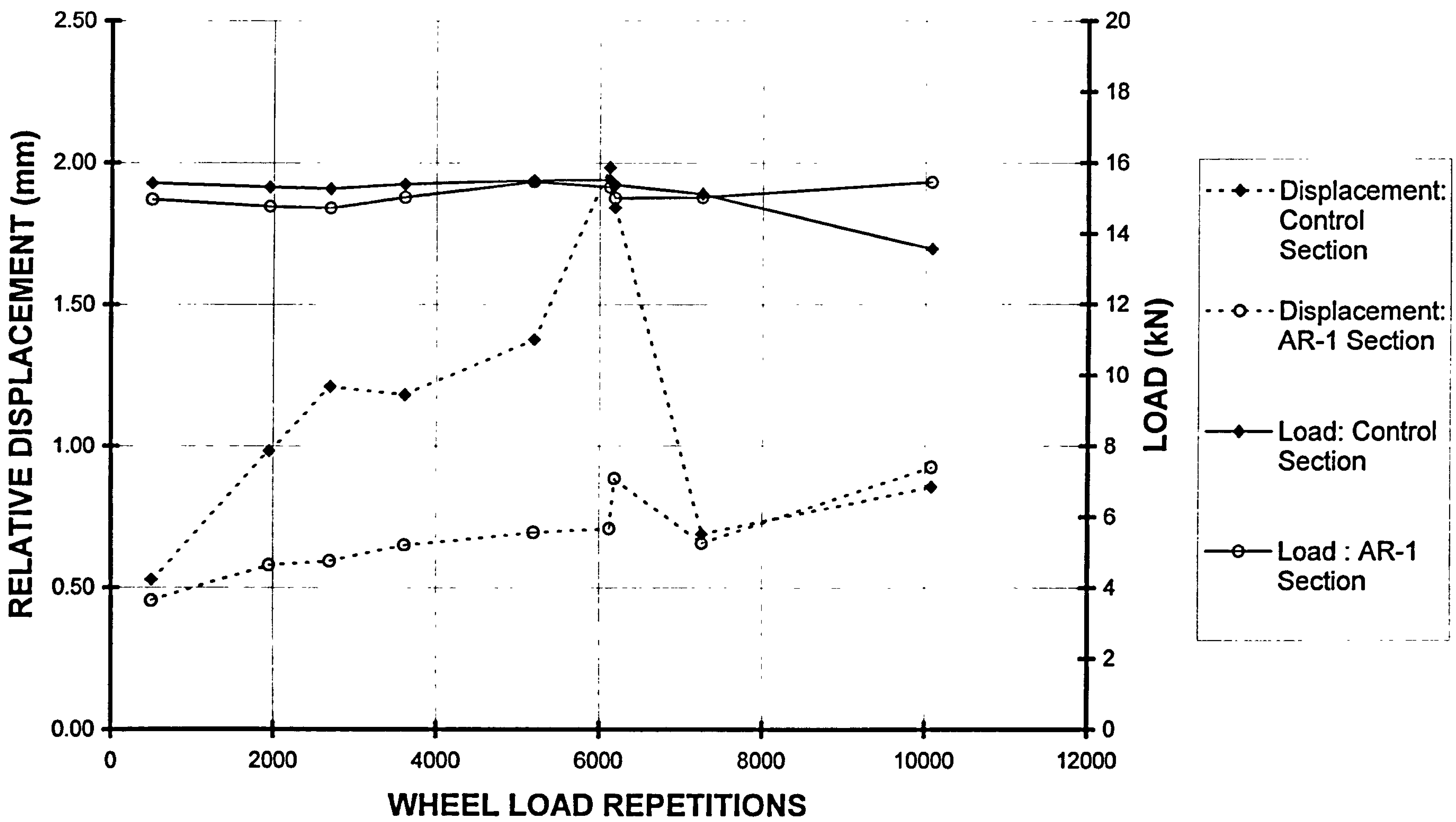


FIGURE 8.D-1
MEASURES OF LOAD AND DEFLECTION :PTF1

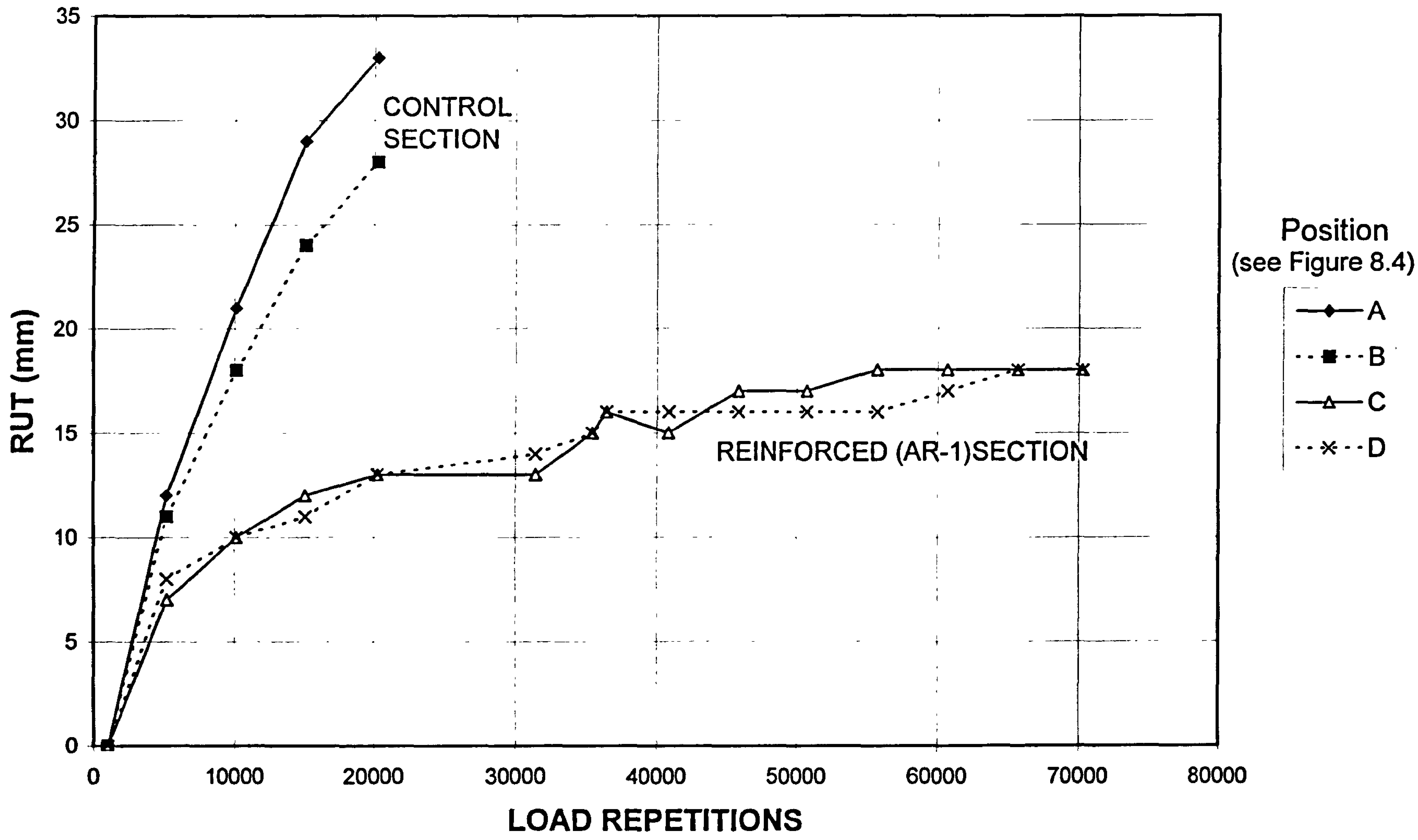


FIGURE 8.D-2
RUTTING: TRANSVERSE CRACK TEST PTF1

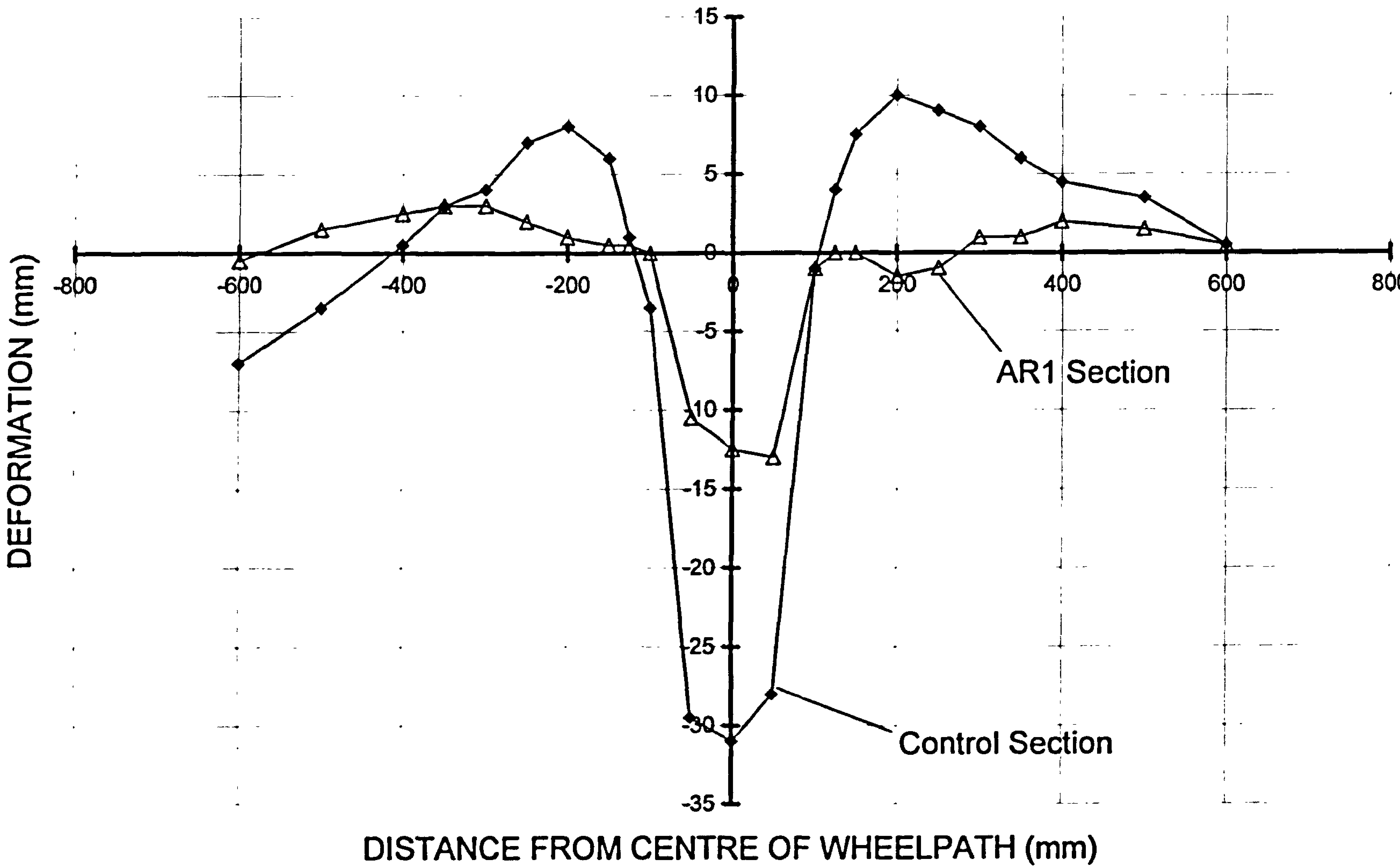


FIGURE 8.D-3
RUTTING: TRANSVERSE PROFILE, TRANSVERSE CRACK TEST
N=20273

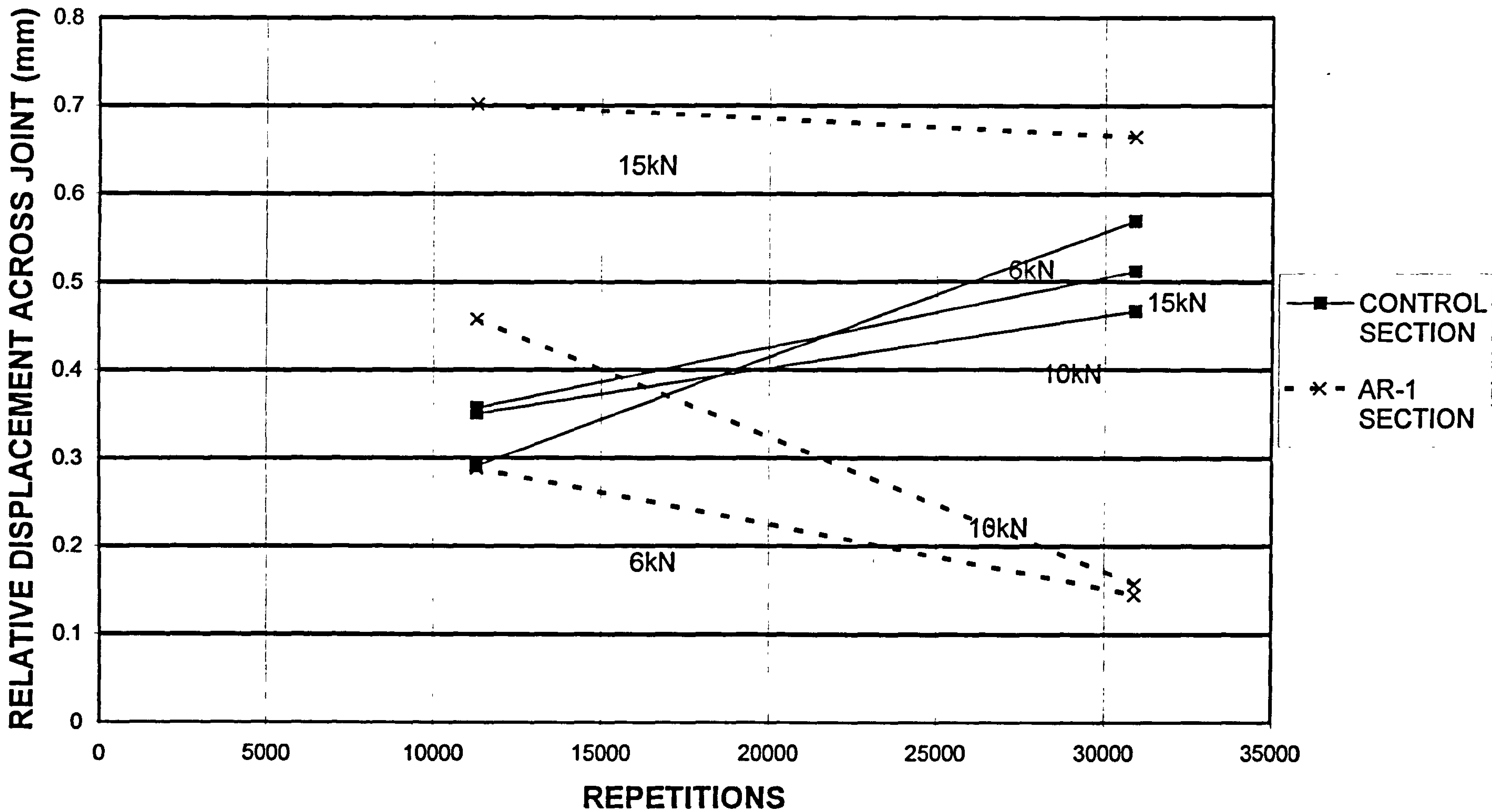


FIGURE 8.D-4
MEASURES OF LOAD AND RELATIVE DEFLECTION:PTF1 LONGITUDINAL CRACK TEST

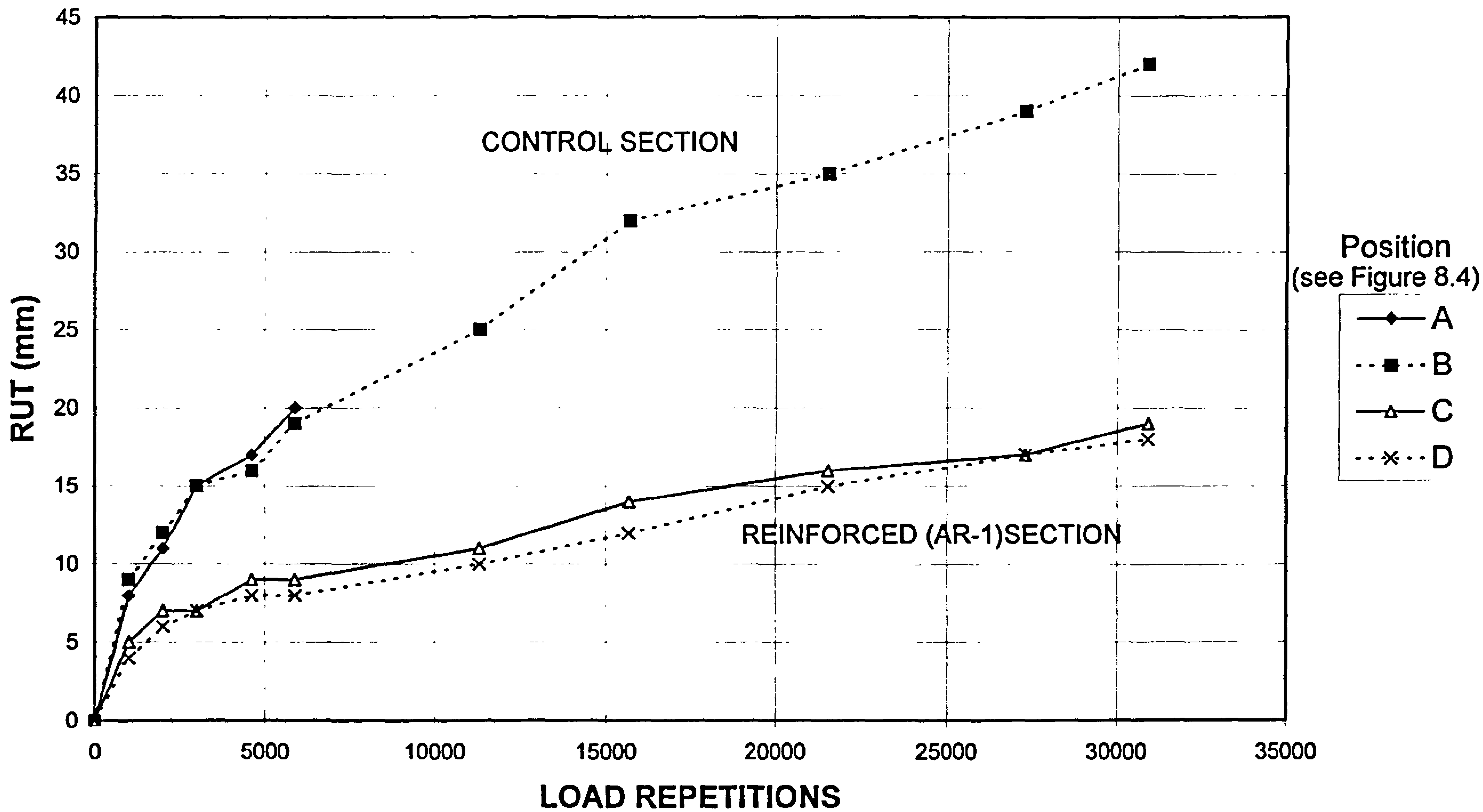


FIGURE 8.D-5
RUTTING: LONGITUDINAL CRACK TEST, PTF1

APPENDIX 8.E

Pavement Test Facility: Test PTF2

Deflection measurements
Vertical deflections measured with the LVDTs had little or no relationship with horizontal deflections measured on the surface of the asphalt with the DEMEC gauge. This was due to the difficulty of anticipating where cracks would form, and hence where to place the DEMEC "pips". In addition, the time taken to measure movements in each of the pips was found to significantly disrupt the test (due to the need to stop trafficking whilst moving the gauge from position to position). DEMEC gauges were therefore not used in test PTF3.

Measurement of initial deflections. Before trafficking began, relative deflections across transverse and longitudinal joints were measured, and are shown in Table 8.E.1. It is noted that although initial deflections give a good indication of the potential longevity of a test section, it is by no means a sure way of predicting which section will fail first. In particular, Table 8.E.1 shows the unreinforced section in the Road-Mesh and Rotaflex wheelpath to have the lowest deflection, but it was then found to have the shortest life, i.e. failure after 20000 repetitions.

Table 8.E.1. Measurements of initial deflections: PTF 2.

Test section	Relative deflection measurements (mm)	
	Transverse	Longitudinal
	0.597	0.705
AR-G	0.548	0.653
Control (AR1/AR-G wheelpath)	0.702	0.631
Rotaflex	0.775	0.564
Roadmesh	0.528	0.453
Control (Road-Mesh/Rotaflex wheelpath)	0.416	0.424

PTF2: AR1-Control-AR-G Wheelpath

Relative vertical deflection

For each section trafficked in this wheelpath, the relative deflections in Figures 8.E-1 and 8.E-2 are seen to increase up to around 25 000 before becoming more constant. Then, at around 45 000 passes deflections measured on the reinforced sections decrease, whereas those for the control section increase. There are various possible explanations for this phenomenon. First of all, deflections changed somewhat erratically as the test progressed, and the final readings could simply be a continuation of this "trend". Secondly, local tilting of the LDVT (as the slabs move relative to each other) could account for some of the differences, although calculations indicate that deflections are likely to alter only by around 10% of the measured value and this should not alter the overall shape of the deflection curve. Thirdly, the apparent decrease in relative deflection could simply mean that slabs under the reinforced sections are moving simultaneously, even though their absolute deflections are actually increasing. This could happen when the material under the edge of the slabs gets progressively more compacted (relative to material under the centre of the slabs), thus allowing more 'rocking' to occur. However, if reinforcement tends to hold slabs together, the differences in relative deflection would be expected to be less than for unreinforced sections, which is the case for this test.

Active transverse cracking was first observed in the unreinforced section at around 20,000 repetitions whereas the reinforced sections cracked at around 54,000 repetitions. It is interesting to note how relative deflections in the reinforced sections decreased after cracks had appeared. In addition to the transverse cracks, "top-down" cracks on the shoulders of the rut adjacent to the wheelpath were noted early in the tests (i.e. from about 4,000 repetitions). From the occurrence of cracking in the reinforced and unreinforced sections, it would appear at first sight that a design life ratio of 2.5 is appropriate.

Permanent Deformation

Deformation measurements are shown in Figure 8.14 and show both the grid-reinforced and composite-reinforced sections to develop around 50% of the rutting of the control section. This is consistent with tests carried out in 1985 [8.5] with AR1 grids. The reason for the divergence of the AR1 and AR-G lines in Figure 8.14 is thought due to the nature of the interlayer bonding, i.e. grid interlock for the grid, and a mixture of interlock and bitumen adhesion with the AR-G composite. It is considered that the interlock of the AR1 and asphalt does not change significantly with load repetitions, whereas the lower interface of the AR-G reinforcement (which relies on a bitumen bond) may have degraded at around 46,000 repetitions or so.

Cracking

Transverse cracking in this wheelpath was seen to occur most in the control areas (see

Figure 8.11) and particularly at the junction of reinforced and unreinforced areas. Longitudinal cracking was found in each section of the wheelpath and appeared to be *linked to the incidence of "rut shoulders".

PTF2: Rotaflex-Control-Road-Mesh Wheelpath

Relative Vertical Deflections

Similar to the performance of the AR1 and AR-G-reinforced sections, the Road-Mesh reinforced section took around twice as long to crack as the unreinforced section. The glass-reinforced composite (Rotaflex) section, however, only carried a similar number of repetitions before cracking as did the unreinforced section, although the Rotaflex section was subjected to larger relative deflections across transverse joints (see Figure 8.E-3). If the larger deflections in the Rotaflex-reinforced section are taken into account by plotting normalised deflections versus traffic to cracking, however, the Rotaflex performance seems consistent with that of other reinforced sections.

Further evidence of the benefits of the presence of reinforcement in the asphalt is indicated by the general level of relative transverse deflection measured in the sections, as indicated in Table 8.E.2 It is evident that both reinforced asphalt sections were subjected to greater deflections than were the unreinforced sections, but cracking occurred at the same time or later than the unreinforced section. Table 8.E.2 also shows how relative deflections in reinforced sections increased at a slower rate than deflections in unreinforced sections.

Table 8.E.2 Relative Deflections Measured Across Transverse Joints: PTF2

Load Repetitions	Roadmesh-reinforced section		Rotaflex-reinforced section		Unreinforced section
A	B(mm)	Ratio C=B/F	D(mm)	Ratio E=D/F	F (mm)
0	0.528	1.27	0.775	1.86	0.416
5 000	1.096	1.22	1.570	1.74	0.900
10 000	1.430	1.14	1.800	1.44	1.250
20 000	1.570	1.31	1.960	1.63	1.200
40 000	1.540	1.03	1.480	0.99	1.500

Permanent Deformation

The relationship between permanent deformation and wheel repetitions measured during the test is shown in Figure 8.14. First of all it is noted that permanent deformation measured on the Rotaflex section was considerably higher than that for the Roadmesh section and, up to 30000 repetitions, not greatly different to that of the unreinforced section. Also, the unreinforced sections for both wheeltracks show quite similar permanent deformation behaviour whereas both sections reinforced with grids and the AR-G composite, showed considerably less rutting than the Rotaflex-reinforced section.

Cracking
The crack patterns seen in Figure 8.11 showed cracking over transverse joints to be worse than over longitudinal joints and is similar to cracking in the other wheel path.

After trafficking, cores were taken in each of the test sections for measurement of stiffness and density and air voids. A summary of test results is given in Table 8.E.3.

Table 8.E.3 Summary of Asphalt Properties: PTF2

Parameter	NAT Stiffness[1] (MPa)	Above Interface[2]		Below Interface[3]	
		Density (kg/m^3)	Air Voids (%)	Density (kg/m^3)	Air Voids (%)
Maximum	718	2.3	21.3	2.3	18.1
Average	553	2.2	14.0	2.2	15.0
Minimum	325	2.0	9.0	2.1	11.9
Standard Deviation	113.6	0.06	2.3	0.05	2.11

Note 1 19 test results
Note 2 20 test results
Note 3 9 test results

The low values of stiffness and high voids content are compatible with the high deformations and relatively quick cracking noted during the test. Results of tests carried out on the PTF cores for all sections are discussed in Section 8.5.3.

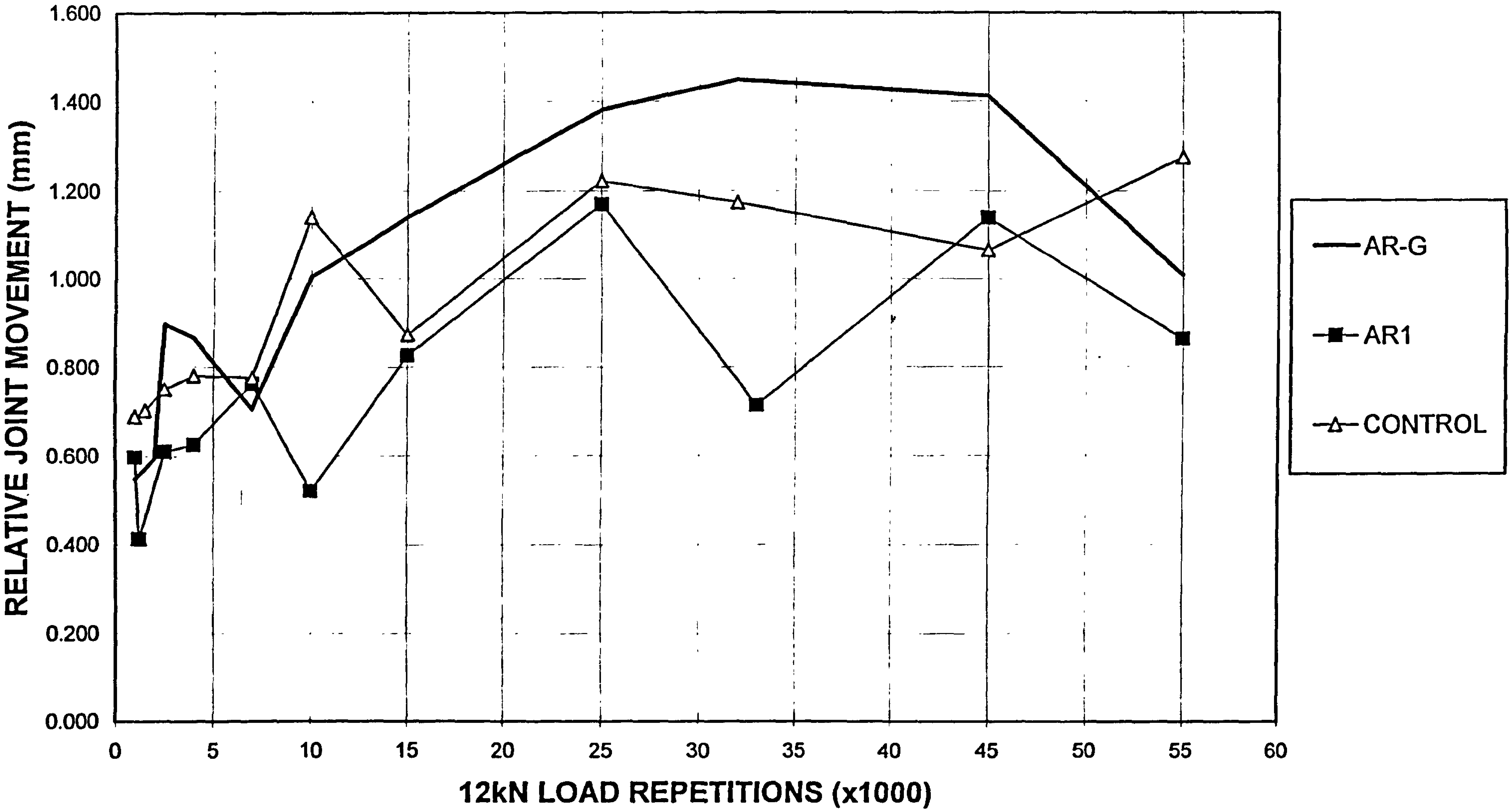


FIGURE 8.E-1
TRANSVERSE JOINT MOVEMENT: PTF2

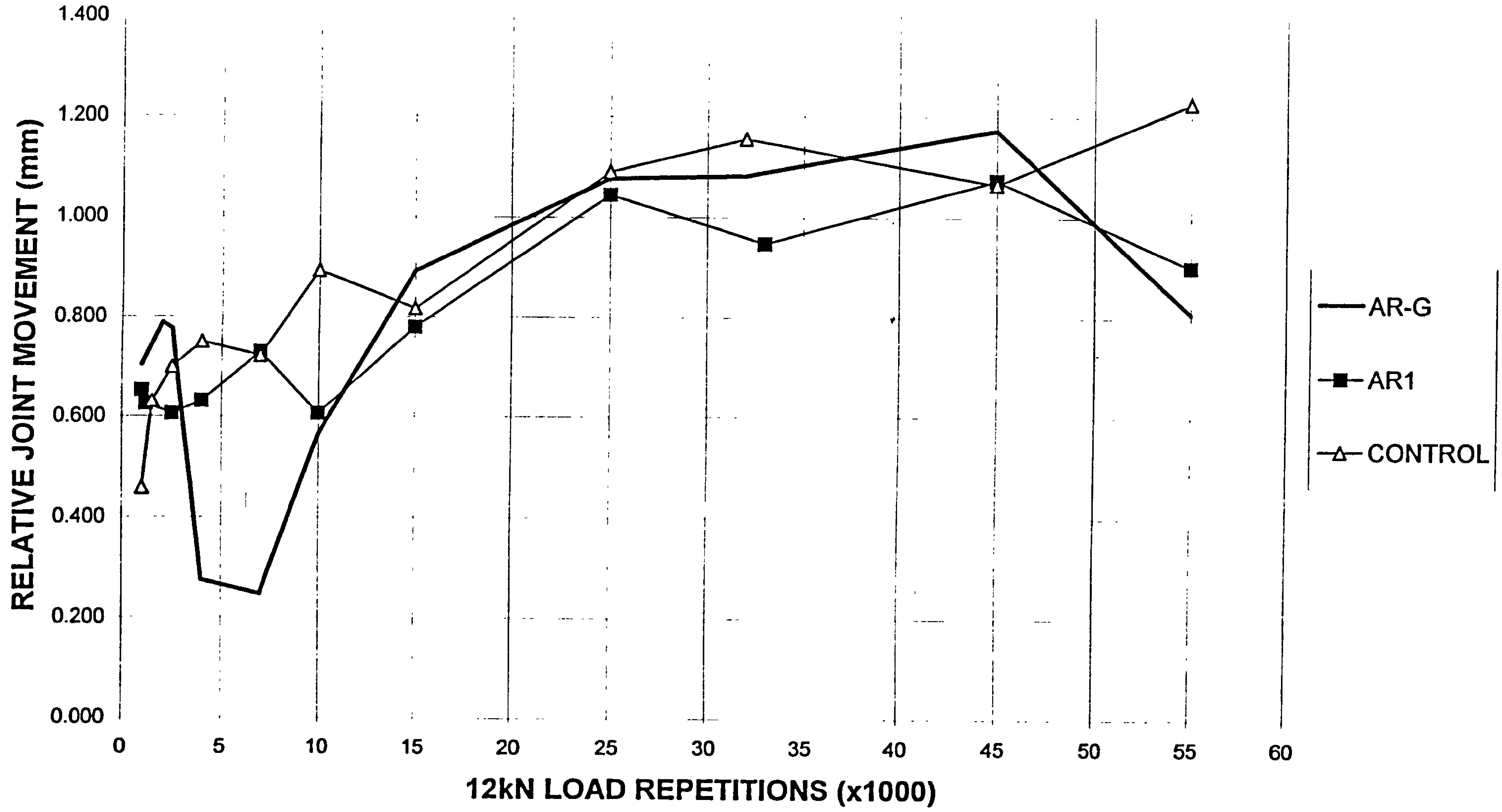


FIGURE 8.E-2
LONGITUDINAL JOINT MOVEMENT: PTF2

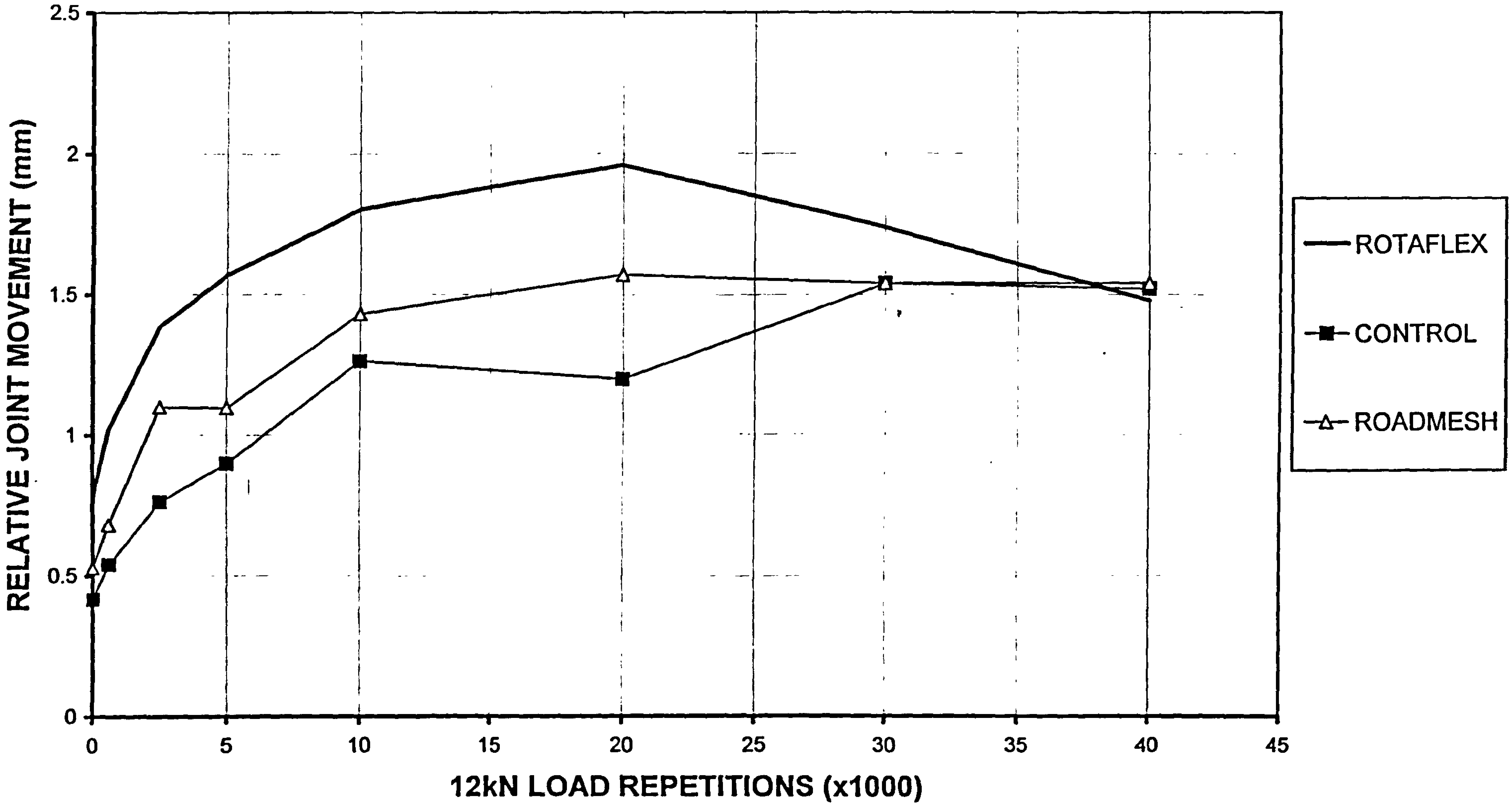


FIGURE 8.E-3
TRANSVERSE JOINT MOVEMENT: PTF2

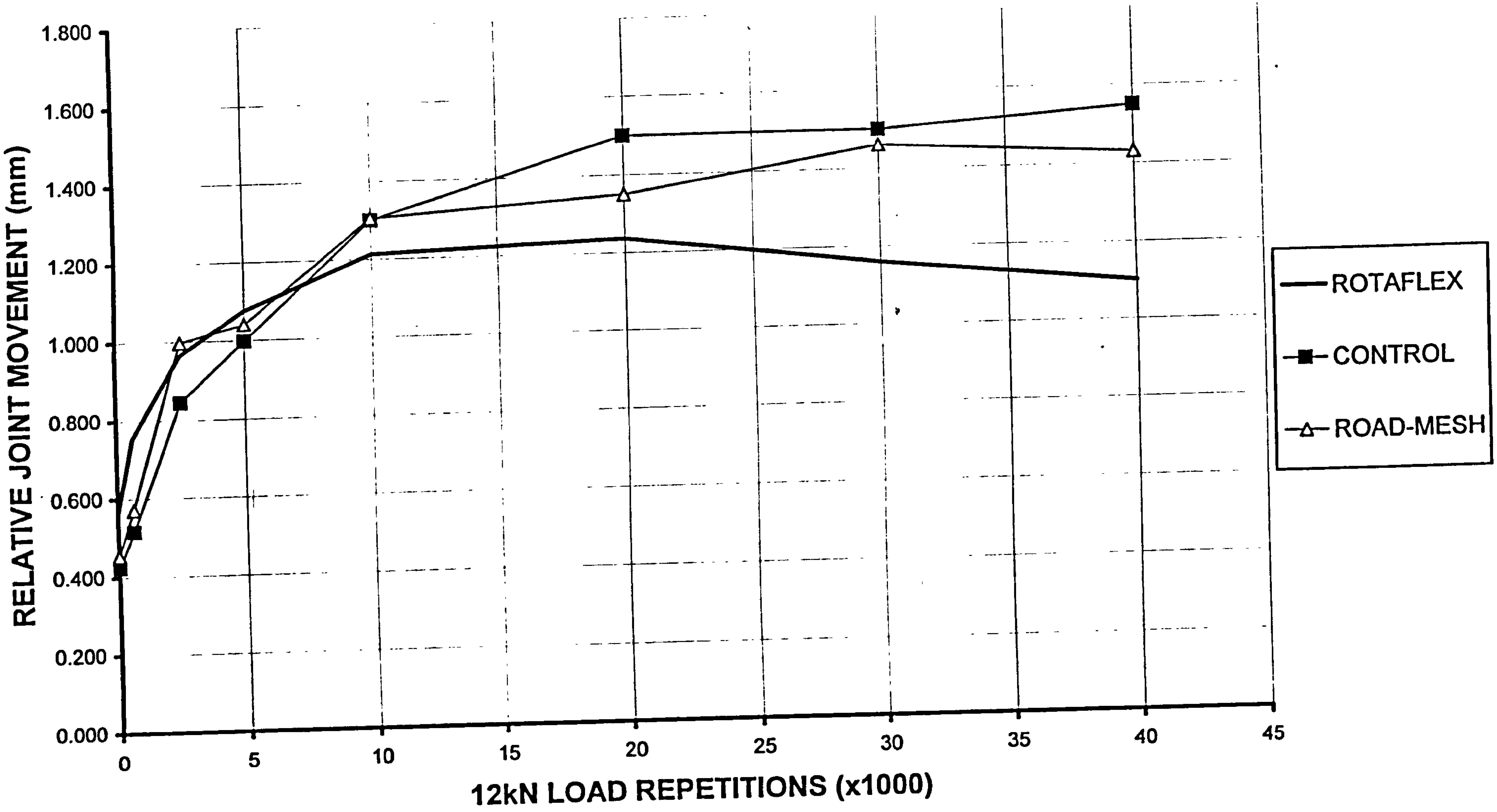


FIGURE 8.E-4
LONGITUDINAL JOINT MOVEMENT: PTF2

APPENDIX 8.F:

Pavement Test Facility: Test PTF3

Deflection Measurements (see Figures 8.F-1, 8.F-2 and 8.F-3)
In addition to relative deflection measurements across transverse and longitudinal joints, "absolute" measures of deflection were taken during test PTF3 to help interpret measurements of relative deflections. It was hoped that the additional information would help clarify the reasons for relative transverse deflections apparently reducing towards the end of trafficking as was the case in PTF2. Measures of "absolute" deflection were made by using an LVDT attached to the side of the Test Pit via a metal "arm", on the LVDT targets adjacent to the wheelpaths.

PTF3: AR1-AR-G Wheelpath

Relative Vertical Deflection
Similar to PTF2, relative deflections measured across transverse joints generally appear to reduce towards the end of the test. At the same time, absolute deflections increase, possibly suggesting that material under the slabs had compacted differentially, and, at the edges of slabs, more compaction allowed additional "absolute" movement to occur, whilst the differential movement reduced. Results from the unreinforced section show relative deflections measured across longitudinal joints to be larger than those measured across transverse joints and larger than "absolute" deflections, although typically, cracks are found on transverse joints rather than longitudinal joints.

PTF3: Rotaflex - Control-Road-Mesh Wheelpath

Relative Vertical Deflection
The most noticeable feature of the behaviour of deflections shown in Figures 8.F-1 and 8.F-2 is the relatively high deflections in the Rotaflex section. However, as seen in Table.8.14, the number of wheel passes to cracking "failure" was only 10,000 for the control section, i.e. considerably less than the number required to fail the reinforced section. Although failure of the unreinforced sections was expected to occur before the reinforced sections, the relatively low number of wheel passes measured on the section between the Rotaflex and Road-Mesh sections (10,000) appears uncharacteristically low. In this case, failure occurred first at the junction between the Rotaflex and Control sections, and would not be typical of a large installation of reinforced asphalt. This point is therefore not included in the regression seen in Figure 8.20.

PTF3: General Comment
The condition of the foundation does not seem to have any direct influence on the

performance of the sections as subgrade conditions are reasonably consistent across the test area (see Appendix 8.C). Plate testing on the sub-base showed that the strengths of the sub-base and top of subgrade are reasonably consistent, with the AR1 section appearing the most stiff, and the AR-G section the least stiff. This runs counter to the trends of the deflection plots.

Cracking-General

With the reduction in rutting and associated "shoulders", less longitudinal cracking was noted than in previous tests, but the crack pattern across transverse joints was more definite. The majority of the cracking was present within the control section or over the joints between the control and the reinforced sections. Cracking at the junction of control and reinforced sections suggests that (a) the reinforced interface is having an effect on performance, and (b), if small areas of reinforcement were to be used (to deal with localised cracking for instance), there is a danger that cracks will be induced at the edges of the reinforced area. Intuitively, it is considered that where a "patch" or "strip" repair is to be carried out, reinforcement used should be made or cut in a manner that reduces sharp changes between reinforced and unreinforced sections. This will be difficult to effect in practice, and would lead to additional time and costs. It is also reasoned that where a stiff (relative to asphalt) interface material is used, it is more likely that edge effects will occur.

Cracking tended to be more obvious within the Rotaflex and Control sections, although cracks were quite well-defined on a transverse joint in the Road-Mesh section. Cracks also formed over the joints marking the edges of the reinforced and control section. Cracks on the junction of the Rotaflex and Control sections were seen to lengthen quicker than elsewhere once cracking had begun. Comments made for the AR1 and AR-G wheelpath are also applicable for these test results.

Material quality

Cores were taken from the pavement from all sections and used for determination of density and for visual inspection. The results in Table 8.F.1 show relatively low densities and high void contents, which are thought due to a combination of the thin layers and speed of construction. Unless air temperatures are high and construction is quick, thin layers cool quickly, leading to the binder becoming stiffer and resulting in a material more difficult to compact.

Table 8.F.1. PTF3 asphalt – density and void content

	Above Interface		Above Interface	
	Density (Mg/m^3)	Voids (%)	Density (Mg/m^3)	Voids (%)
Maximum	2.26	17.19	2.31	16.47
Average	2.19	15.04	2.24	12.98
Minimum	2.13	12.03	2.15	10.17

Rutting - General
The magnitude of rutting was found to be similar to the values obtained from PTF2, but the sizes of the rut "shoulders" were smaller than the shoulders measured in PTF2. Also, with less pronounced shoulders their curvature was reduced and tensile strains were smaller, resulting in less longitudinal cracking adjacent to the shoulders. Figure 8.18 shows how rutting increased with trafficking and how the relative performance of the reinforced sections improved with trafficking. Rutting in the sections reinforced with AR1 and AR-G materials reduced to around 60% of the control rutting. It is interesting to note that although the AR-G is a composite material, the effect on rutting is similar to the AR1 grid. As both the grid and the composite have the same profile on the top interface, it could be that this factor has influence on rutting, particularly on the horizontal "shoving" movement of the asphalt. This possibility and other aspects relating to the test measurements of permanent deformation are discussed further in Section 8.4.7.

Permanent Deformation
The most significant feature of Figure 8.18 is probably the rutting measured on the Rotaflex section being greater than values measured on the Control section. The reason for this could be linked to the shallow profile of the grid and fabric, which may provide less resistance to horizontal movement than for unreinforced material. This is discussed, in Section 8.5.2.

As for test PTF2, the ability of interface materials to help reduce rutting can be ranked by correlating grid profile height or the ratio of grid aperture to aggregate size with the number of wheel passes to achieve different rut depths. The rankings are given in Table 8.F.2 and are further discussed in Sections 8.5.

Table 8.F.2. Ranking of Reinforcement materials according to rut-reduction: Test PTF3.

Performance	**Analysis by Profile Height**		
	Material N=20 000	Material N=30 000	Material N=40 000
1(Best)	RoadMesh	RoadMesh	RoadMesh
2	Rotaflex	AR1	ARG
3	ARG	ARG	AR1
4	AR1	Rotaflex	Rotaflex

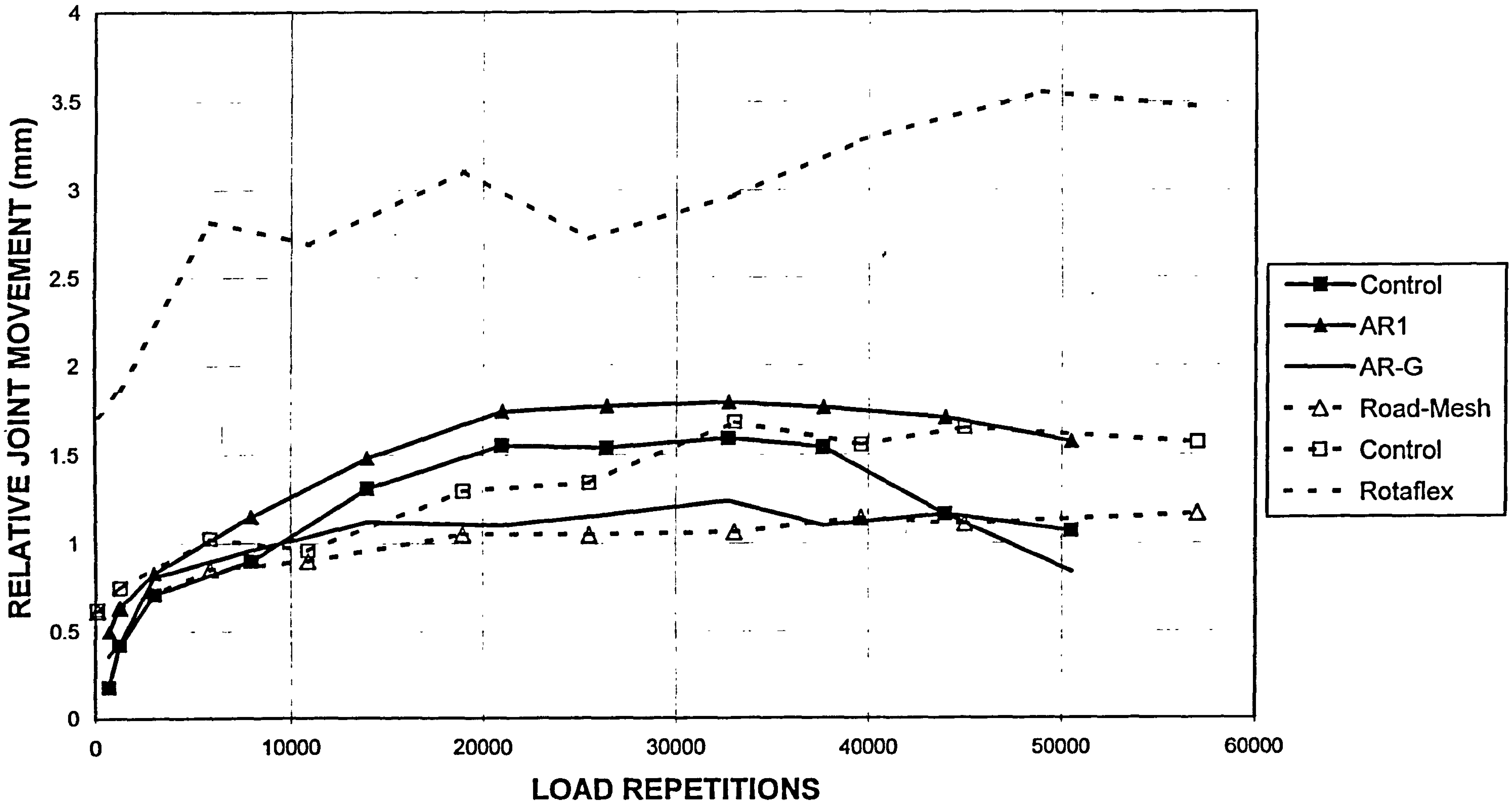


FIGURE 8.F-1
TRANSVERSE JOINT MOVEMENT: PTF3

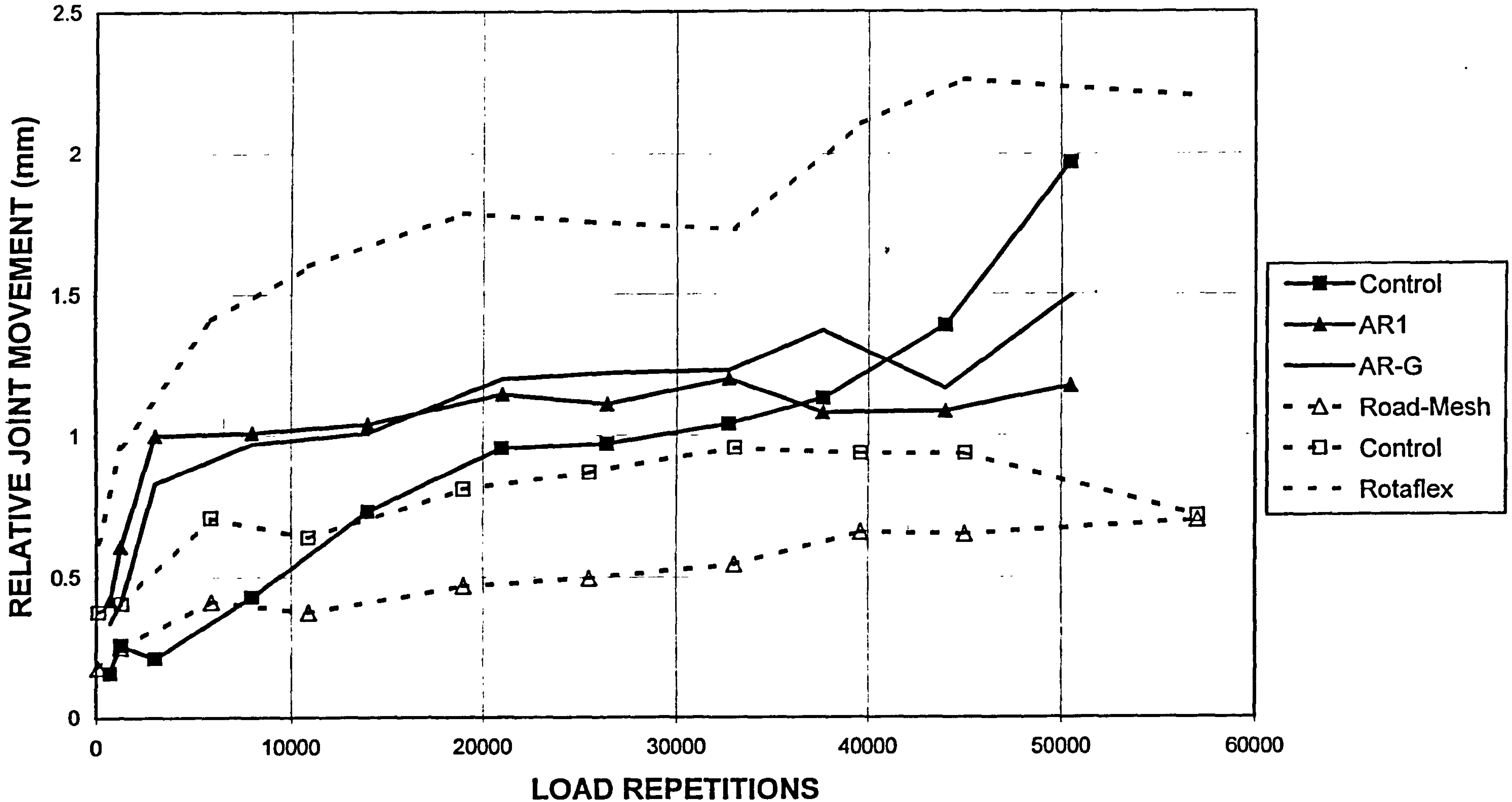


FIGURE 8.F-2
LONGITUDINAL JOINT MOVEMENT: PTF3

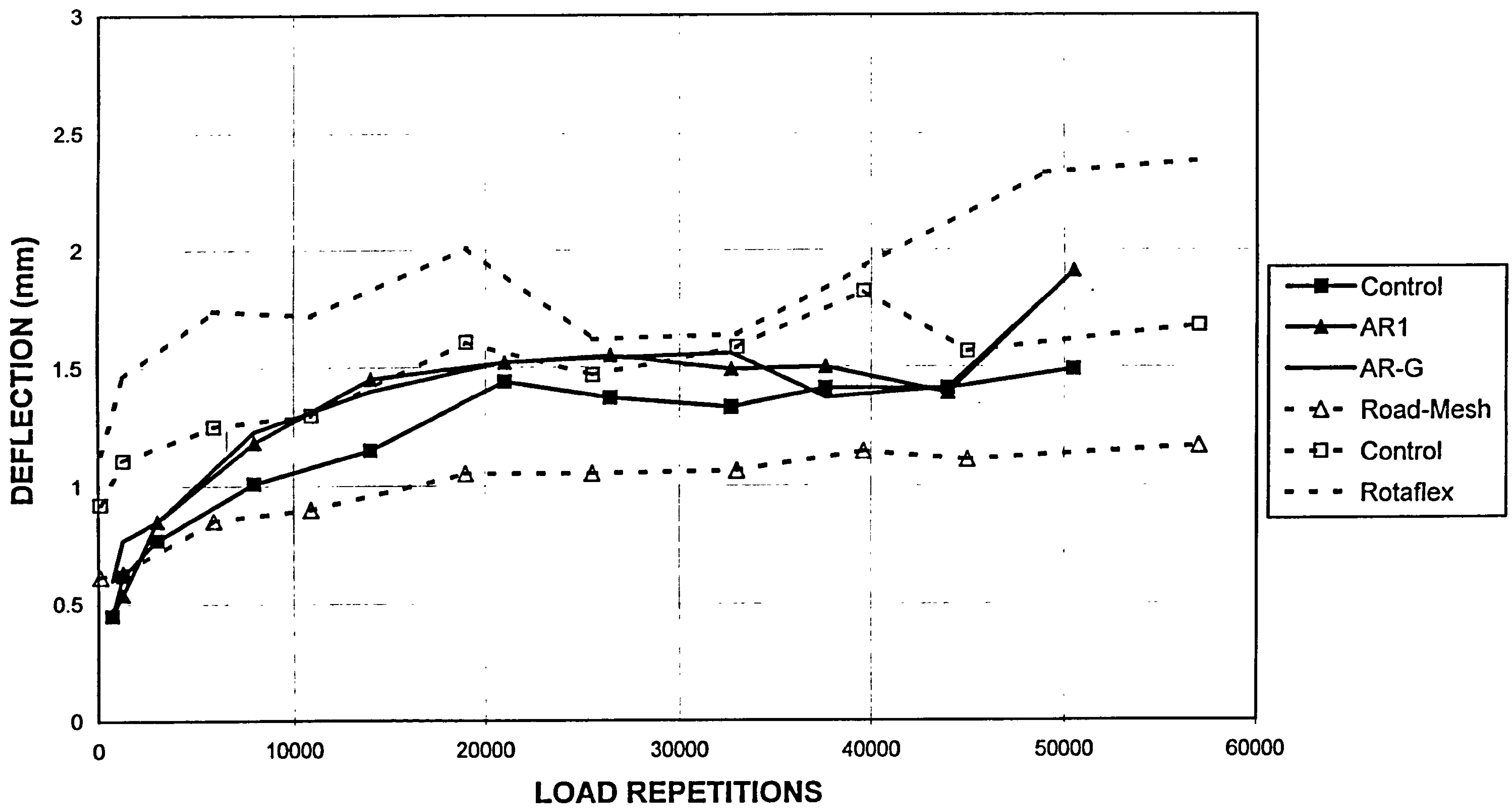


FIGURE 8.F-3
ABSOLUTE JOINT MOVEMENT: PTF3

CHAPTER 9

NUMERICAL MODELLING – FINITE ELEMENT ANALYSIS

CONTENTS

CHAPTER 9
Numerical Modelling – Finite Element Analysis

9.1 Introduction

The application of test results from laboratory specimens to full-scale field situations often presents problems due to differences in loading, environmental influences and construction. One way of dealing with this problem is to numerically model a relatively well-controlled (laboratory) test configuration, and then, when successfully modelled, apply findings to other more difficult configurations, as are found in the field.

Traditionally, multi-layer linear elastic programmes have been used to model pavements under wheel loading [9.1]. However, due to assumptions in the theory, i.e. plane-strain conditions (infinite lateral dimensions), continuity and homogeneity, the theory is not suitable for application to laboratory test specimens with finite lateral boundaries. Furthermore, the presence of joints and or cracked pavement layers in typical field situations makes sensible use of these programmes difficult. However, to enable the analysis of discontinuous layers and problems with 'complex' geometry to be carried out, the Finite Element Analysis (FEA) technique, (one of a number of continuum mechanics approaches) can be used.

Continuum mechanics is based on the principle that it is possible to derive equations and relationships that accurately describe the behaviour of a small part of a body. By dividing the body up into 'small' elements, and using conditions of equilibrium and continuity, it is possible to obtain a reasonably accurate prediction of stresses and strains within the body. The FE approach is particularly useful for modelling discontinuities and is thus particularly suited to modelling cracked and jointed pavements.

When material cracks it does not alter from an intact material to a cracked state instantaneously, but does so progressively. In fracture mechanics this is modelled by a zone of material with reduced strength in front of the crack tip – the 'process zone'. To apply the linear elastic fracture mechanics model to the asphalt specimens modelled in this study, it has been assumed that this plastic zone is small in relation to the crack.

During cracking three stages can be defined [9.2]:

Stage 1: the beginning of the fatigue process where the plastic zone is small and may be assumed to be the same size as the voids in asphalt.

Stage 2: in this stage the plastic zone is larger than in Stage 1, but significantly smaller than the crack.

Stage 3: this stage is characterised by unstable crack growth and a large plastic zone.

The application of the Linear Elastic Fracture Mechanics approach can be considered valid in Stage 2.

The FE programme CAPA-2D [9.3] was chosen for modelling laboratory test configurations as it is well-suited to modelling reinforced asphalt through use of a combination of quadrilateral and interface elements. CAPA-2D is a PC-based programme using a linear-elastic material model and also featuring a remeshing routine which automatically propagates cracks through the mesh by disconnecting adjacent nodes. (See Figure 9.1). As output, the programme gives stresses, displacements and Stress Intensity Factors (SIFs) at crack tips, which in fracture mechanics are used to represent the energy available for crack propagation, and are calculated using the shear modulus and displacements in the vicinity of the crack tip. More detailed descriptions of the programme can be found in References 9.4 and 9.5.

The Stress Intensity Factor 'k' can be expressed as

$$k = \sigma\sqrt{\pi a}\,\beta$$

Where σ is the reference stress
β is a factor taking into account the geometry of the crack,
and a is the crack length

As implied above, the main reason for using numerical models was to aid in interpreting the results of the beam test and help define significant parameters determining the rate of crack development. In particular, the influence of reinforcement strength and bond stiffness on beam behaviour and performance required investigation, especially in view of the range of the reinforcement products used in the beam tests. Then, subject to successful modelling of the beam test results, the possibility of modelling field conditions was also to be investigated. Although a full parametric study comparing the effects of different values of stiffnesses of reinforcement and interface bonds was beyond the scope of this work, the possibility of using CAPA-2D to develop design curves was to be investigated.

As CAPA-2D provides SIFs as output, it was convenient to use the relationship described by Paris and Erdogan [9.6] to estimate the speed of crack progression:

$$\frac{\delta c}{\delta N} = A.\left[k_{eq}\right]^{n}$$

where the number of load repetitions 'N' required to propagate the crack a distance Δc can be computed. A and n represent material parameters, and k_{eq} is an 'equivalent' stress intensity factor that takes into account crack propagation for different conditions e.g. predominantly symmetrical 'bending-induced cracking' (Mode I) or more asymmetrical 'shear–induced cracking' (Mode II), as seen in Figure 9.2.

9.2 Modelling the Beam Test

The two main aspects of the beam test to be modelled were; (i) rate of crack propagation across the interface for different reinforcement products, and (ii) the apparent debonding adjacent to the reinforcement that was seen to occur in some of the beam tests (see the 'crack maps' in Chapter 7).

Primarily, therefore, CAPA-2D was used to obtain measures of SIFs for unreinforced and reinforced beams to relate the speed of crack propagation to the magnitude of the SIF using Equation 1 (assuming constant material parameters n and A). In addition, however, to help explain the apparent debonding, the levels of stress adjacent to the reinforcement and crack path were also investigated, as was the effect of debonding on crack development. For comparison of performance, the repetitions required for the crack to propagate from a height of 20mm (10mm below the interface) to 40mm (10mm above the interface) was used. This also removes differences in crack initiation, which were often found to be significant.

9.2.1 The Modelling Approach

To model the beams, the meshes shown in Figures 9.3 and 9.4 were used, as were the material parameters and applied load given in Table 9.1.

Table 9.1 Data used to run CAPA-2D

Component	Input Variable	Range of Values
Asphalt Stiffness	MPa	1500
Interbond Stiffness	Dtt (N/mm/mm^2)	1-80
Reinforcement (single bar) stiffness	EA (N at spacing in mm)	1-100 000 at 26mm 25000 at 65mm
Rubber Stiffness	MPa	2.5
Load	kN	2.750

Values of SIFs and shear and normal stresses were recorded from the output files and used to calculate relative rates of crack progression.

9.2.2 Rate Of Cracking - Reinforced Versus Unreinforced Beams.

Figure 9.5 shows how the magnitude of the SIF is calculated to vary as the crack progresses through glass- and polypropylene-reinforced beams and an unreinforced beam. In the absence of interface bond measurements for the steel-reinforced beam, modelling of this beam was not carried out. Figure 9.6a shows the values of SIF in Figure 9.5 converted to crack growth using the Paris' Law and assuming parameter values n=3, and A=3x 10^{-6} (mm/cycle) using information from de Bondt [9.5] and Jacobs[9.7] for initial guidance, and then using iteration to obtain a good fit with the test data. The data fitting was carried out in two phases, first of all values of A and n were varied to obtain curves similar to the data being modelled (see Figure 9.6a), and secondly, to obtain a better fit with test data, the computed curves were translated along the x-axis by deducting repetitions (see Figure 9.6b). It is noted that in each case, the computed curves lie to the right of the test data showing the predictions to be optimistic. The greatest adjustment was applied to the curve corresponding to the glass-reinforced beam and the least adjustment was applied to the curve representing the unreinforced beam. It should be noted that, initially, it was intended to model an 'average' curve of each reinforcing type but when the averages were computed, the characteristic shapes of the curves were distorted and were not considered to represent the data particularly well. Instead, therefore, one curve for each of the grid test types, was selected to represent each type of reinforced beam. Values of A and n that fitted all three of the beam types were found through iteration, and as single values of A and n were found to give reasonable fits of all test curves it seems that these parameters represent material properties of the asphalt, and the reinforcement plays its part in suppressing crack propagation by altering the stress intensity at the crack tip.

Notwithstanding the limitations of using the assumed values, a clear distinction between the behaviour of reinforced and unreinforced beams is noted.

Consideration of differences between beam test results and the (uncorrected) FE predictions suggests that the principal reason was due to variations in crack initiation and initial propagation from the notch. Inspection of asphalt density measurements in Figures 7.24 and 7.25 of Chapter 7, suggests there was no obvious difference in asphalt quality, and construction of interlayers was well-controlled following manufacturers specifications, with the bitumen emulsion monitored to ensure that it had 'broken' before the reinforcement was applied. Also, the temperature of the asphalt was monitored to ensure that the emulsion was applied in similar conditions and that the top layer of asphalt was compacted over a material of 'similar' properties. Another possible reason for differences between test curves and predicted curves is debonding between the asphalt and reinforcement, causing higher strains in asphalt and thus quicker crack propagation.

In particular, it is reasoned that the large differences in stiffness between glass and asphalt may promote debonding as beams deflect. A limited investigation (using CAPA) was therefore carried out to look at the effects of debonding using different combinations of reinforcement and interlayer bond stiffnesses. This is described in Section 9.2.3.

9.2.3. Investigation of Interface Bond Stresses

The potential influence of a range of reinforcement and interface stiffnesses on interface stresses is illustrated in Figures 9.7 and 9.8.

Figure 9.7 shows values of SIF and maximum stresses in elements adjacent to the reinforcement, above and below the reinforcement (for the first increment of cracking above the interface). The most obvious feature of this graph is the significant increase in tensile normal stress and reduction in SIF when stiffnesses are varied from 1000N to 100 000N. It is also noted that there is only a slight change in shear stresses over the range of strengths, and that shear stresses above the reinforcement are significantly less than those below the reinforcement. Values for reinforcement stiffness measured in the laboratory (see Chapter 5) for the Rotaflex materials (around 30,000N/ε), coincide with the steepest section of the graph, which implies that a relatively small change in reinforcement stiffness can result in a significant increase (or decrease) in bond stress, leading to earlier bond failure. Furthermore, although glass is inherently brittle, during grid testing, the Rotaflex samples were seen to fail in a 'gradual' fashion, i.e. with individual glass fibres failing sequentially, see Figure 5.5 in Chapter 5. This suggests that a lower stiffness was measured than would be the case if all glass strands failed simultaneously, which may occur when the reinforcement is confined (i.e. surrounded by asphalt). The effective stiffness of the Rotaflex would then be higher, leading to bigger induced interface stresses, and thus earlier bond failure. It is considered that this was probably a contributing factor to the relatively quick cracking rate through glass-reinforced interfaces, (see Figure 7.16 of Chapter 7). However, it is acknowledged that there would need to be a substantial increase in stress for the interface to fail within a few repetitions as the direct tensile stresses measured in the tension tests described in Chapter 7 are in general considerably higher than the CAPA-computed values. The slower loading in the PTF test, however, would lead to lower bond resistance and thus be more likely to cause failure in this fashion.

The reduction in SIF as reinforcement stiffness increases in Figure 9.7 is potentially important, although, with increased reinforcement stiffness, the increase in interface stresses may cause earlier bond failure. An implication from this observation is therefore, that, when choosing the reinforcement strength to optimise reinforced asphalt performance, a balance between reducing SIF values and limiting the increase in interface stress needs to be found.

Figure 9.8 shows Stress Intensity Factors calculated for a selection of bond stiffnesses and constant reinforcement stiffness of 30kN (strands at 26mm centres). Of particular note is the large increase in SIF below 10 N/mm/mm^2, which indicates how poor construction could result in poor interlayer bond, leading to high SIFs, and hence quick crack propagation. This phenomenon is perhaps another factor contributing to the relatively quick cracking across the interface of the glass-reinforced beams, although beams were constructed with care to attain consistency within the samples. To confirm Paris' law parameter values, and the apparent ratio of cracking rate of unreinforced beams to reinforced beams (2 to 3 - which agrees with the findings of Lytton and Jayawickrama[9.8]), additional beam tests results would be useful. In addition, tests with different reinforcement types might show a variation of n values with reinforcement type, which would also help improve analysis.

9.2.4 Debonding

During testing, what appeared to be a crack along the interface (adjacent and below reinforcement for reinforced beams) occurred in varying degrees, typically around the centre of the beam. This apparent debonding, it was reasoned, must affect crack growth through redistribution of stresses, and, as debonding would be difficult to measure experimentally, numerical modelling was used to estimate the potential influence on cracking.

The beam was modelled such that the crack had already progressed through the asphalt beneath the reinforcement and was beginning to move through the first layer of elements of the asphalt above the reinforced interface (see Figure 9.9). If stresses exceeded 'failure' stresses measured in the shear box and direct tension tests, the elements concerned were 'debonded' i.e. bond stiffnesses were set to zero. The programme was then run again, and stresses compared to failure stresses.

Inspection of the numerical output showed that for fully bonded reinforced beams, the bond between the reinforcement and asphalt in elements 120 and 200 (see Figure 9.4) would be the first to 'fail', as would elements 185 and 110 in the unreinforced beams (see Figure 9.3). Also, the ratio of measured bond strengths to calculated stresses indicate that the bond stresses would fail both in shear and in direct tension.

Where stresses do not exceed bond strengths, fatigue relationships can be used to carry out detailed analysis of the behaviour under repeated loading. For example, a relationship given in the Shell Manual [9.9] can be used to estimate fatigue resistance for normal stresses, and a relationship derived by Jannsen [9.101] for shear stresses. A procedure to estimate pavement life using this approach has been proposed by Gaarkeuken et al [9.11] and includes a 'multi-crack' option with interface failure, and so-called 'secondary cracking'. A secondary crack in this case refers to a vertical crack occurring above the interface, but a short distance away from, and parallel to the 'primary' (vertical) crack, which is normally found between the crack initiator (the notch in the case of the beams modelled) and the reinforced interface, (see Figure 9.10).

Secondary cracking is apparently caused in three main stages [9.11]:

(a) An initial crack progresses up to the interface.
(b) The interface bond strength is exceeded and the bond between the reinforcement and asphalt is broken near to where the crack and the interface meet.

(c) Stresses reorientate and at some distance along the interface, a 'secondary' crack forms above the interface, often roughly parallel to the primary crack.

With limited debonding, SIFs are seen to reduce, albeit by different amounts depending on whether the interface above or below the reinforcement is debonded (see Table 9.2). This compliments the findings of Brown et al [9.12] who noted that a degree of debonding led to a decrease in the rate of crack growth. However, when debonding occurs, tensile stresses above the interface increase, and if this leads to significantly large debonded lengths i.e. the beam acting as two separate beams, high deflections and tensile stresses occur at the lower face of the upper beam.

It follows that experiments to determine if the extent of debonding could be controlled (in practice) would be useful, as, if this was the case, crack resistance might be improved, as compared to beams showing no, or excessive debonding.

Table 9.2 Influence of Debonding on SIF Values

Position	**No of decoupled (cracked) elements (measured from interface)**			
	1	2	3	4
Above Reinforcement	1.9	2.6	2.2	1.5
Below Reinforcement	1.9	2.5	2.0	1.3
No debonding	2.0	2.7	2.1	1.3

The distribution of stresses for a beam with and without reinforcement with limited debonding either side of the vertical crack is given in Figures 9.11 and 9.12. Figure 9.11 shows a concentration of stresses at the edge of the debonded zone with a corresponding decrease in the stresses at the crack tip (if compared to Figure 9.12). Figure 9.13 shows the tensile stress distribution when the beam is debonded for approximately 63mm either side of the vertical crack. The distribution of tensile stresses is quite different to that seen in Figures 9.11 and 9.12 and shows virtually no tensile stress below the interface. The majority of the tensile stress contours are, however seen just above the interface which suggests that this area is the most likely area to develop vertical tensile cracks (see Figure 9.10). Thus, in practice, a correct balance between debonding and good bond is required. An extreme case where all interface elements below the reinforcement are debonded is shown in Figure 9.14. It is obvious from this figure that the limit where debonding is beneficial needs to be defined, and possible means to develop a degree of 'controlled debonding' should be found.

9.2.5 Effect Of Reinforcement Stiffness On A Debonded Beam

To investigate the influence of reinforcement stiffness on crack resistance during debonding, FE runs were carried out with various debonded lengths of interface either side of the 'primary crack'.

The results are seen in Figures 9.15 to 9.17 where reinforcement stiffness is seen not to alter stresses significantly in either of the interfaces adjacent to the reinforcement, but does shows values of SIF reducing significantly as the stiffness of reinforcement increases.

9.2.6 Effect Of Interface Bond Stiffness

FE runs were carried out where the interface bond stiffness was varied, but the reinforcement kept constant (at 30 000N at 26mm centres). The results are given in Figures 9.18 and 9.19 where stresses below the interface are seen to be higher than those above the interface, and the most marked change in stresses is found in shear stresses, in the asphalt directly below the interface.

9.2.7 Effect Of Asphalt Stiffness On Values Of SIF

As the values of asphalt stiffness used in modelling were lower than expected in field conditions, the model was run using an asphalt stiffness of 3500MPa to investigate the effect on SIFs. The resulting values of SIF for reinforced and unreinforced beams are given in Tables 9.3 and 9.4 and compared with values obtained using a modelled stiffness of 1500MPa.

Table 9.3 Influence of Asphalt Stiffness on Stress Intensity Factors- Reinforced Beam

Crack Increment	Asphalt Stiffness =1500MPa	Asphalt Stiffness =3500MPa
	SIF	SIF
1	1.94	2.25
2	2.53	3.09
3	2.03	2.78
4	1.34	2.15
5	0.377	1.14

Table 9.4 Influence of Asphalt on Stress Intensity Factors- Unreinforced Beam

Crack Increment	Asphalt Stiffness=1500MPa	Asphalt Stiffness = 3500MPa
	SIF	SIF
1	9.2	11
2	5.6	6.9
3	3.8	4.9
4	2.9	4.0
5	1.7	2.5

Tables 9.3 and 9.4 and Figure 9.20 show that for both asphalt stiffnesses the reinforced material has lower values of SIF, but the effect of reinforcing asphalt is greater when used with less-stiff material. This may imply that

reinforced asphalt has greater benefits in the short term than in the long term, when typically, asphalt hardens and becomes stiffer.

9.3 Modelling Cracking Through a Reinforced Pavement

As the relatively successful modelling of the beam test results suggested that CAPA-2D could be used effectively for modelling this test, a limited investigation was carried out to assess the suitability of CAPA-2D for modelling cracking in full-scale pavements. Accordingly, the mesh shown in Figures 9.21 and 9.22 was used with the assumption of plane-strain conditions. With the plane-strain assumption and using a standard 20kN wheel load, computed deflections were considerably higher than expected, due to the use of 2-Dimensional theory. For modelling purposes this was not considered satisfactory, as SIFs are calculated using deflections, and exaggerated deflections could not be used with confidence. As a solution therefore, it was thought that defining equivalent elastic parameters of the pavement by matching deflection bowls calculated from a multilayer linear elastic model (MLLEM) and CAPA-2D would give more accurate simulations. Accordingly the following procedure was carried out:

(a) Deflection bowls were calculated using the MLLEM.
(b) Loading and elastic layer parameters were altered in the FE model until deflection bowls matched to less than a 1% difference (calculated using the root mean square of the differences between three deflections in the centre of the of the bowl).
(c) A range of reinforcement and bond stiffnesses were substituted in CAPA-2D to model their effect on SIFs.
(d) To model the effect of a moving wheel, loads were modelling in five positions from the crack, i.e. 0, 100, 200, 300 and 1000mm. For each height within the FE mesh equivalent SIFs were calculated using the following expression [9.11].

$$k_{eq} = k_I \mathrm{Cos}^3 \frac{\Theta_m}{2} - k_{II} \mathrm{Cos}^2 \frac{\Theta_m}{2} . \mathrm{Sin} \frac{\Theta_m}{2}$$

Where θ_m is the angle of crack extension and can be calculated iteratively from

$$k_I \mathrm{Sin}\Theta_m + k_{II}(3\mathrm{Cos}\Theta_m - 1) = 0$$

(e) For each set of load positions, the maximum value of k_{eq} was used to calculate crack propagation:

$$\frac{dc}{dN} = Ak_{eq}^n, \text{ and}$$

$$\text{Loads 'N' to propagate a length 'c'} = \left[\frac{C}{\left(\frac{dc}{dN}\right)} \right]$$

Estimates of crack height versus load repetitions are plotted in Figure 9.23.

The MLLEM simulation used a 20kN load on a pavement with the material parameters given in Table 9.5.

Table 9.5 Parameters used to model cracking through a pavement on a 'weak' foundation (Multi-layer Linear Elastic Model)

Pavement Layer	Elastic Stiffness (MPa)	Poisson's Ratio (MPa)
Asphalt overlay	5000	0.40
'Existing' Asphalt	3000	0.4
Subbase	300	0.35
Subgrade	40	0.35

The Interface conditions between the overlay and the 'existing' asphalt that were modelled using CAPA-2D are given in Table 9.6

Table 9.6 Combinations of Interface Properties Modelled.

Upper Interface stiffness ($N/mm/mm^2$)	Reinforcement strand Stiffness (kN) and spacing (mm)	Lower Interface stiffness ($N/mm/mm^2$)
0.2	Unreinforced beam	
0.15	25.0 at 65mm	0.15
6	25.0 at 65mm	6
25	25.0 at 65mm	12
25	25.0 at 65mm	25
100	25.0 at 65mm	100

Note that the low values of interface stiffness used for the unreinforced beam and the beam with an interface stiffness of $0.15N/mm/mm^2$ were used to be compatible with the values measured on the cores taken from the Pavement Test Facility. Also, to compute rates of crack progression, values of A and n derived from beam test modelling were used.

From Figure 9.23 it is seen that:

- Using similar interlayer stiffnesses to those measured on cored material taken from the Pavement Test Facility (PTF), the time taken for the crack to progress through the reinforced pavement was approximately 1.7 times longer than the unreinforced pavement. This compares reasonably well with the PTF findings where the ratio was approximately three times, especially if it is appreciated that considerable uncertainty in crack initiation exists, which could distort the comparison.
- Bond stiffness has a large effect on crack resistance. The shape of the curves help illustrate the effect of the interaction of bond and reinforcement stiffness. As a crack propagates up through the pavement, it normally tends to open up. However, with reinforcement this tendency is reduced if adequate bond is available to mobilise the properties of the reinforcement. The stiffness of the reinforcement remains the same for all the runs, but

9.4 Conclusions

Detailed modelling of beam tests has led to the following conclusions and observations:

- Finite Element Analysis is able to model the main features of cracked reinforced asphalt layers, although a 2-Dimensional model is difficult to use to model 3-Dimensional loading conditions.

- The programme CAPA-2D is a useful PC-based FE programme that incorporates an automatic 'remeshing' routine to help model cracking. The routine simulates cracking by disconnecting nodes on the crack path.

- Modelling shows that by reinforcing an asphalt layer, stress intensity factors (SIFs) can be significantly reduced, implying that the rate of cracking though the asphalt can also be reduced. However, although the reinforced asphalt gave longer life with both beam testing and the Pavement Test Facility, clear-cut improvement was not always obvious, particularly with the beam test results. Closer inspection shows that factors such as crack initiation and propagation early in the test can distort comparisons. The use of empirically-fitted values of the parameters used in the Paris Law (A and n) are also a potential source of differences.

- Stresses computed adjacent to the reinforcement suggest that debonding can occur in the interface layer close to the crack path.

- Debonding tends to reduce the SIF at the crack tip, especially if debonding occurs below the reinforcement. Debonding above the reinforcement, however, has a smaller effect.

- The modelled performance of the beam appears to be consistant with the trends of measured behaviour, depending on the values of A and n used in the Paris equation. The observations that reinforced beams tend to resist cracking for up to two or three times the number of loads of unreinforced beams over the interface region of test beams seems justified.

- The importance of interlayer bond is illustrated, suggesting that this parameter is at least as important as the stiffness of the reinforcement. This has particular implications the field, as with poor construction control, areas of poorer bond can quite easily occur, leading to higher cracking rates and hence earlier failure.

- A reinforced interface needs to be properly designed to be effective. For instance, an increase in reinforcement stiffness can reduce SIFs, but, in addition tends to increased bond stresses, which in turn may lead to excessive debonding of the asphalt-reinforcement bond.

- To model a reinforced asphalt pavement (in 3-Dimensions) using CAPA-2D, 'equivalent' pavement stiffnesses were used to overcome problems with loading in 2-Dimensions. These stiffnesses were determined through minimising differences between deflection bowls calculated using multi-layer linear elastic theory and CAPA-2D.

- Although the modelling of a loaded pavement is limited to specific cases, the importance of interlayer bond is illustrated. This has important implications in the design and construction of reinforced asphalt pavements, where often, it seems, more emphasis is placed on properties of the reinforcing material than on the bond between it and the adjacent asphalt layers.

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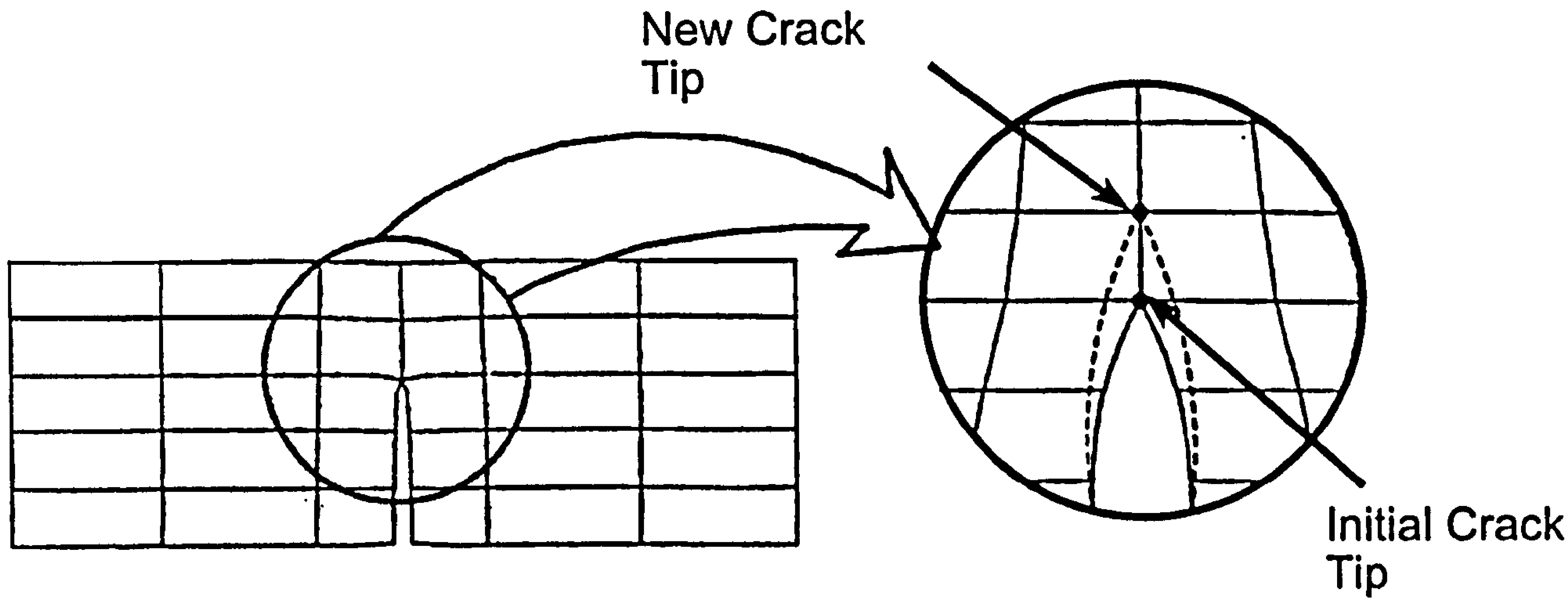


FIGURE 9.1
AUTOMATIC REMESHING ROUTINE:
CAPA-2D

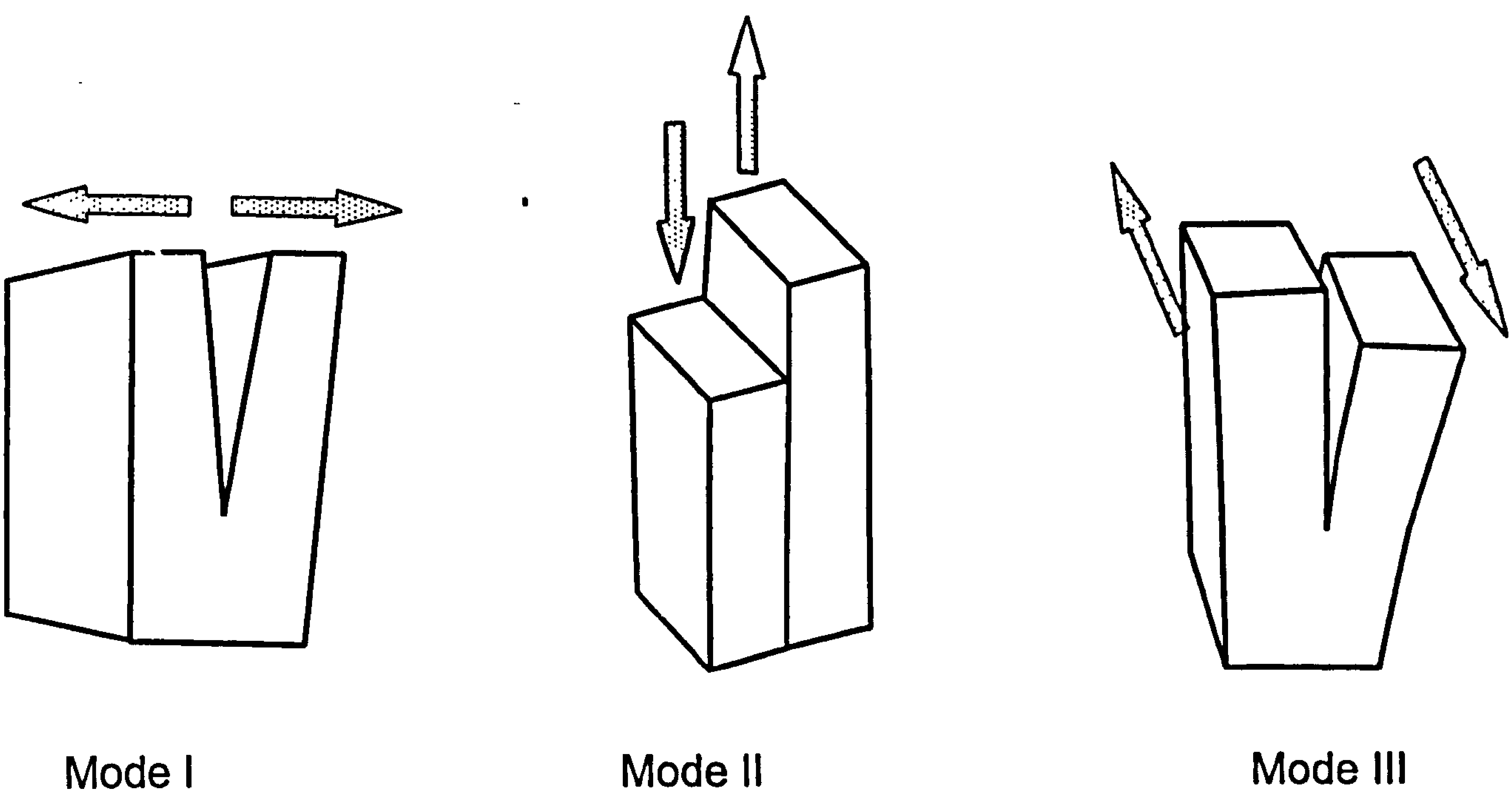


FIGURE 9.2
MODES OF CRACKING

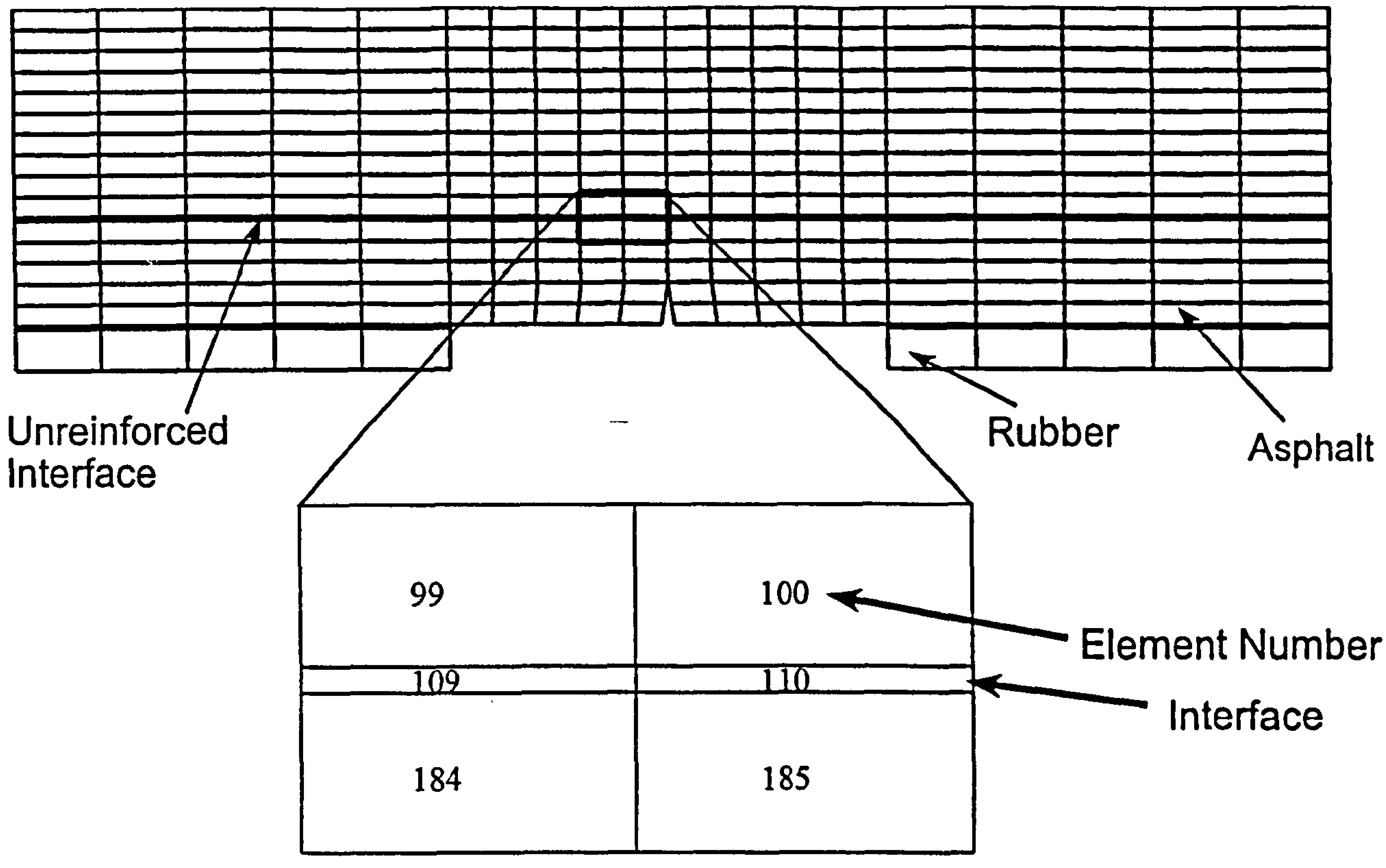


FIGURE 9.3

FINITE ELEMENT MODEL: UNREINFORCED BEAM

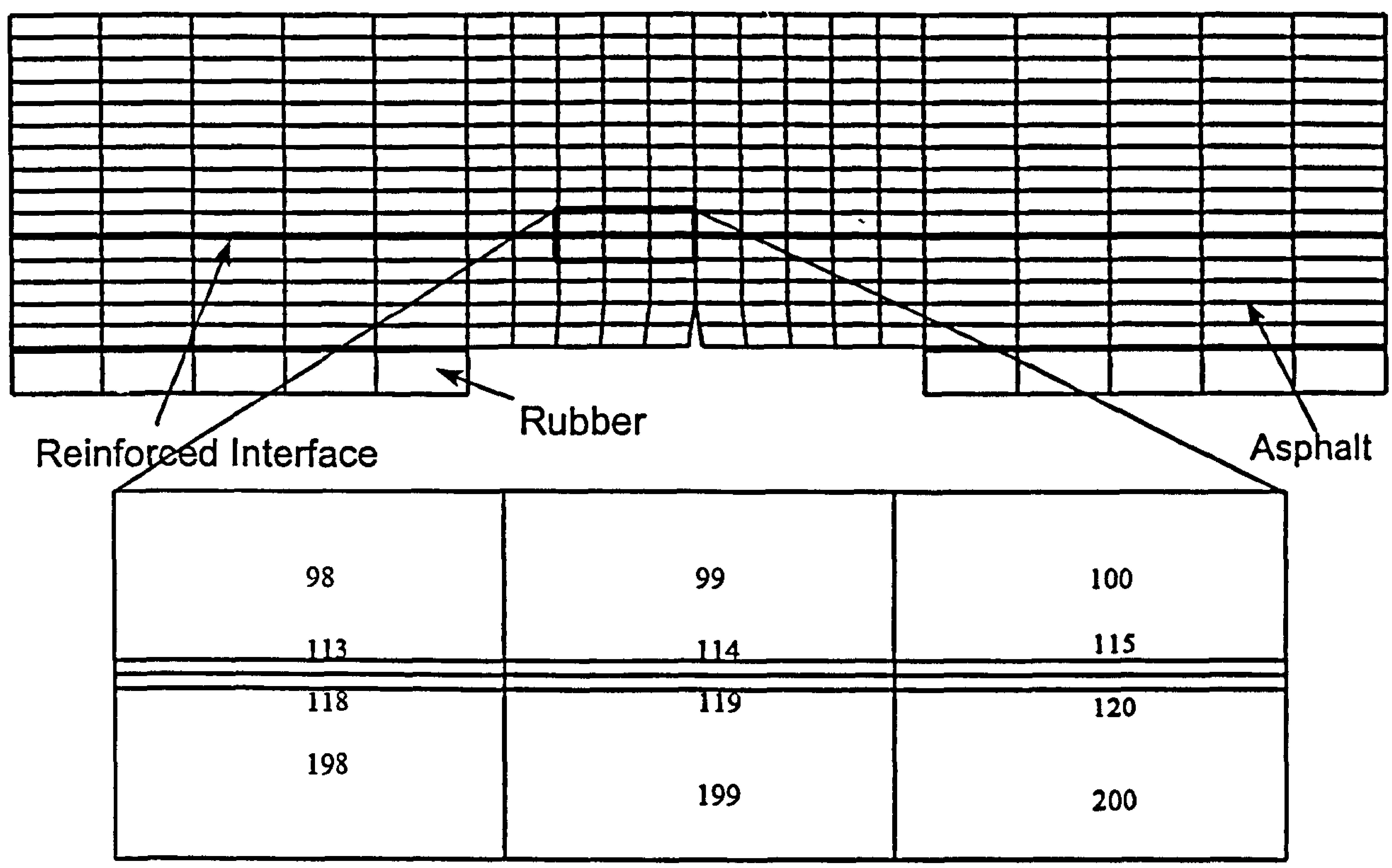


FIGURE 9.4

FINITE ELEMENT MODEL: REINFORCED BEAM

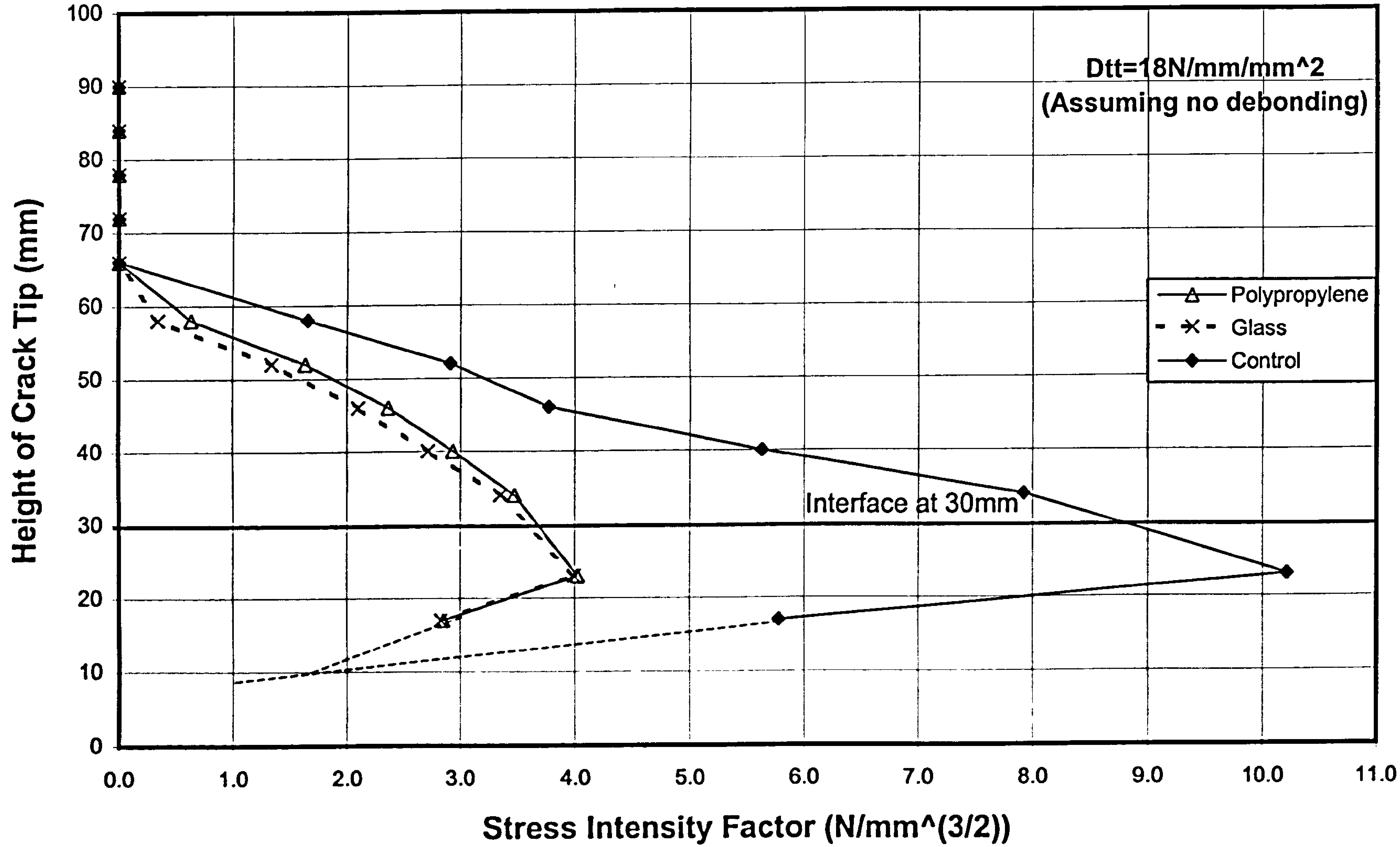


FIGURE 9.5
BEAM MODELLING: SIF versus HEIGHT

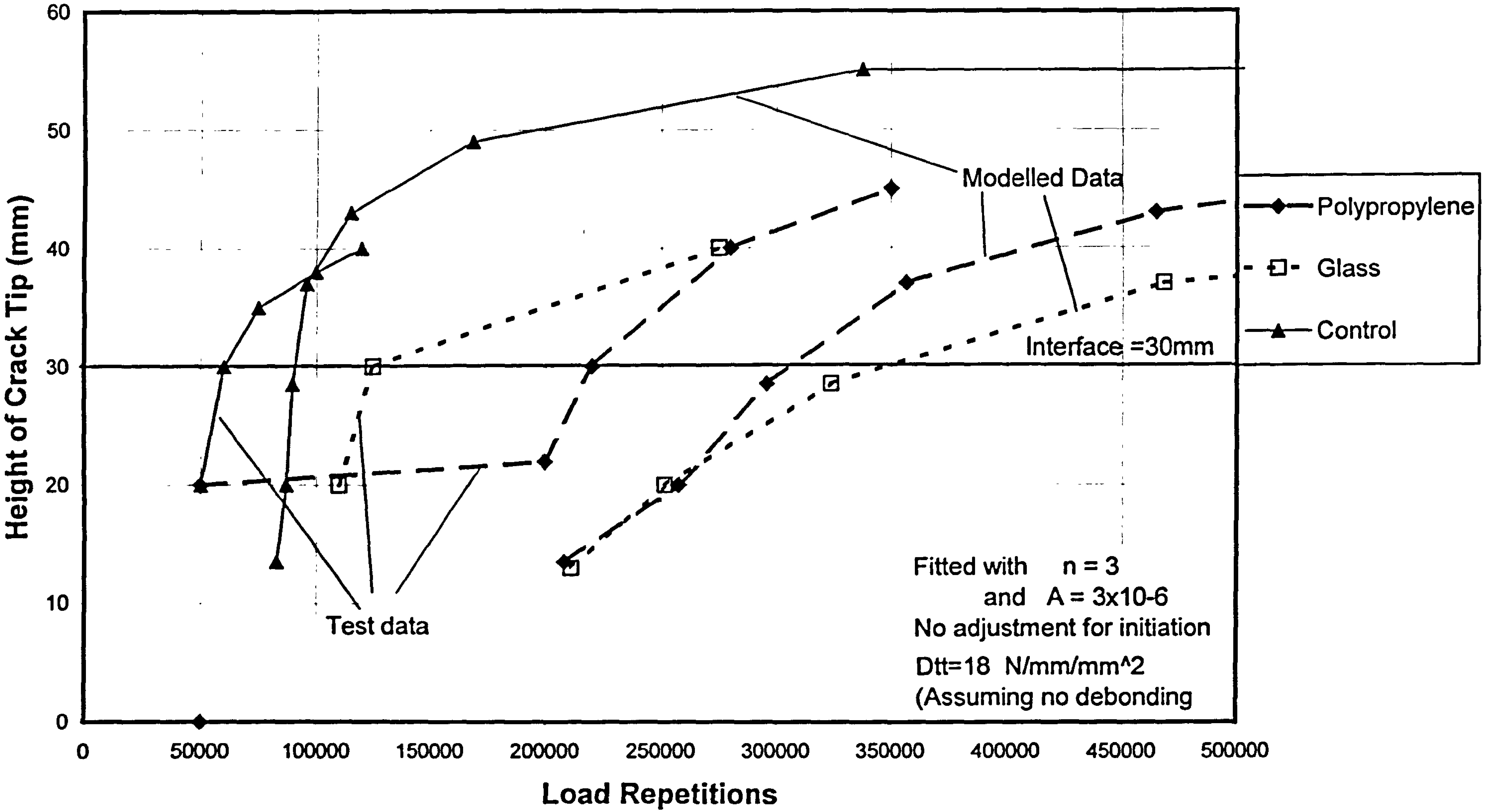


FIGURE 9.6a
MODELLING OF BEAM TEST: CRACK HEIGHT versus REPETITIONS

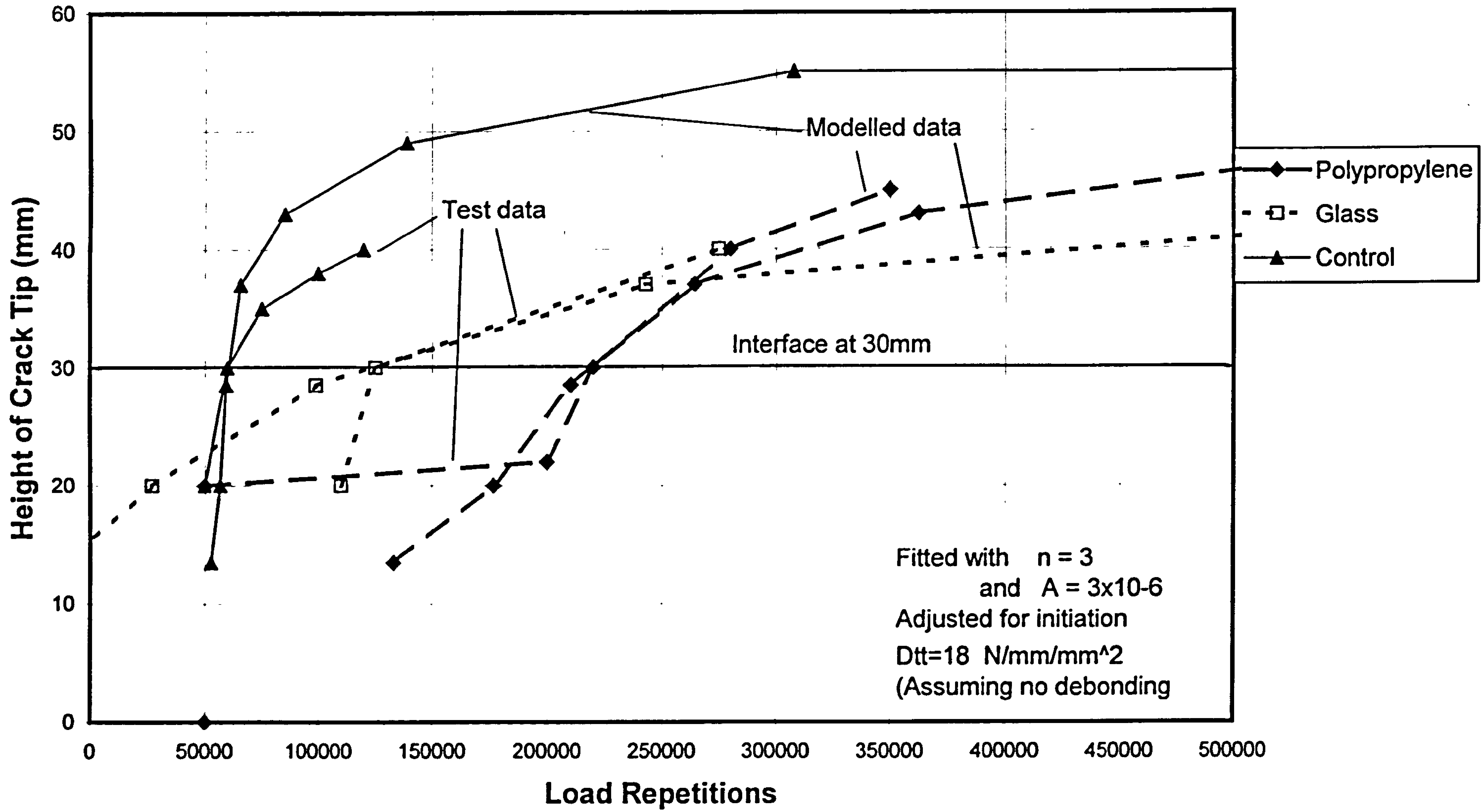


FIGURE 9.6b
MODELLING OF BEAM TEST: CRACK HEIGHT versus REPETITIONS

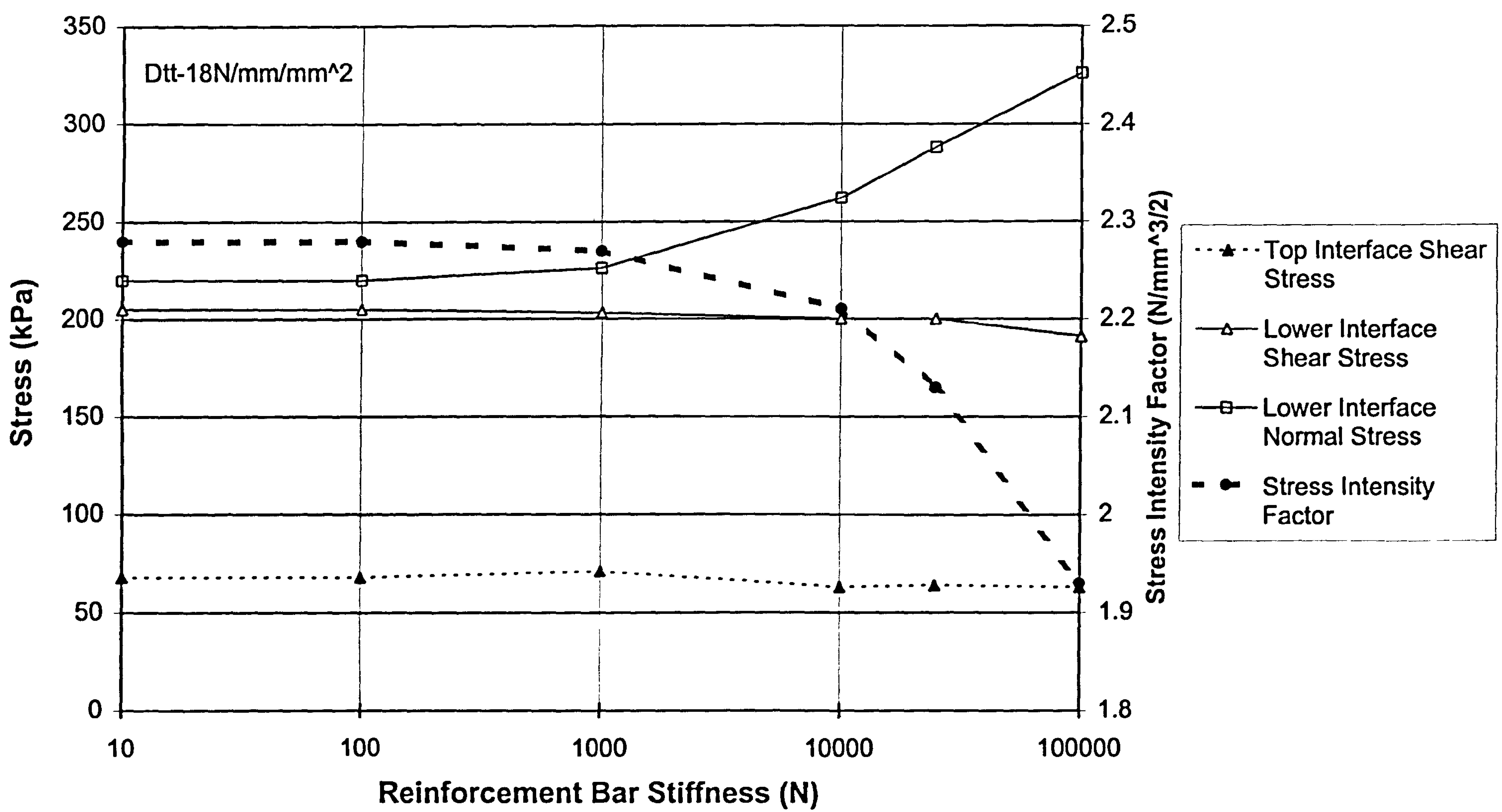


FIGURE 9.7

INFLUENCE OF REINFORCEMENT STIFFNESS

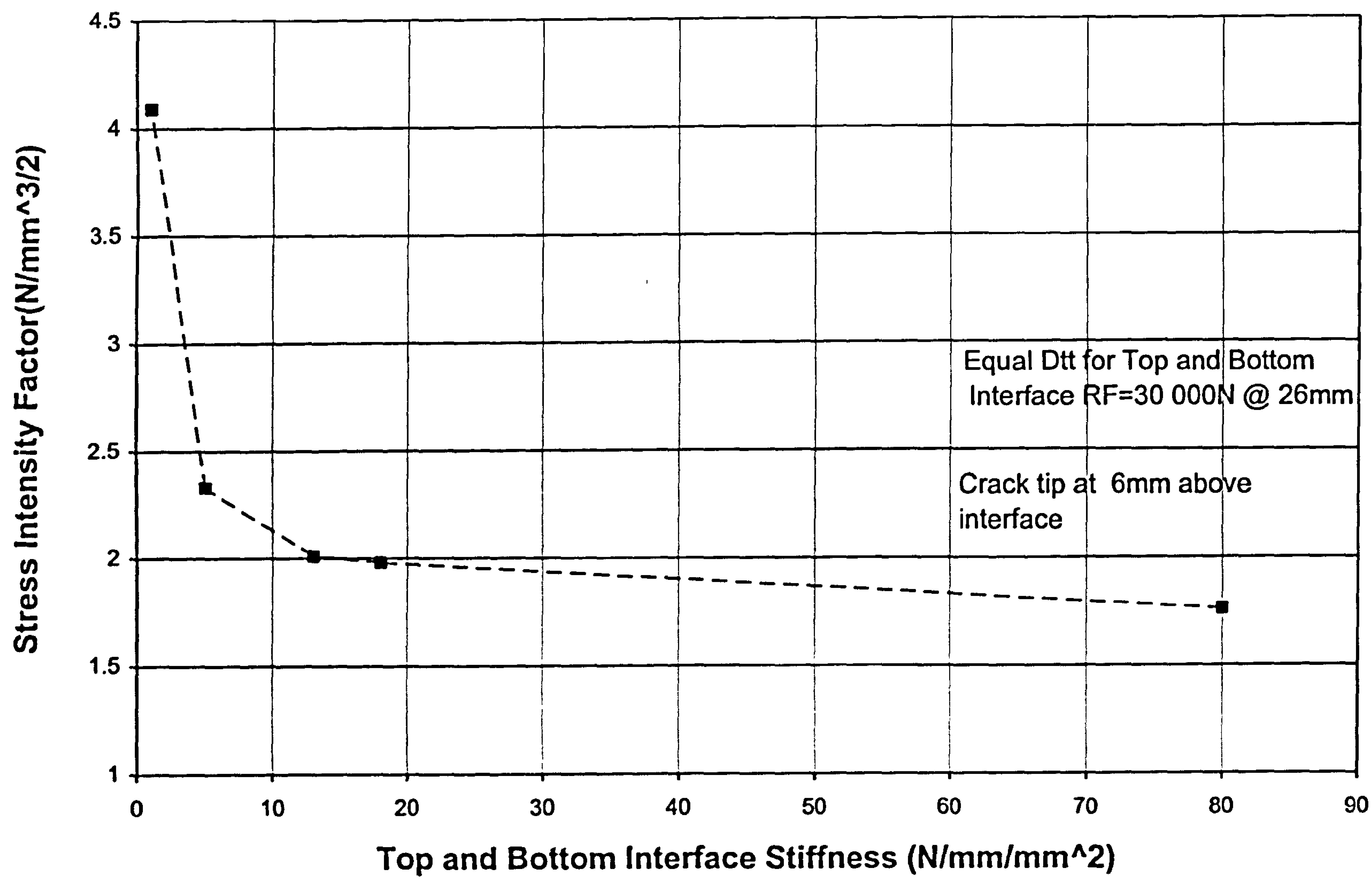


FIGURE 9.8

INFLUENCE OF BOND STIFFNESS ON SIF

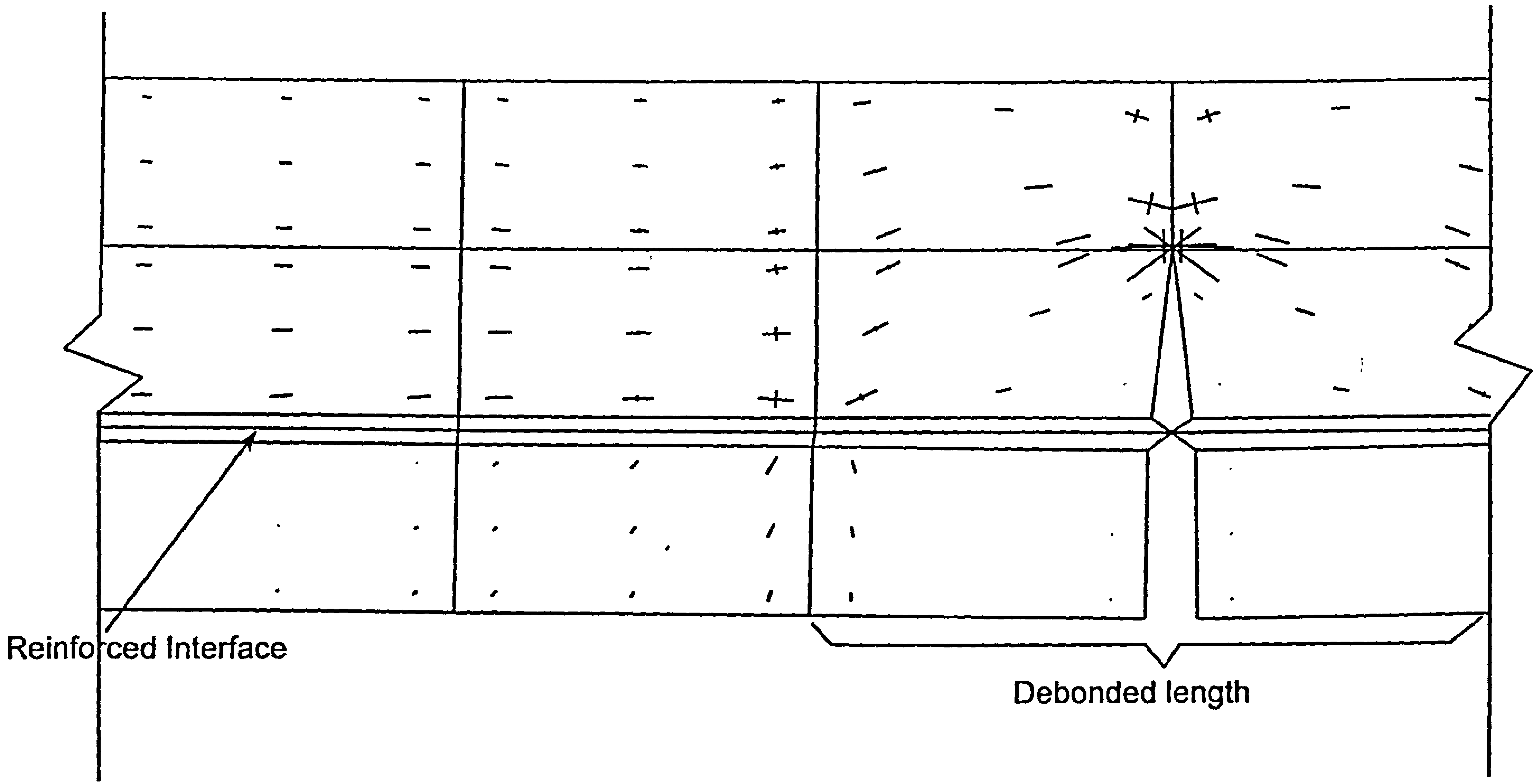


FIGURE 9.9
CAPA-2D STRESS ANALYSIS-DEBONDING OF REINFORCED INTERFACE: DETAIL OF ORIENTATION OF TENSILE STRESSES AT CRACK INCREMENT No.1

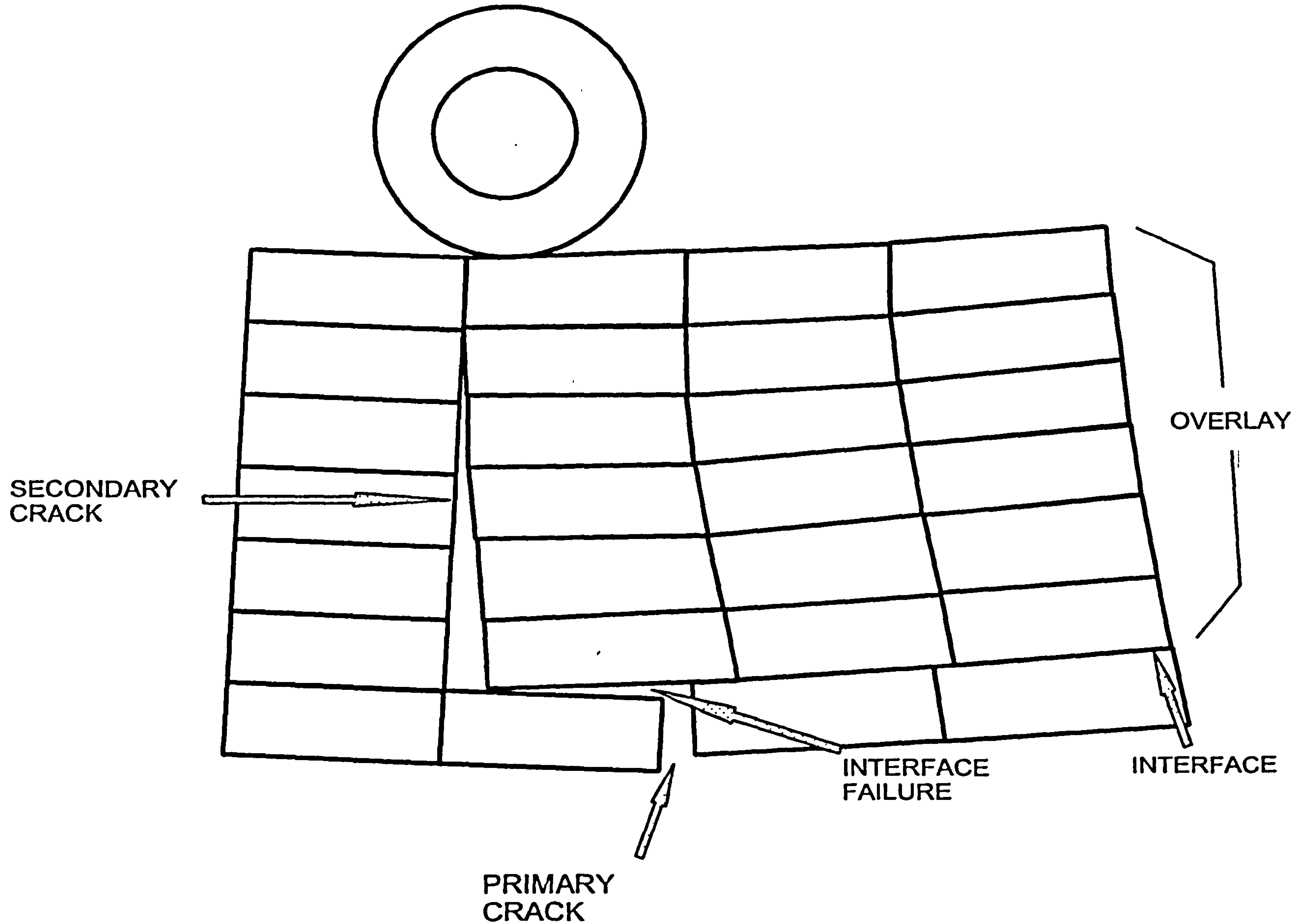
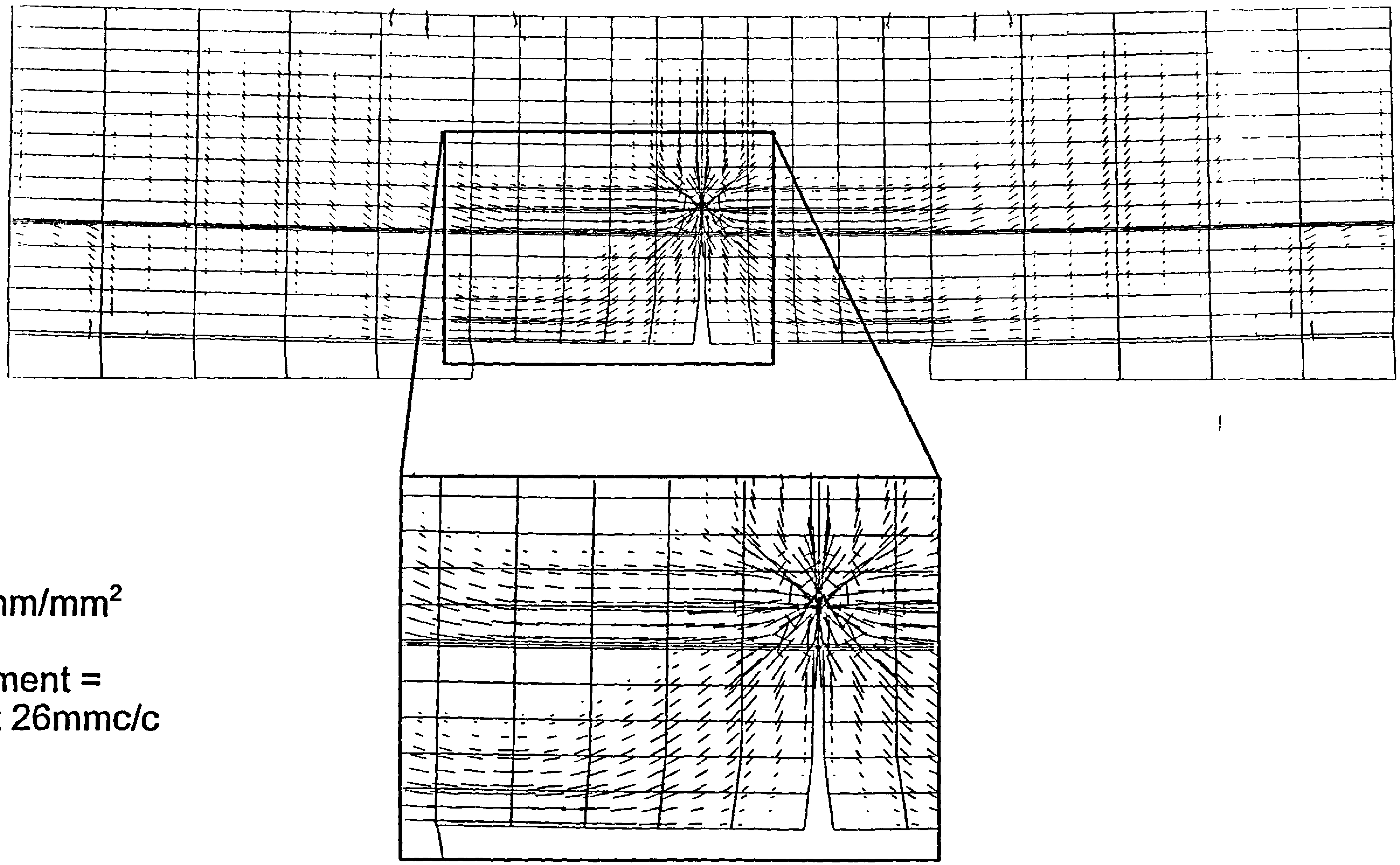


FIGURE 9.10
PRIMARY AND SECONDARY CRACKING

FIGURE 9.11
TENSILE STRESS DISTRIBUTION FOR CRACK INCREMENT 1 ABOVE THE INTERFACE AND DEBONDING BELOW THE REINFORCEMENT OF 25mm



Dtt=18N/mm/mm^2

Reinforcement =
30000N at 26mmc/c

FIGURE 9.12
TENSILE STRESS DISTRIBUTION FOR CRACK INCREMENT 1 ABOVE THE INTERFACE (NO DEBONDING)

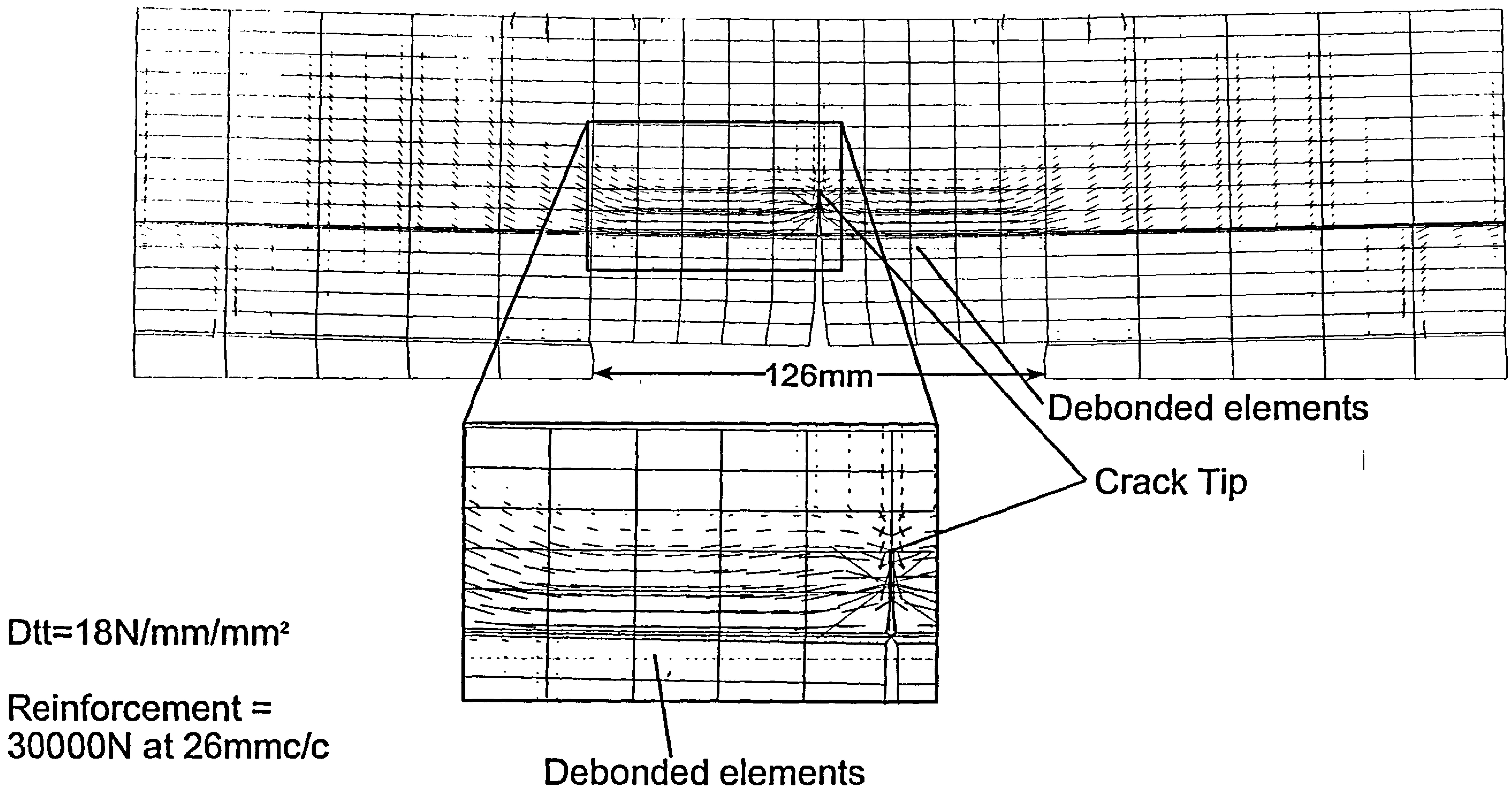
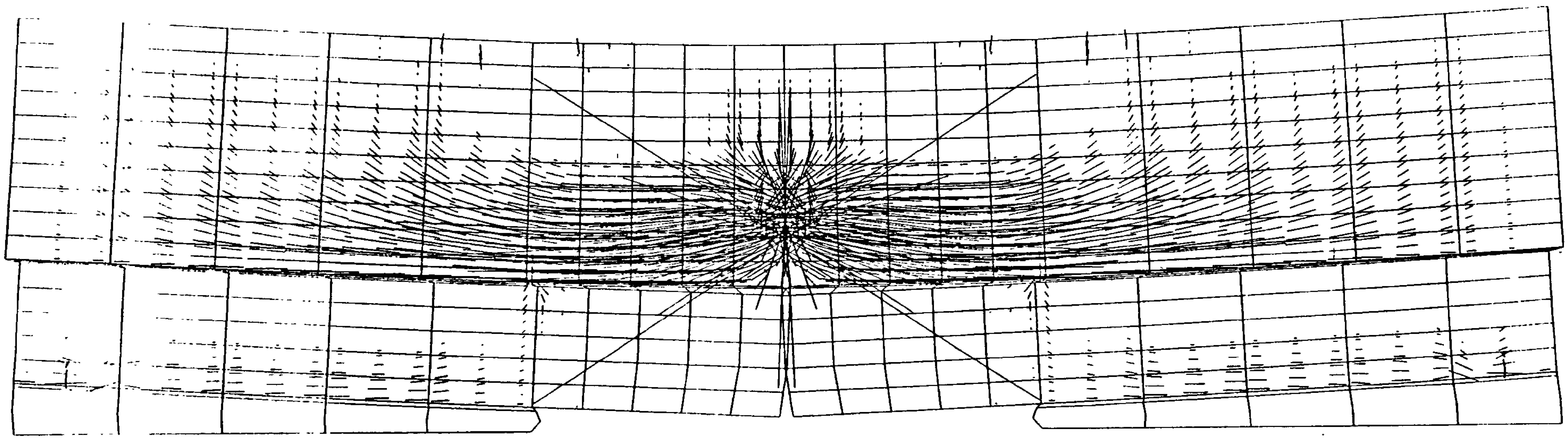


FIGURE 9.13
TENSILE STRESS DISTRIBUTION FOR CRACK INCREMENT 1 ABOVE THE INTERFACE AND DEBONDING OF 126mm



Dtt=18N/mm/mm^2

Reinforcement = 30000N
at 26mm c/c

FIGURE 9.14
TENSILE STRESS DISTRIBUTION FOR CRACK INCREMENT 1 ABOVE THE INTERFACE WITH ALL INTERFACE ELEMENTS DEBONDED

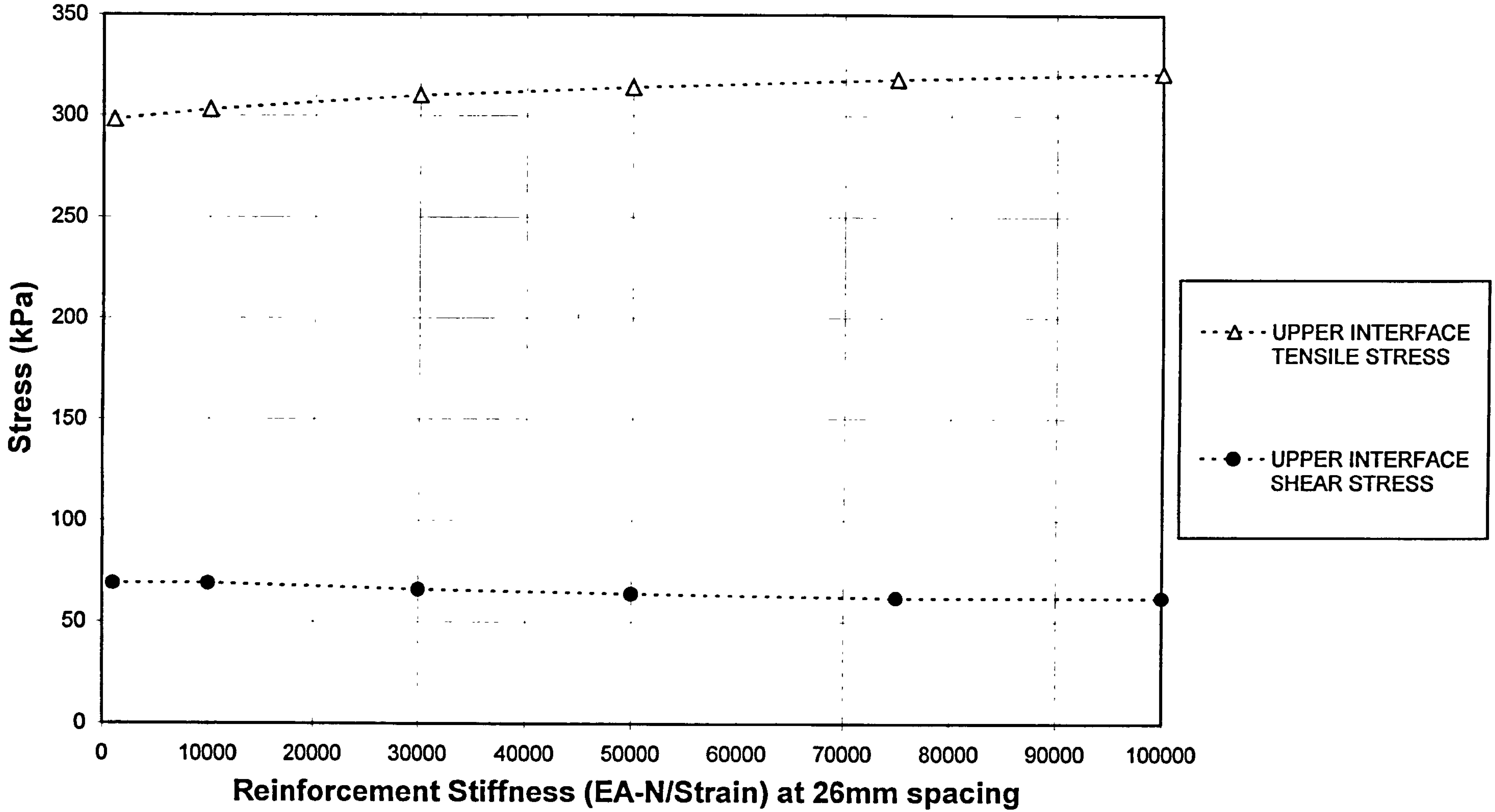


FIGURE 9.15
EFFECT OF REINFORCEMENT STIFFNESS ON STRESS ABOVE THE INTERFACE WITH DEBONDING OF 25mm

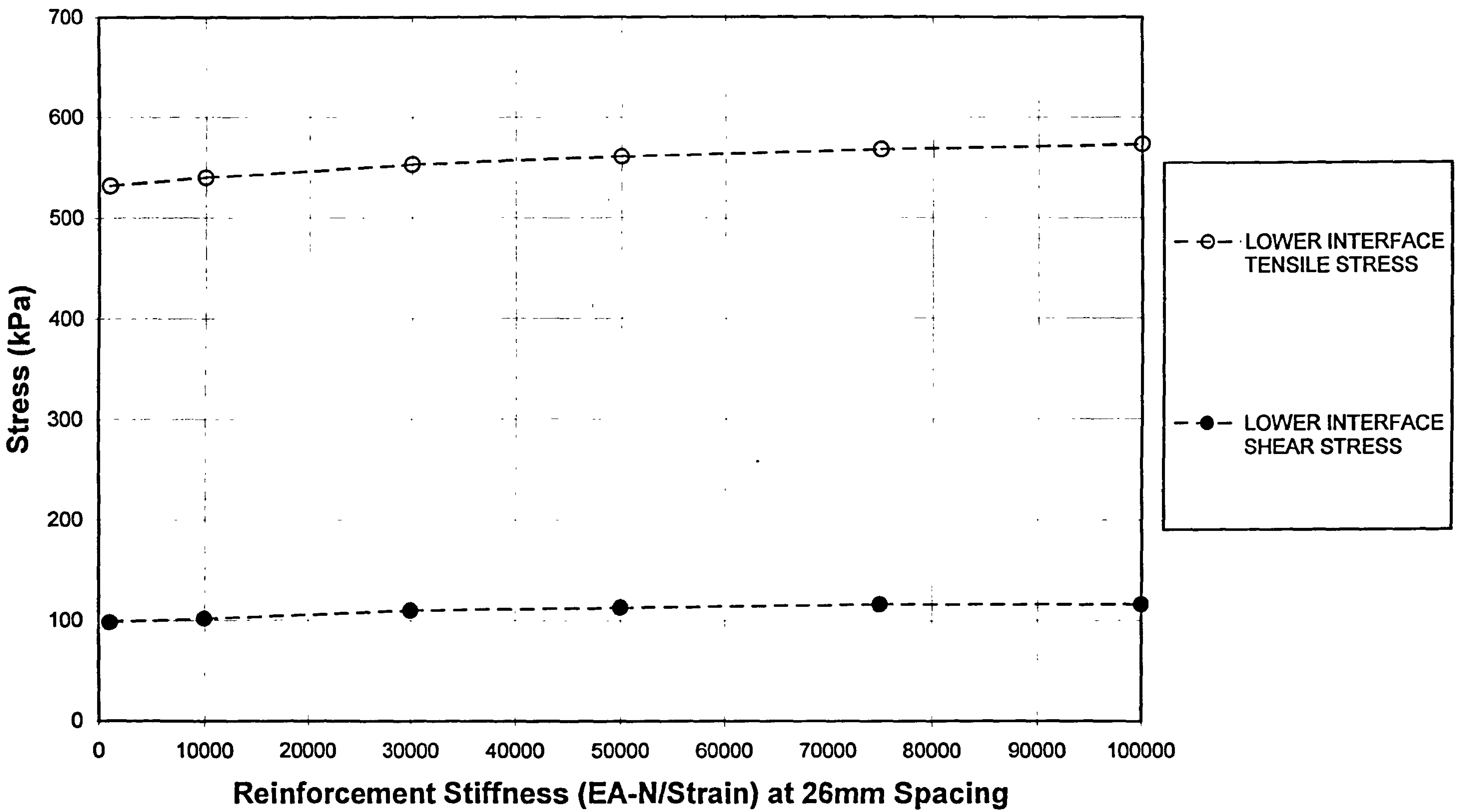


FIGURE 9.16

EFFECT OF REINFORCEMENT STIFFNESS ON STRESS BELOW THE INTERFACE WITH 25mm DEBONDING

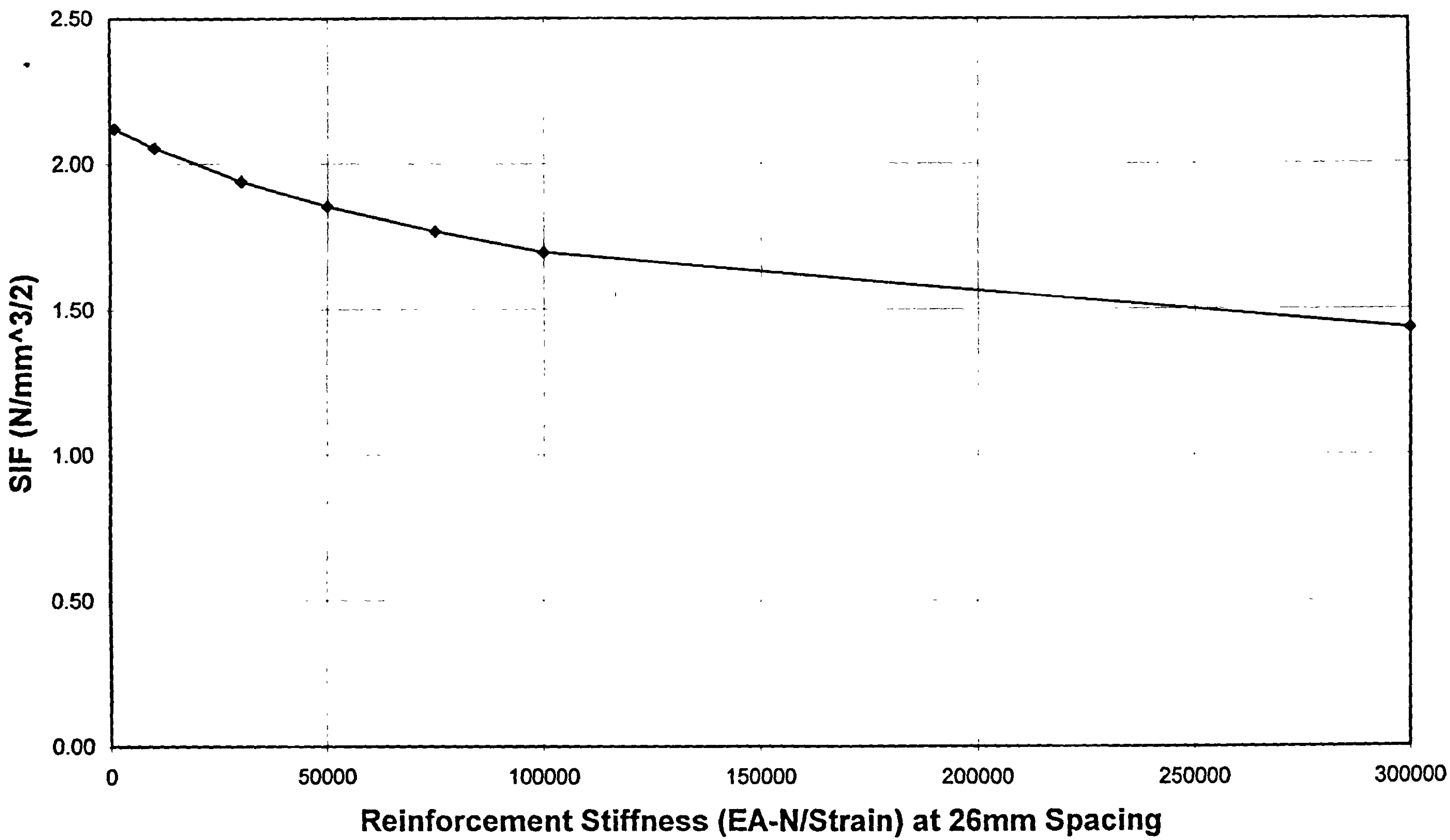


FIGURE 9.17
EFFECT OF REINFORCEMENT STIFFNESS WITH
25mm DEBONDING

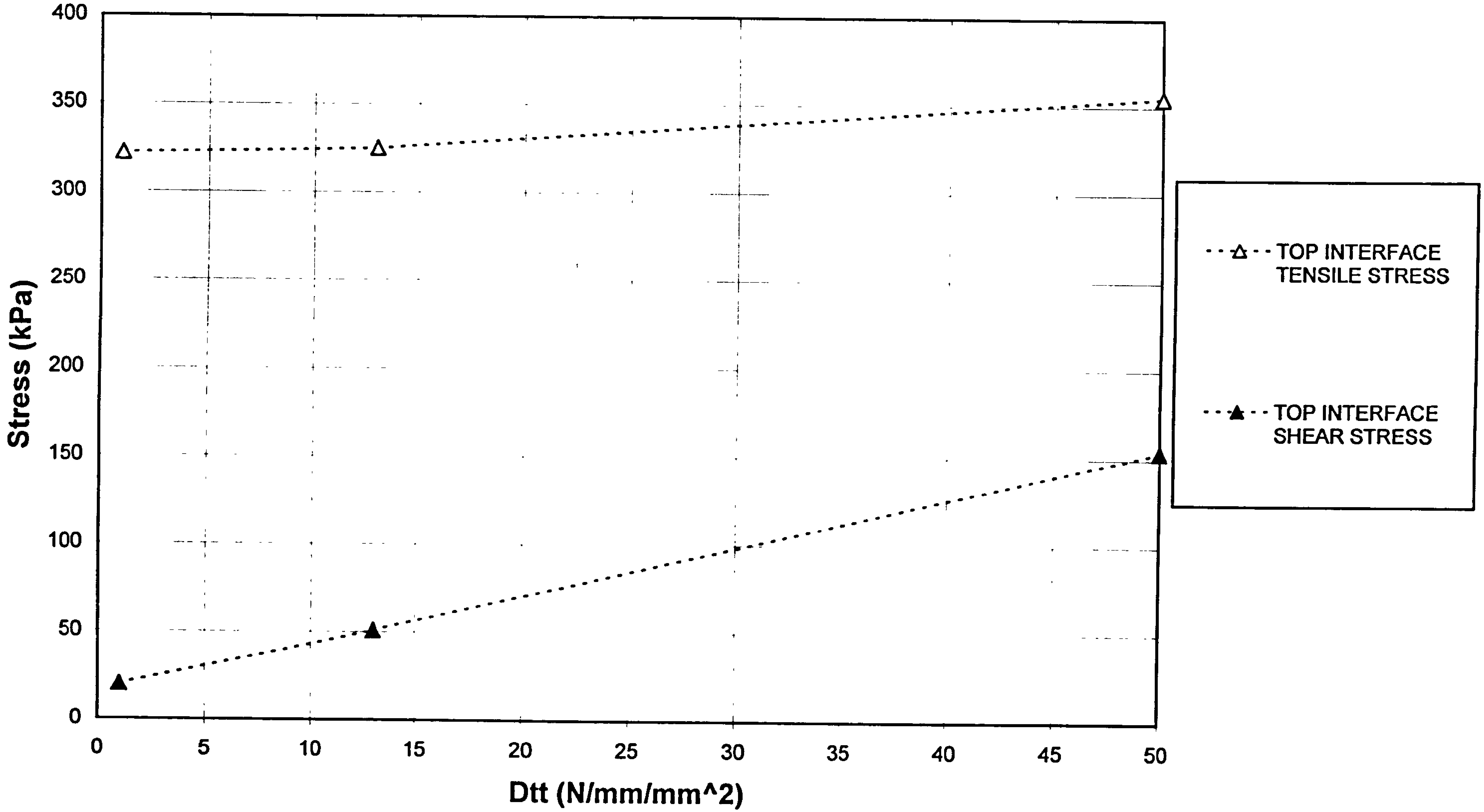


FIGURE 9.18
EFFECT OF BOND STIFFNESS ON STRESSES ABOVE THE INTERFACE WITH 25mm DEBONDING

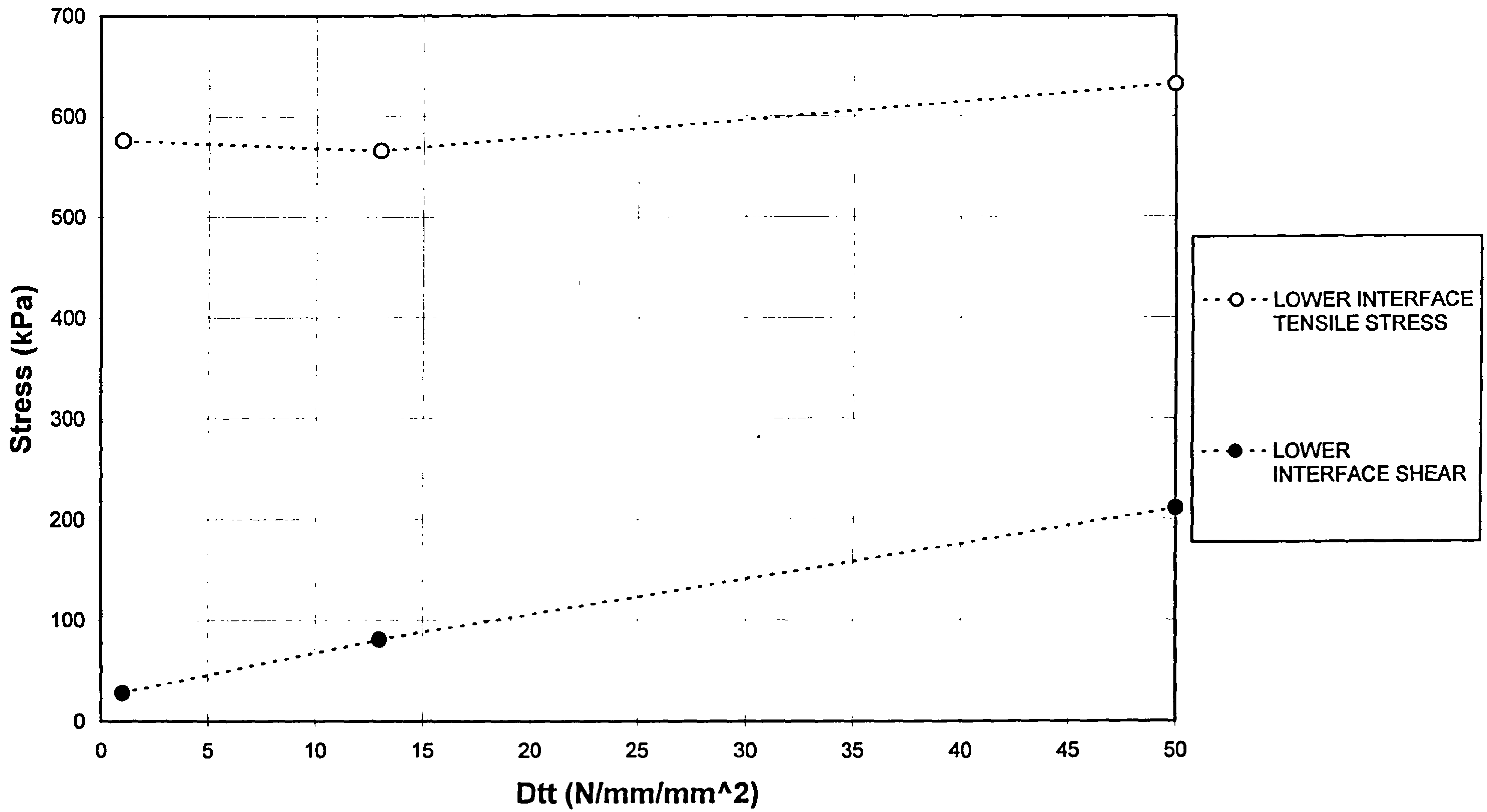


FIGURE 9.19
EFFECT OF BOND STIFFNESS ON STRESSES BELOW THE INTERFACE WITH 25mm DEBONDING

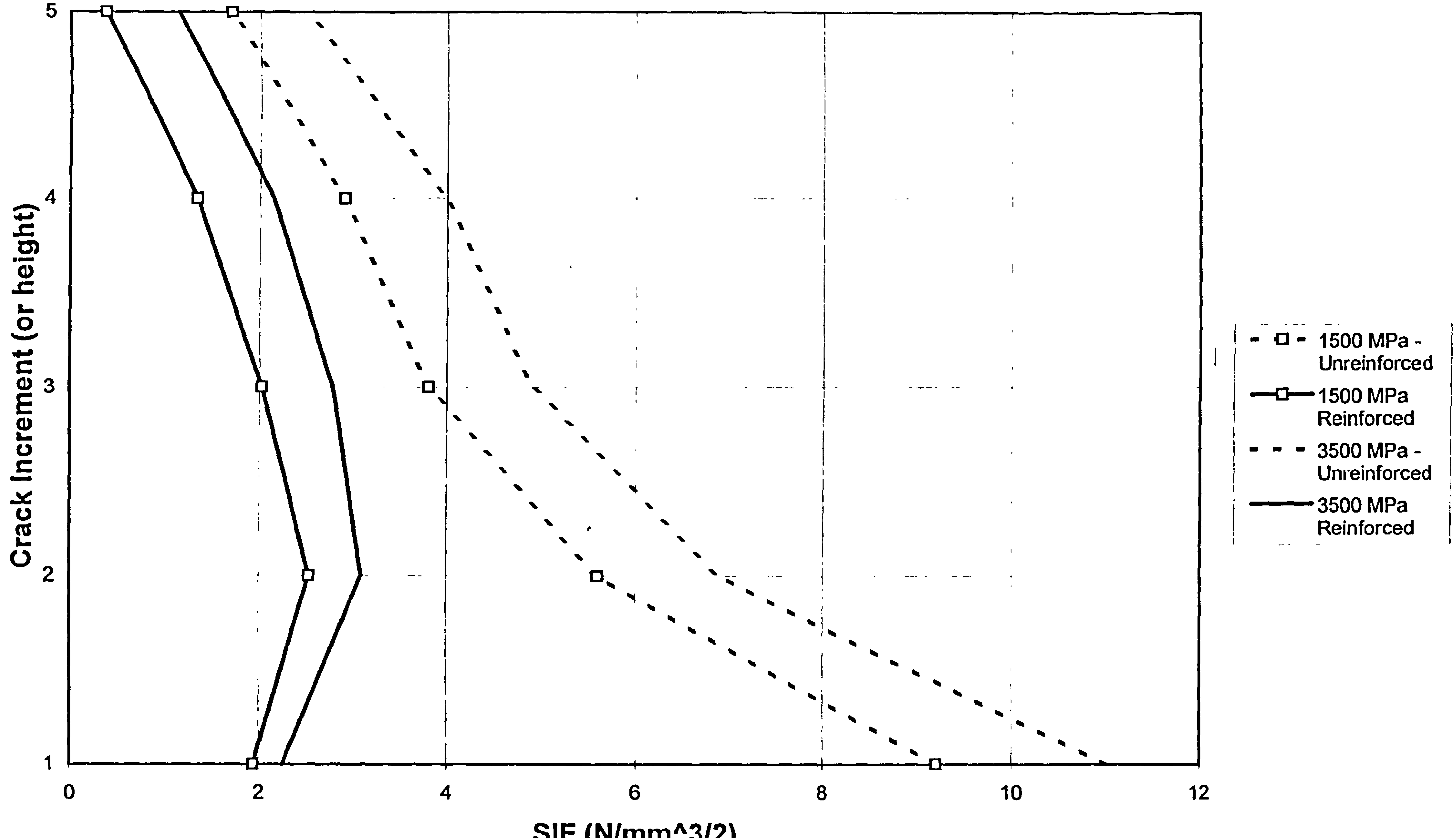


FIGURE 9.20
EFFECT OF ASPHALT STIFFNESS ON SIF

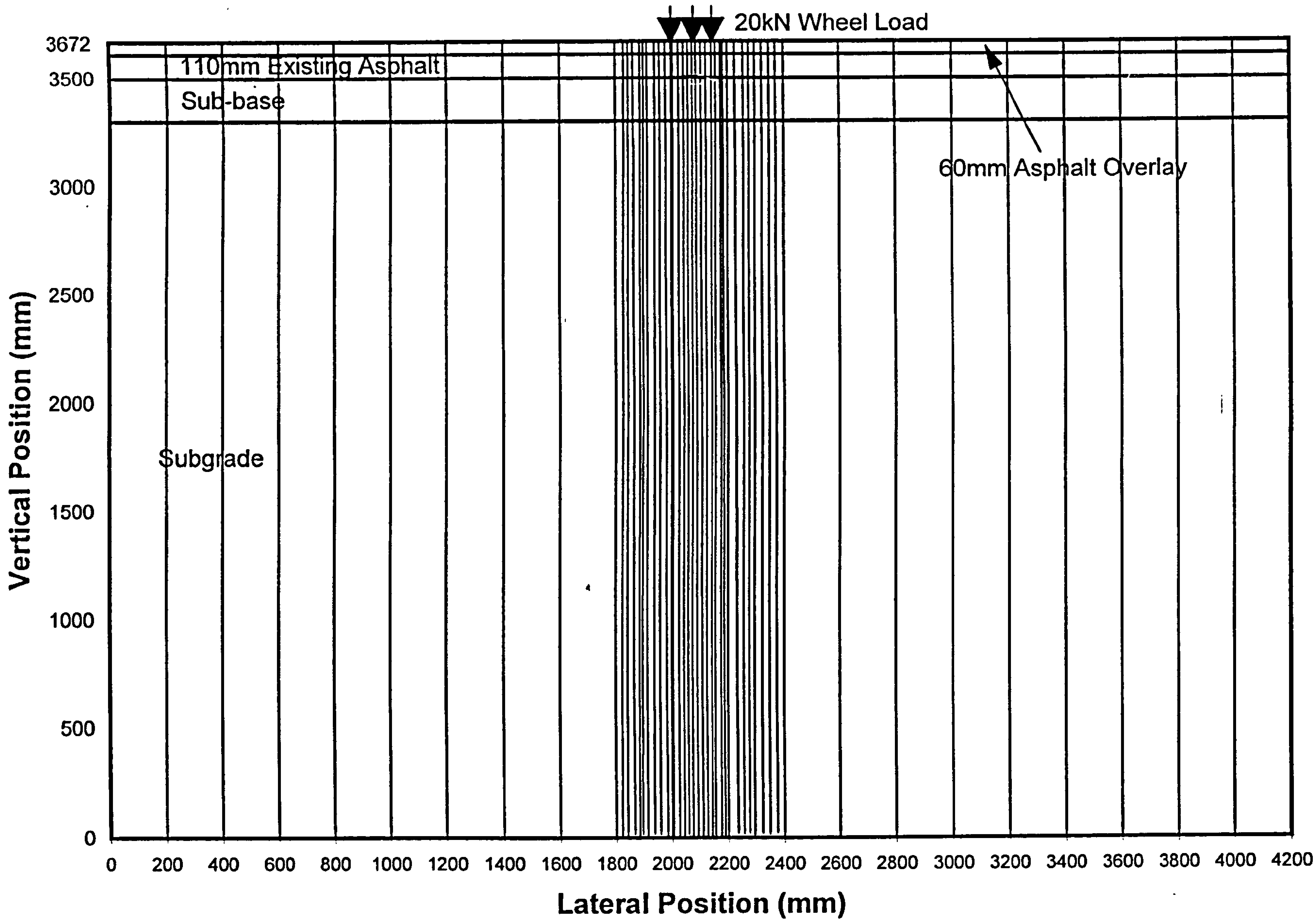


FIGURE 9.21
CAPA-2D PAVEMENT DESIGN MESH

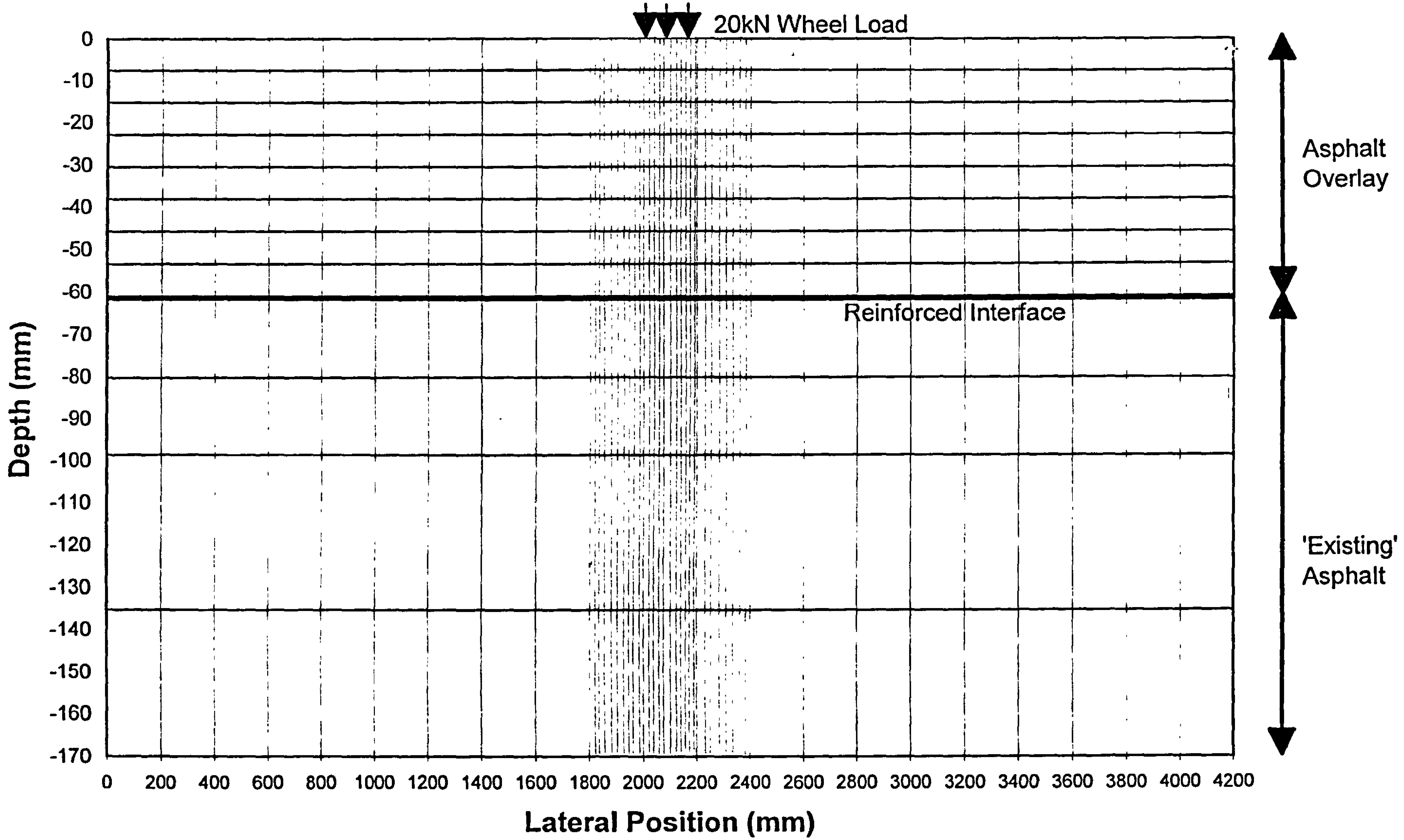


FIGURE 9.22
CAPA-2D PAVEMENT DESIGN MESH-DETAIL

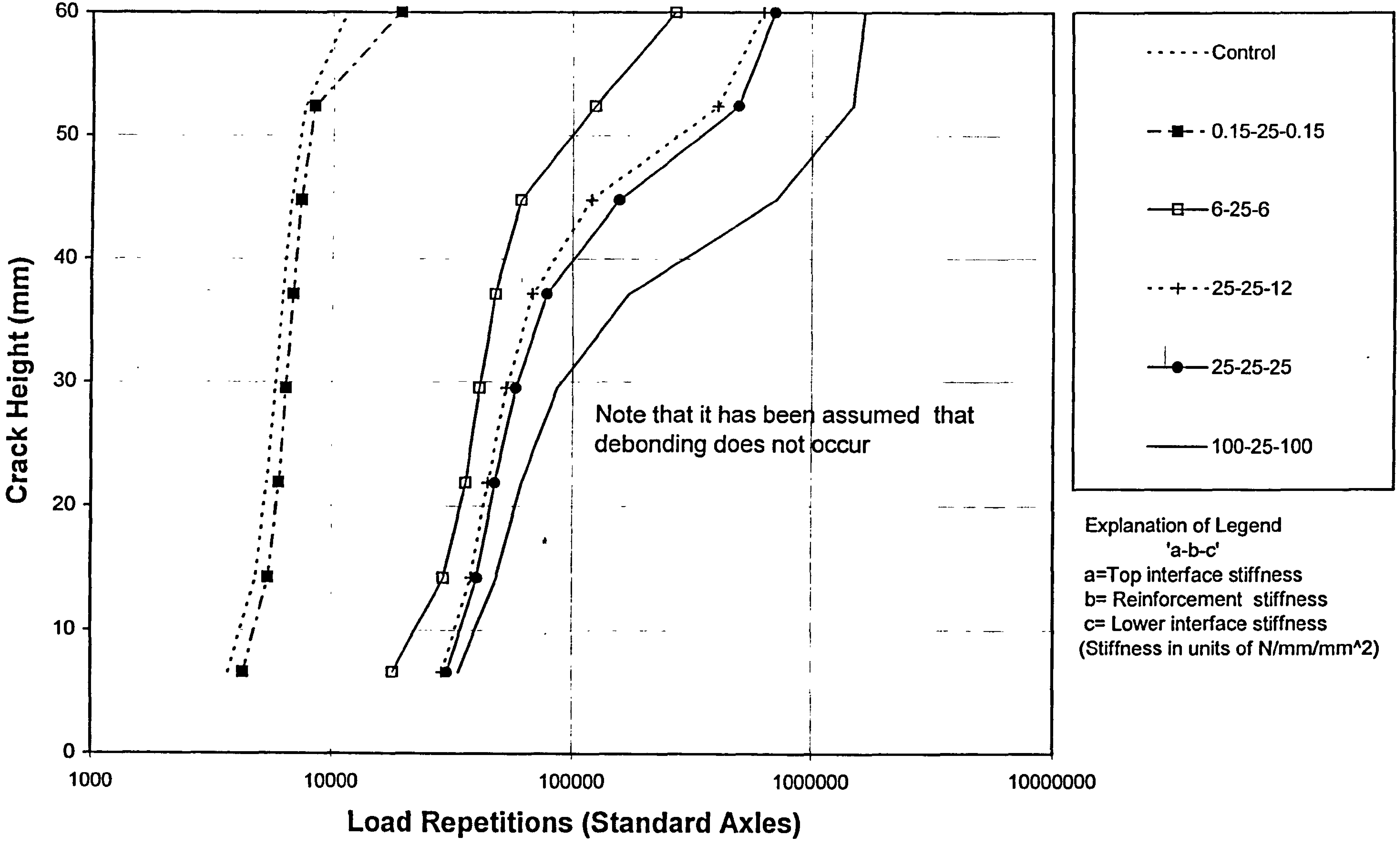


Figure 9.23
Crack Propagation: 60mm Overlay on a 'Weak' Pavement

CHAPTER 10

ECONOMIC APPRAISAL: WHOLE LIFE COSTING

CONTENTS

CHAPTER 10

ECONOMIC APPRAISAL: WHOLE LIFE COSTING

10.1 General

As noted in the literature review, summarised in Chapter 3, the issue of economic viability is ultimately the most important issue when assessing the possible use of reinforced asphalt (or any maintenance treatment). Notwithstanding the importance of this issue, the means whereby a fair comparison of maintenance treatments can be made is however, not clear-cut, and is normally either very simple, or relatively complicated through the need to take into account 'all' cost-related issues. The simple approach is usually only concerned with the supply and installation costs of alternative treatments, with the assumption that treatments have the same longevity. The more complicated approach, on the other hand, takes into account the whole life-cycle costs of a pavement.

Clearly, the validity of the 'simple' approach mentioned above is questionable, as the longevity of alternative treatments is seldom the same, and can vary considerably depending on a range of factors. This then suggests that the second more involved approach should be used to estimate the benefits. Accordingly, a Whole Life Costing (WLC) approach has been used, as described below.

10.2 The Whole Life Costing (WLC) Concept

WLC is a technique used to take into account all the costs of an asset (in this case a pavement) over the life of the asset. In simple terms, WLC takes the maintenance regime of the asset in question and applies present-day costs to the treatments applied. These are then discounted to a common date where they can be directly compared.

For pavements, the total cost is broadly divided into two main components; 'Works' costs and 'User' costs. Works costs are the construction costs incurred during initial construction and subsequent maintenance, and user costs are costs to the road user (the public) incurred as a result of road condition and maintenance activities. User costs can have a large effect on whole life costs and include costs due to

- congestion-induced traffic delays,
- pavement roughness (variations in the profile leading to increased fuel consumption),
- accidents at roadworks,
- skidding accidents

The model used to calculate WLCs for reinforced and unreinforced pavements has been described by Abell [10.1], and was developed primarily to assess the relative economic benefits of alternative new constructions. The model has around 300 data items that can be altered by the user, which relate to the following:

- New construction cost
- Maintenance works cost
- Traffic delay cost due to congestion at roadworks
- Cost of accidents at roadworks
- Skidding accidents
- Fuel increase with pavement unevenness
- Residual value at the end of the evaluation period
- Allowance for residual delay costs for major treatments after the evaluation period.

Details of each of these components can be found in Reference 10.1.

As noted above, the calculation of whole life costs depends on maintenance strategies, i.e. when, and in what circumstances different treatments are applied. In turn, the maintenance strategies depend on policy decisions and sets of rules and relationships that relate pavement deterioration to time or traffic.

10.3 Deterioration Modes

For bituminous pavements the deterioration mode normally used is the traffic-dependant mode. This uses deflection of the pavement to determine when and what thickness overlays are applied (or when reconstruction is carried out). Rutting is assumed to be a surface defect that is remedied by resurfacing. In reality, rutting may of course also be structural and so indicate a 'weak' pavement. If this is the case however, deflections will also be high, and will trigger the application of an overlay, therefore also removing any rutting in the surface.

Skidding resistance has been shown to vary with the number, and not merely the weight of heavy vehicles using the road. To correct any skidding defects, the model applies a surface dressing.

In addition to treatments triggered by traffic, there are other activities that are scheduled on a time basis. These include the repair (patching) of randomly-occurring defects such as potholes.

Where pavement deterioration dictates that an overlay is to be applied near the end of the evaluation period, it leaves the pavement with considerable strength (or 'life'). To take this into account in the calculations, the values of the proportion of life remaining in the overlay is removed from the maintenance costs. This can have a large effect on WLCs and the relationship between the length of the evaluation period and the application of the last overlay should be noted when assessing results. Linked to this are the traffic delay costs, which are largest at pavement strengthening works. If strengthening occurs just inside or outside the evaluation period, these costs can also have a significant effect on the overall magnitude of the WLC, and to counter this, an allowance for delays incurred at the first strengthening after the end of the evaluation period is included in the WLC.

Figure 10.1 gives the overall logic of the model.

10.4 Modelling

The examples used to model the effect of reinforced asphalt on WLC are carried out from the perspective of a highway authority, where the overall costs (both user and works costs) are important. It is appreciated that user costs may not be an important factor for some organisations to consider in the short-term, such as, for instance, a maintenance section of a local authority road where the pressures of short-term construction costs may be dominant. Nonetheless, in view of the government's commitment to appraising works on a whole life cost basis and moves towards sustainable development, it will be increasingly important for all organisations involved in the maintenance of roads to take all likely (predictable) long and short-term costs into account.

The pavement types and conditions shown in Table 10.1 were used to estimate the cost-effectiveness of reinforcement.

Table 10.1 Initial Conditions Assumed for Calculation

	Single Carriageway	**Dual Carriageway (APTR)**	**Dual Carriageway (D3M)**
Length (m)	1000	1000	1000
Width (m)	9.0	8.6	14.3
Traffic Flows (vph)	200 400 800	200 800 1500	400 800 2500
Deflection (0.01mm)	21	13	13
Rut Depth (mm)	10	9	9
SFC	0.35	0.37	0.37
Traffic since last strengthening (msa)	10	91	91

The analysis period used was 30 years, and a 6% discount rate was assumed. As existing pavements were modelled, initial construction costs were not included in the calculations. Treatment costs used are given in Table 10.2.

Table 10.2. Treatment Costs

Treatment	Surface Dressing	Overlay	Resurfacing	Thin Wearing Course
Cost (£/m^2)	3.3	15 to 25	8	4.5

For each of the traffic flow bands:

- WLCs were calculated for an unreinforced pavement.
- The times at which resurfacing and overlay treatments were applied (taken from the maintenance profile) were noted.
- Works and user costs were noted.
- For the reinforced pavement, 'fixed interventions', (i.e. selected treatments applied at set times) were used to model the effect of the reinforced layer, taking into account the times that treatments were applied for the unreinforced pavement.
- Works and user costs were then taken from the output.
- The discounted cost of reinforcement was added to construction costs.

In estimating the maintenance treatment regimes when reinforcement is used, it has been assumed that cracking and rutting develop at half the rate of that of an unreinforced pavement. Consequently, the longevity of treatments for rutting and cracking problems is increased by a factor of two. This factor was used as a consequence of the results of the beam and Pavement Test Facility (PTF) tests earlier described. It is also assumed that the installation of the reinforcement is 'good' and premature failure through poor workmanship does not occur.

The costs of supplying and installing reinforcement to a pavement varies considerably depending on factors such as the size of the contract, the type of reinforcement used and the geographic location. A range of prices from £2.0/m^2 to £5/m^2 has therefore been used and was taken from information made available by reinforcement suppliers and a specialist contractor.

The difference in values of WLC for both the reinforced and unreinforced pavements were calculated and are plotted in Figures 10.2, 3 and 4. The figures illustrate the effects of grid installation costs and traffic flow (and therefore user costs) on WLC.

Pavement structures used for analysis

The pavement structures shown in Table 10.3 were used through the analysis.

Table 10.3 Pavement structures used in the Analysis.

Traffic Flow (vph)	**Single Carriageway**	**Dual Carriageway (APTR)**	**Dual Carriageway (D3M)**
Surface Course (mm)	40	40	40
Binder Course (mm)	60	125	125
Roadbase (mm)	260	275	275
Subbase (mm)	150	150	150

10.5 Results

The results of the analysis are given in Figures10.2, 3 and 4 as plots of reinforcement installation cost versus the difference between WLC of unreinforced and reinforced pavements.

For the set of parameters and data used in the model, the graphs show that

- For a given cost of grid installation larger reductions in WLC are found with higher traffic flows.
- The cost effectiveness of reinforcement does not increase linearly with an increase in traffic flow. This is due to rules within the WLC model, i.e. depending on the relationship between the traffic level and the pavement structure and the evaluation period, an overlay might be applied near the end of the evaluation period. This has the effect of reducing the WLC due to the large component of residual value.

It should be noted that in practice, relatively large variations in rates for maintenance treatments are common. This, coupled with significant variations in traffic management rates and the type of traffic mangement adopted could give different results for different combinations of factors. In addition, only common maintenance treatments have been assumed, with no options for recycling having been considered.

The position of the lines in Figures10.2, 3 and 4 can be better understood if the difference in interventions for reinforced and unreinforced pavements are noted. This information is given in Table 10.4.

Table 10.4 Details of Maintenance Interventions

Road Type	Traffic Flow (vph)	Unreinforced Pavement		Reinforced Pavement	
		Year	Treatment	Year	Treatment
Single Carriageway	200	3	SD	3	RS +RF
		11	SD		
		14	RS	17	SD
		22	SD		
		30	SD	30	SD
	400	3	SD	3	SD
		7	RS	7	RS+RF
		15	SD	15	SD
		23	OL	23	RS
	800	4	RS	4	OL+RF
		13	OL	18	RS
		20	RS	25	SD
		26	RS		
Dual All Purpose Trunk Road (APTR)	200	3	RS	3	RS+RF
		11	RS	11	SD
		19	RS	18	RS+RF
		27	RS		
	800	3	RS	3	RS+RF
		10	RS	10	SD
		17	OL	17	OL+RF
		24	RS	25	SD
	1500	3	RS	3	RS+RF
		10	RS		
		13	OL	13	OL+G
		20	RS		
		27	RS	30	RS
Dual 3-lane Motorway (D3M)	400	3	RS	3	RS
		11	RS		
		19	RS	13	OL+G
		27	RS		
		30	OL	27	RS
	1500	3	RS	3	RS
		12	OL	12	OL+G
		20	RS	19	TWC
		28	RS	26	RS
	2500	3	RS	3	RS
		10	OL	10	OL+G
		18	RS	17	TWC
		26	RS	23	RS

Key: SD represents Surface Dressing
RS Resurfacing
OL Bituminous Overlay
TWC Thin Wearing course
RF Reinforcement

It is noted that even though it might not be necessary to carry out a structural treatment on a pavement due to the presence of reinforcement, applications of surface dressing are still required to maintain the skid resistance. Furthermore, for a pavement with a low traffic flow, the user costs are similarly low, and the cost of the grid is relatively high. This helps to explain the information in Figure 10.2.

From the analysis the following points are noted:

(1) Comparison of different maintenance treatments is not simple, and depending on which factors are taken into account, can give a range of answers. This is further compounded by the range of longevity for any maintenance treatment.

(2) For the assumptions and conditions used in the modelling, the following observations are made regarding the cost effectiveness of reinforced asphalt over a 30 year analysis period:

Single carriageway roads: savings are possible if reinforcement costs less than £4.80/m^2 and traffic flows are greater than or equal to 200vph.

Dual carriageway roads: savings are possible if reinforcement costs less than £4.80/m^2 and traffic flows are greater than or equal to 800vph.

D3M roads: savings are possible if reinforcement costs less than £1.70/m^2 and traffic flows are greater than or equal to 400vph.

(3) Estimation of the cost effectiveness of reinforcement is not straightforward. The combination of types and costs of traffic management, costs of accidents, the value of time lost through congestion, and other secondary factors that determine these costs, make it difficult to be precise in predicting when reinforcement can be cost effective.

It is important to note that the above analysis represents particular sets of pavement conditions and traffic flows, maintenance treatment types and strategies. Also, possible benefits due to waterproofing effects of paving fabrics have not been taken into account, due to the difficulties in quantifying their effect on pavement performance.

Another important aspect of evaluating whole life costs is the estimation of the time taken to place the treatments, i.e. the rate of working. This in turn, is heavily influenced by the environment in which the treatments are to be carried out. For instance, if work can only be carried out on heavily trafficked pavements in urban areas at night, the allowable working window may be reduced to 6 hours or less, which increases cost rates and the elapsed time required to complete the works. Conversely, the same amount of maintenance on a rural road carrying modest traffic flows may be relatively quick and inexpensive to complete.

To make appropriate use of WLC data, the overall purpose and limitations of this approach must be taken into account. Whole life costing is a tool to enable comparison of alternative treatments and strategies on a common time-and-cost basis, and is particularly well-suited to scenario planning. However, it is difficult to predict traffic growth, construction costs and treatment performance with accuracy, thus making precise predictions of maintenance strategies difficult. Also, over an analysis period of 30 years, for instance, many unknowns relating to developments in materials and policy changes at national and local levels, are unknown. Notwithstanding these drawbacks, the approach remains a very useful technique to assess the long term cost implications of pavement maintenance.

10.6 Conclusions

- To evaluate the effect of a maintenance treatment, both the initial and consequential costs of the treatment need to be taken into consideration. In this regard, the Whole Life Cost approach can be a useful technique, and can take into account both works and user costs over a specified analysis period.

- Using the assumption that rut development and crack growth are reduced to half the rate of unreinforced material, the analysis shows that reinforcement appears to be more effective as traffic levels (and hence user costs) increase.

- Beyond a limited analysis period, Whole Life Costing is not likely to give accurate values of cost. However, the approach provides a means whereby different types of maintenance treatments applied at different times can be compared on an equal basis.

10.7 Reference

10.1 Abell, R, (1989). TRRL Whole Life Cost Model for Flexible and Rigid Pavements, Working Paper WP/PE/51, Pavements Department, Infrastructure Division, Transport Research Laboratory, Crowthorne, Berkshire.

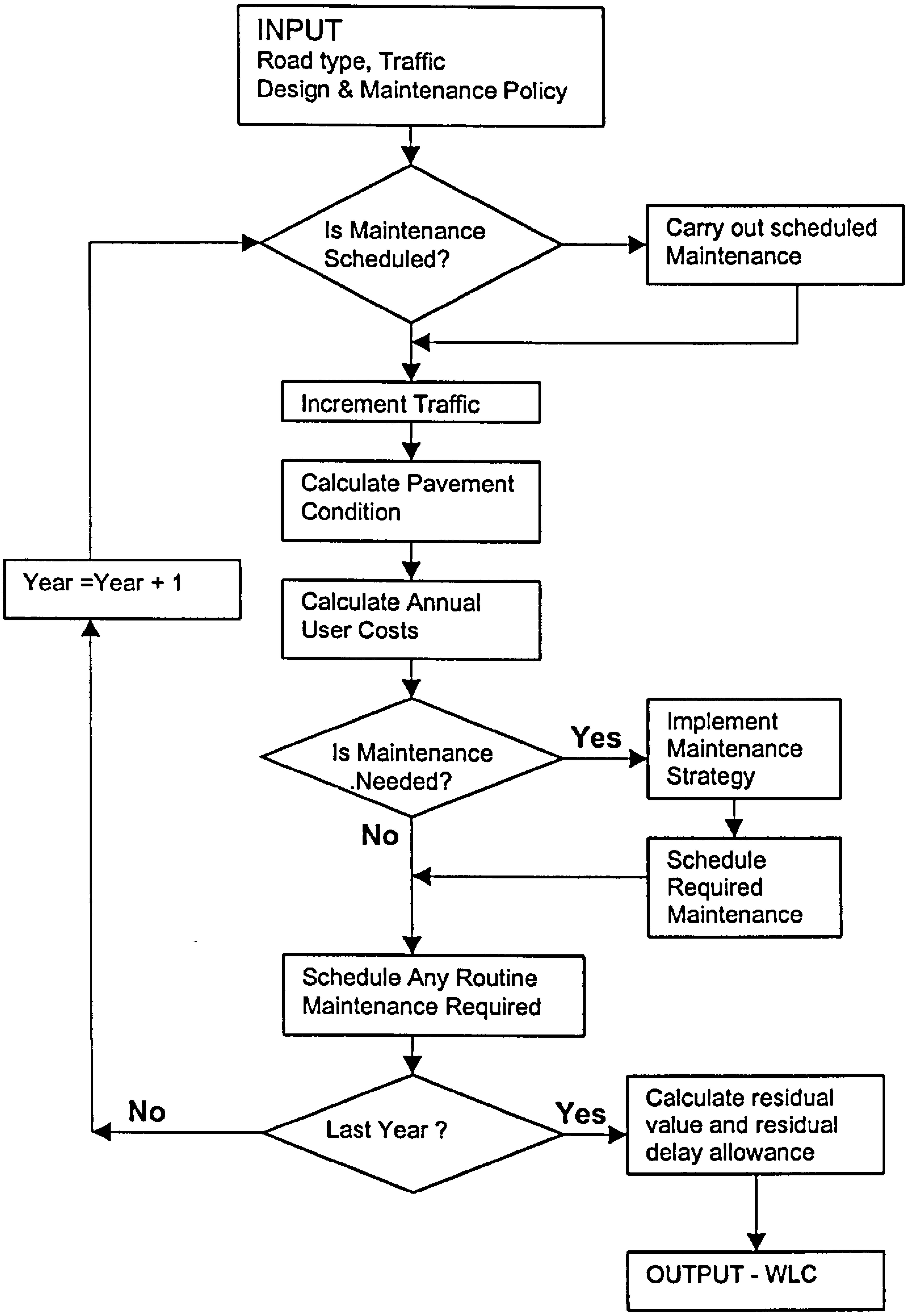
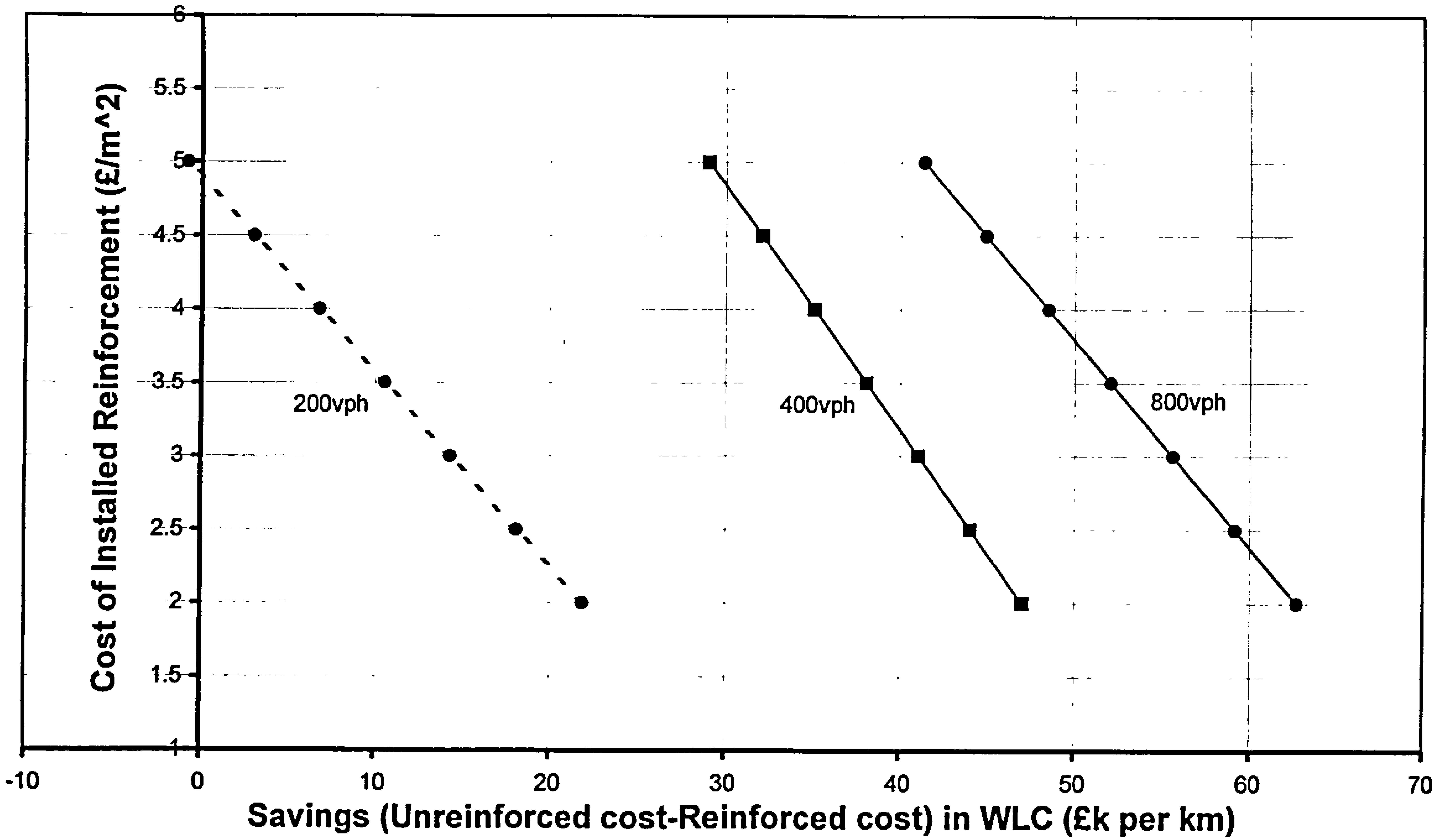
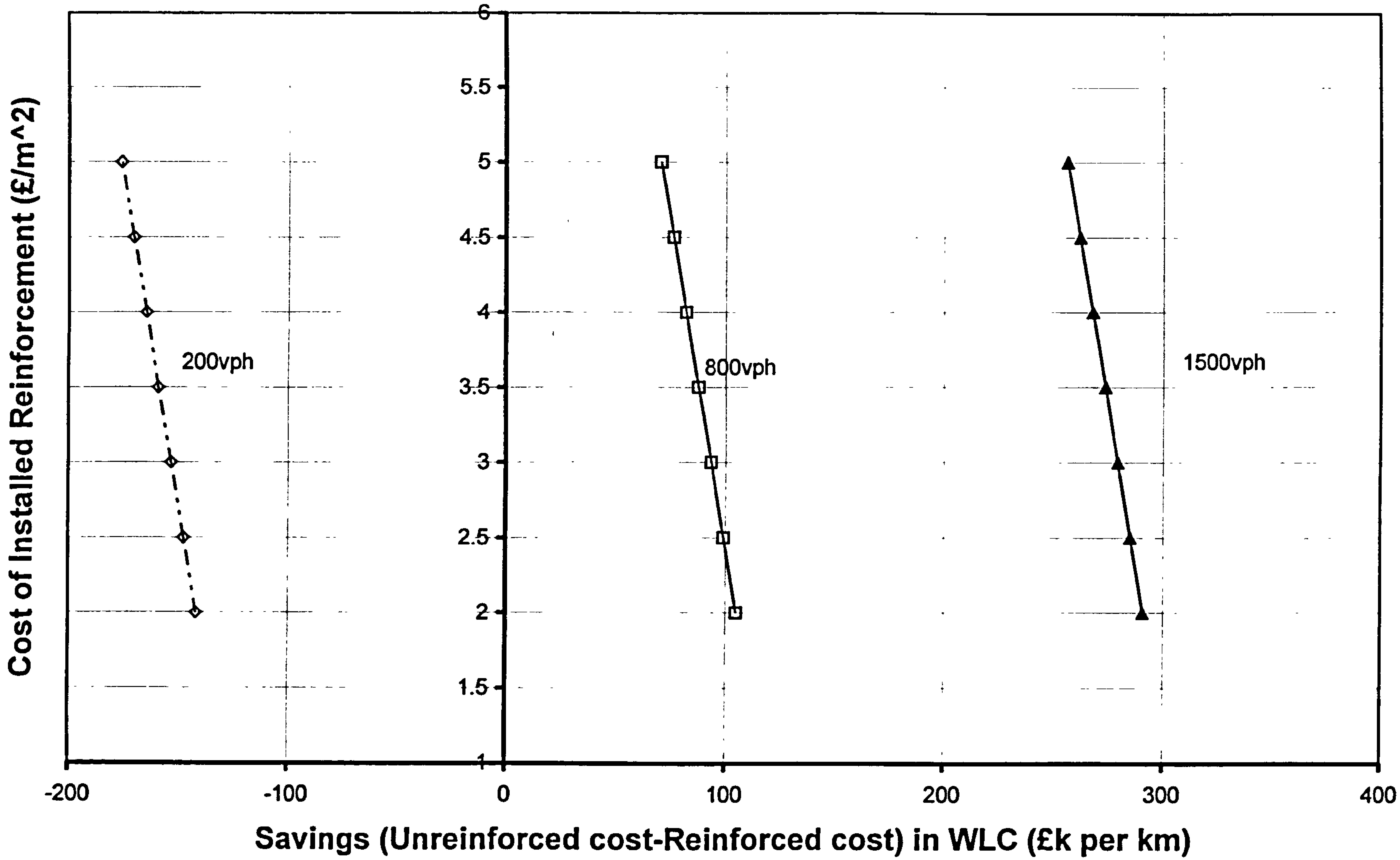


FIGURE 10.1
WLC MODEL



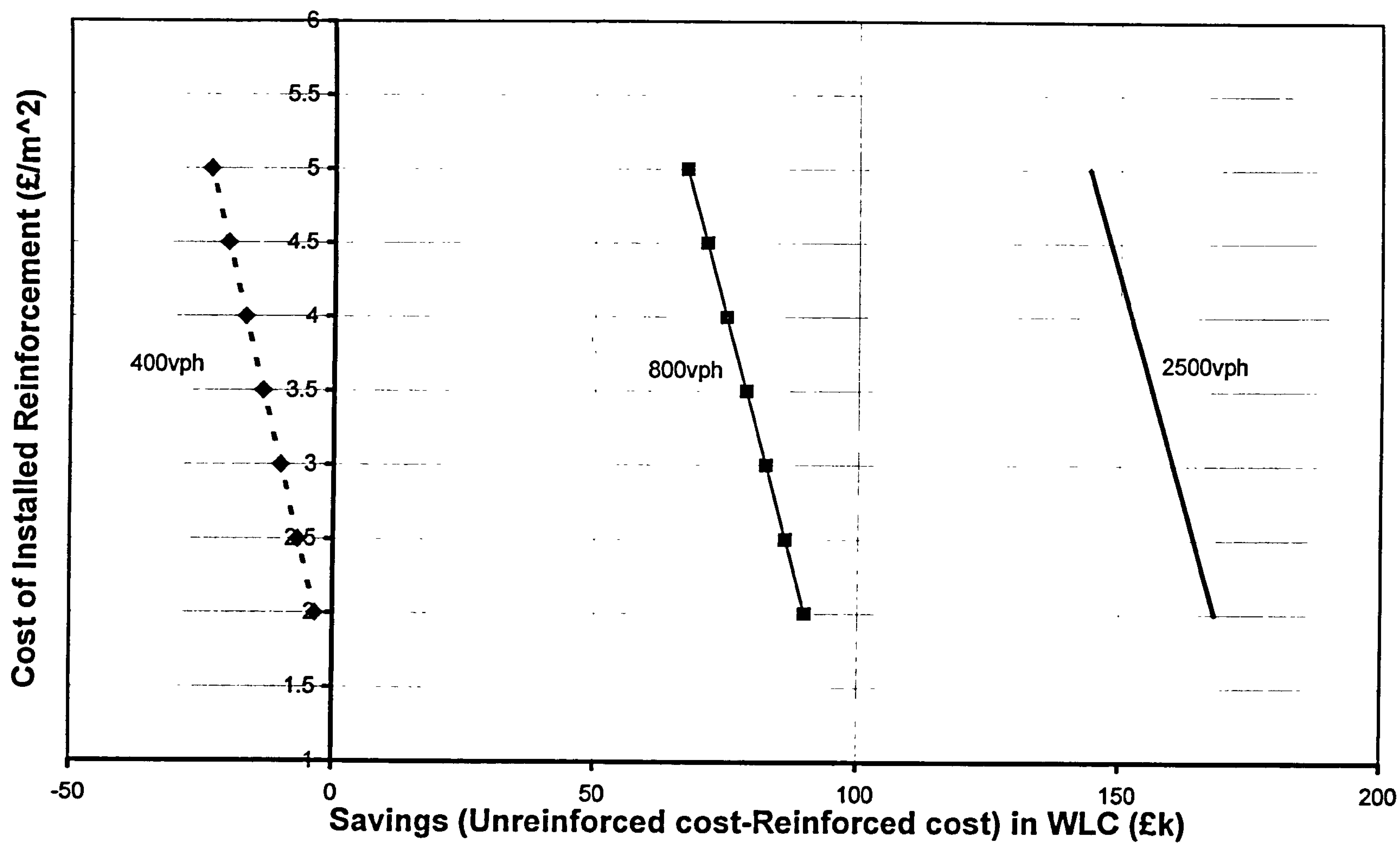
SENSITIVITY OF WLC TO GRID INSTALLATION PRICE AND TRAFFIC FLOW SINGLE CARRIAGEWAY ROAD

FIGURE 10.2
WLC ANALYSIS



SENSITIVITY OF WLC TO GRID INSTALLATION PRICE DUAL2-LANE TRUNK ROAD

FIGURE 10.3
WLC ANALYSIS



SENSITIVITY OF WLC TO GRID INSTALLATION PRICE AND TRAFFIC DUAL 3-LANE MOTORWAY

FIGURE 10.4
WLC ANALYSIS

CHAPTER 11

GUIDELINES FOR DESIGN

CONTENTS

11.1 General

Chapters 2 and 3 highlighted a number of important pointers to be borne in mind when designing a reinforced asphalt solution to reduce (or eliminate) reflection cracking in bituminous overlays. The points made were derived from case histories on different pavement types in various climatic zones. They included general observations on the type of reinforcing product that can be most effectively used on different types of pavement, and recommendations were made on crack width limits, and on the vertical and horizontal deflections that can be applied to geotextiles.

Although the recommendations on deflection and crack width limits can help a designer determine which materials can, or should not be used, large differences in the estimated performance of a reinforced asphalt pavement are common, depending on which design approach is used. Therefore, to help clarify the selection process, results of the laboratory tests and the numerical modelling exercise provide valuable information.

The main findings of the laboratory wheel tracking tests (the tests closest to field conditions – see Chapter 8) indicate that for a thin pavement with relatively high deflections, reinforcing the asphalt with grids or composites can result in an increase in life against cracking by a factor of around 2 to 3. This finding agrees with those of Lytton [11.1], and is compatible with those of Hughes [11.2]. In addition, it was found that when polymer or steel grids, or polymer composites were used, rutting within the asphalt was roughly halved, which also agrees with the findings of Hughes [*ibid*].

Notwithstanding the agreement between the sources mentioned above, it is not recommended that the above relationships should, in general, be directly applied in the field for any reinforcement type without careful consideration, as laboratory findings relate to specific conditions which may be significantly different in the field. In particular, due to the limited trafficking time and the controlled environment in the laboratory, no account of any environmental influence is included in the test results. Historically, it appears that the influence of the environment has in general been underestimated, especially with thick asphalt pavements, where top-down cracking often occurs. Top-down cracking may even be encountered with 'determinate-life' pavements[1], although, it seems that this mode of cracking occurs less frequently than with long-life pavements[1]. As the mechanisms of crack formation are less influenced by traffic, the design approach to prevent reflection cracking for these pavements may differ significantly.

To select an effective reinforced asphalt solution, therefore, careful appraisal of the factors having the largest influence in any given situation is required. Then, to make informed decisions as to which type of reinforced asphalt

[1] A determinate life pavement is distinguished from a Long Life Pavement (LLP) by long-term structural performance. An LLP has a structure sufficient to provide a threshold strength that does not weaken with traffic, provided surface distress that appears in the form of cracks and ruts in the surfacing is treated before it affects structural integrity.

solution to adopt, understanding of the reinforced asphalt mechanisms is also required. These are summarised below.

11.2 Mechanisms of reinforced asphalt

To be effective, asphalt and the reinforcing material must be 'connected' in a manner that allows the properties of both materials to be utilised. Typically, this can occur by (i) interlock, and/or (ii) adhesion. Once the asphalt and the grid are connected, the manner in which the properties of the different materials interact needs to be considered.

If an interlayer material is to reinforce in the true sense of the word, the cross-sectional area and stiffness of the product must be adequate to carry applied loads without excessive strain. Typically, however, it is apparent that for most of the products routinely used with good effect in practice, (polymer grids, for example), this is not the case. For high-strength glass and steel reinforcing products, on the other hand, there may be some justification in assuming this to be true, although the cross-sectional area of these materials is still relatively small compared to the combined stiffness and cross section of a pavement. A second mechanism is where reinforcing products do not reinforce a pavement in the true sense of the word, but provide continuity across any cracks that may develop. Even though the reinforcement may not be adequate to prevent cracks from initiating, its presence across the crack helps to reduce stresses at the crack tip, thus slowing crack propagation, (see Chapter 9). Furthermore, where reinforcement is not stiff enough to prevent the formation of cracks, but still provides continuity across cracks, it must be flexible enough to strain with the asphalt. In this case, the nature of the bond between the asphalt and reinforcement mechanism is particularly important and needs to be understood.

The main features of the two main types of asphalt-reinforcement connection are now discussed.

11.2.1 Grid-Asphalt Interlock

To obtain good interlock and thus mobilise the properties of a grid, the asphalt mixture must have aggregate whose dimensions are compatible with the grid apertures. Only when good interlock is achieved, can the reinforcement be effective and either reinforce the layer, or bridge across any cracks that might form.

To ensure good interlock, a recommended ratio of aperture to aggregate of between 3 and 4 has been proposed [11.3]. This is slightly smaller than the ratio used in the laboratory tests (i.e. 1:4.5 for AR1, and 5.7 for steel grids), so it would appear that 3 to 4 may a minimum ratio, but if the ratio is larger, interlock may be improved further. The largest ratio that remains effective for interlock effects still needs to be defined.

It has been noted (see Chapters 2 and 3) that if installed correctly, polymer grids have been shown to be effective in the field, especially on flexible pavements subject to high deflections. High deflections may occur, for instance where loaded trucks traffic relatively light pavements built on peat subgrades (e.g. in forestry areas in Scotland). Case histories show pavements to remain intact, implying that the asphalt-grid interlock 'connection' remains, and furthermore, that grids must be able to deform considerably without breaking.

The ability of polymer grids to deform up to 10% or more before breaking is seen in the results of tests on the reinforcement materials in Chapter 5. Also, as their stiffness is reasonably compatible with asphalt at typical UK temperatures, they seem suited for use with bituminous mixtures. In addition, the elastic recovery of grids may also be important in helping to promote interlock between the two sides of a crack, so the stress-strain properties of a grid need to be considered if high deflections are possible.

Slip between reinforcement and the asphalt layers can occur if a stiff material (comprising glass for instance) is used, unless the bond is either strong or ductile enough to allow differential movement between the reinforcement and asphalt to occur. With slip between layers, increased tensile strain in the asphalt, (and hence cracking) is likely to occur. Although a grid may be comprised of very stiff material, it can still only make a small difference to the load-carrying ability of a pavement unless sufficient cross-sectional area is present. Also, if grids comprising brittle materials such as thin glass fibres are used, the possibility of failure in shear also needs to be taken into account. This can happen during placement (compaction, in particular), or during service, if laid across cracks or joints that undergo relatively high vertical movements.

In addition to the strength or stiffness of grids, the nature of the connection between the longitudinal and transverse strands also needs to be taken into account. De Bondt [11.4] shows how the connection of a grid can make a significant difference in the load-carrying and load-spreading capability of the reinforcement. Figure 11.1 is taken from Reference 11.4 and illustrates (using numerical modelling) how junction stiffness can influence the load spreading abilities of a grid. Simply put, load is able to pull grid strands through the junctions if they are not tightly connected to the cross-strand, whether it be transverse or longitudinal. This reduces the mobilisation of asphalt resistance due to the 'cross-strand', and can lead to overstressing of the bond between the strand and the asphalt.

Grids comprising woven steel wire seem to offer a useful combination of strength and stiffness, and have been shown to be effective in reducing the incidence of cracking and rutting. These products often include hexagonal-shaped apertures, and may be fixed in place with either a bituminous slurry or 'nails'. If grids are nailed to the existing pavement, the strength of the steel may be mobilised to help reduce movements causing reflection cracking, although the combination of the non-fixed grid junctions, and the aperture shape make it difficult to quantify the way in which these grids interact with asphalt.

If slurry is used to fix grids to the existing pavement, it may bring additional benefits to the pavement in the form of waterproofing. Also, in addition to fixing the grid and adding waterproofing properties to the pavement, the slurry may also help blunt cracks by dispersing energy that would otherwise propagate cracks. The possibility of using modified bitumen in the slurry, means that added protection against brittleness at low temperatures and plastic flow at elevated temperatures is possible, although the combination of modified bitumen and steel grids may not be easy to justify economically.

In addition to the above points that are directed toward crack suppression, the grid-asphalt interlock can have an important part to play in reducing permanent deformation in the bituminous surfacing. It was noted in the analysis of wheel-tracking tests (Chapter 8), that test sections reinforced with grids and composite materials with a distinct profile developed significantly less rutting than the other sections. The empirical findings agree with the findings of Hughes [11.2], and appears to be an important property of these materials. Reinforcement of this nature may have good use in Long-Life Pavements where resurfacing treatments may be postponed, thus reducing maintenance costs.

The role of grid-asphalt interaction is also important to consider where thermal cracking may be expected on overlaid rigid or flexible composite pavements. If the pavement layer on which the overlay is to be placed is expected to cause cracking due to its movement relative to the asphalt, then, contrary to the 'normal' situation (where a good bond between layers is required), the reinforcement should be partly decoupled from the layer below, but firmly bonded to the asphalt overlay. To achieve this, the reinforcing material should be placed directly on the concrete without being firmly bonded. It has been shown [11.5] that the bituminous bond between a composite and concrete appears to slip before the bond between the composite and the asphalt. This allows some redistribution of strain and so prolongs life to cracking. Steel and polymer grids, on the other hand appear to actually increase the level of asphalt failure strain by some means, which may also be due to some manner of strain redistribution through interlock of asphalt and the reinforcing material.

11.2.2 Fabric (or composite) adhesion.

In the absence of interlock, the nature of the connection between the asphalt and the fabric (or composite) relies on bitumen adhesion. This bond is influenced (*inter alia*) by the nature of the stress-strain properties of the 'adhesive', the adhesion between both the fabric and the bitumen, and between bitumen and the existing pavement. In addition, sufficient bitumen must be present on the surface of the fabric for adhesion to the existing pavement. To ensure that this is the case, both the absorption properties of the geotextile and the existing pavement must be considered.

It has been noted that geotextiles and fabrics do not normally perform well in cold climates where thermal movements are high, and bitumen is hard and brittle. This is probably due to the stiff bitumen not providing a soft, crack-blunting layer (similar to a Stress Absorbing Membrane Interlayer-SAMI) but this, of course, might be improved by using appropriately modified bitumen.

If insufficient bitumen is applied, less bitumen than is required will be present on the surface of the fabric, possibly leading to debonding and 'high' strains in the upper layer when loaded, and hence cracking. Poor adhesion may also promote slip between reinforcement and asphalt, which can also contribute to cracking. On the other hand, if too much bitumen is applied, bleeding through the surface course can occur, or possibly interlayer slippage, especially in hot conditions where bitumen viscosity reduces. A guide to the amount of bitumen required is normally given by the manufacturers of geotextiles, but in the absence of this information, the relationship supplied by Smith [11.6] may be used:

$$RTC = 0.05(TW)^{0.3}$$

where RTC is the Recommended Tack Coat (in gallons /square yard)
T is the geotextile thickness (inch/1000), and
W is the geotextile weight (oz/square yard).

Unless the adhesive bond can accommodate high strains, under load, the interlayer bond may be broken, continuity across cracks lost, and subsequent cracking failure ensue.

A balance needs to be maintained between the stiffness and brittleness of the bond, i.e. to avoid excessive loss of bond in 'hot' conditions and brittleness in 'cold' conditions. Both can contribute to overall pavement failure: by cracking if brittle, or allowing excessive slip (and thus tensile strain in the top asphalt layer) between layers, if too fluid. An elastic material that can accommodate (recover) large strains may also be an advantage, especially if the grid component (of a composite material) is relatively stiff compared to the asphalt, and high pavement deflections are expected.

From literature, it seems that that no simple and reliable design method providing economic reinforced asphalt solutions exists. However, knowledge of reinforcement-asphalt mechanisms (summarised in the preceding sections) and proper assessment of the existing pavement and expected loading conditions can give an acceptable solution. Using evidence from the laboratory and the literature review, a guide to the selection of reinforcing products is now given.

11.3 Design Approach

It is appreciated that without more laboratory testing, numerical modelling and field trials for confirmation, an authoritative design guide is beyond the scope of this document. However, a great deal of information is currently available from literature that includes case histories, laboratory investigations, and numerical modelling. The laboratory testing and numerical modelling carried out in the current work has also served to clarify certain issues and augment the current state of knowledge. Accordingly, the guidelines given later in this chapter are not confined to the findings of this particular project, but are a compilation of what is considered to be the best consensus of information.

To assist in the selection of reinforcing products to help prevent reflection cracking in asphalt overlays, various factors need to be taken into consideration. These include the cause of the pavement distress, and possible alternative solutions together with an assessment of their relative economic worth. A design approach is therefore proposed that incorporates general pavement engineering approaches in addition to using the findings from the laboratory and Finite Element modelling. The approach can be divided into three main aspects:

- Site investigation
- Selection of alternative solutions, and
- Economic appraisal

11.3.1 Site Investigation

The object of the site investigation is to define the cause (or causes) of existing pavement distress, which may require a number of activities to be carried out. As a minimum these include visual condition inspection and confirmation of the pavement structure and crack direction and depth (if top-down) through coring.

By classifying pavement type and climate, a first indication as to whether distress in the existing pavement is caused primarily by environmental factors, or traffic loading is given. Table 11. 1 summarises the main pavement types and their 'typical' cause(s) of cracking. It has been assumed that rutting in pavements is caused exclusively by wheel loading, and it is therefore not referred to in the table.

Table 11.1 Typical Causes of Failure

Pavement Type		Main Cause of Cracking[1]
Fully flexible	'Thin'[3]	Wheel Loads
	'Thick'[3]	Environment[2]
Flexible Composite	'Thin'[3]	Wheel Loads
		Shrinkage/curing
	'Thick'[3]	Environment[2]
Overlaid Rigid		Load-induced joint movement
		Environment[2]
Rigid composite		Environment[2]

Note 1 Even where traffic loading is not taken to the main cause of cracking, it is still expected to contribute to cracking.

Note 2 The category 'Environmental' includes both temperature and ageing effects.

Note 3 'Thin' or 'Thick' refers primarily to the thickness of the asphalt, but also to the behaviour of the asphalt layer under load. However, there is no definite thickness distinguishing 'thick' from 'thin', as the condition of the pavement in general, needs to be taken into account. Deflection measurements may help in this regard, and any pavements with higher deflections may be regarded as 'thin'.

The 'Ageing' referred to in Note 2 refers specifically to embrittlement of the binder which largely occurs through exposure to the atmosphere (oxygen and ultra-violet light in particular) and repeated temperature changes. This being the case, it follows that ageing is often associated with cracking initiating at the surface of 'thick' or 'thin' pavements, where relative deflections across cracks are usually low. The combination of ageing and large daily or annual temperature changes has been found to give wide cracks.

Details of the site investigation to be carried out to help in the decision of which reinforcement should be applied will vary from site to site. This is due to factors such as the pavement construction type (and any changes in construction across the site) and the condition of the pavement. However, in general the following should be carried out:

Visual assessment

This should include records of **crack pattern,** i.e. whether cracks are predominantly longitudinal, transverse or randomly orientated. Their spacing and width should also be recorded.

The incidence and severity of **rutting** should be recorded. The characteristic shape of the ruts may also give an indication of whether deformation is spread throughout the whole structure or predominantly in the bituminous layers.

From visual records it may be possible to deduce the likely pavement structure under the asphalt overlay. For instance, if regular transverse cracks are noted, then it is likely that the pavement under the overlay is a jointed rigid pavement.

The incidence of longitudinal cracks might be due to lateral movements of an embankment, or due to soft verges, or, if located over a construction joint, load-induced differential movement. The width of cracks is important to record, as beyond a typical value of around 10mm, (see Section 11.4) it is recommended that cracks be filled before using a reinforced overlay. As always with a visual survey, the condition of drains should be noted as this can help explain the overall state of a pavement, especially if the pavement is subject to high deflections. High deflections in a pavement can be an important factor in selecting suitable reinforcement (see Section 11.4).

The combination of high temperatures and heavy loads often leads to rutting. If a pavement is already rutted, it suggests that steps may need to be taken to avoid this reoccurring. Guidance on the type of reinforcement that should be considered to reduce the incidence of rutting is given in Section 11.4.

Coring

There are four main reasons why cores should be taken during a pavement investigation:

- to determine the pavement structure (and details of layer thicknesses)
- to help determine material condition (through visual inspection and by providing samples for testing)
- to see whether cracks initiate at the top or the bottom of the bound materials, and
- to provide access to unbound materials, which may then be sampled and/or tested *in situ* (with a Dynamic Cone Penetrometer, for instance).

Deflection measurement

Vertical deflections need to be measured to define the overall strength of the pavement (and thus its suitability for the anticipated traffic flow), and may help in deciding which types of reinforcement should be considered. Deflections can be used with empirically-based recommendations, (obtained from field observations) to help determine which reinforcement types may be considered. Deflections measured across cracks can be also be used to estimate load-transfer properties of the remaining interlock across the crack which may also be required with some design methods.

Measurement of the horizontal movement of joints and cracks can also be used to select an appropriate reinforced asphalt solution, if indeed it can be successfully applied. Unfortunately, these measurements are not easily obtained and it is normally necessary to estimate movement using temperature data together with relationships between temperature and material expansion, and measurements of the distance between cracks and/or joints (see Section 11.4).

11.3.2 General considerations for selection of reinforcing products.

It should be appreciated that in general, and on trunk roads in the UK in particular, since mid 1999 overlays designed using deflectograph data are thinner than those designed prior to this date. This is due to a reinterpretation of data used to formulate the design relationships. Taking this into consideration together with the policy of evaluating maintenance using Whole Life Cost procedures, the design philosophy proposed here is to use reinforcement to reduce the frequency of maintenance interventions, rather than to construct thinner overlays. Exceptions to this strategy would be where a reduced asphalt thickness has to be used, possibly for headroom reasons (under a bridge) or to reduce load (over a structure), for example. Use of reinforced asphalt in these cases is discussed in Section 11.4. where the potential savings in asphalt thickness for a selection of designs using OLCRACK [11.5] are given.

The need for strengthening overlays has generally reduced in the past few years as better understanding of the behaviour of 'thick' pavements grows. As a general rule, these thick 'long-life' pavements have relatively small deflections, do not require strengthening and cracks normally propagate from the surface. Maintenance of these pavements is often limited to inlays or resurfacing and may be triggered by rutting and/or surface cracking. This suggests that on these pavements, reinforcement to control the likely types of pavement distress should be placed near the surface. Literature appears to suggest that geotextiles or composite reinforcement rather than grids may be well-suited to these cases, as cracking from the surface may be controlled by the SAMI-type behaviour (i.e. 'crack blunting' properties). Also, because grids are more prone to placement problems than geotextiles, there is less risk involved with construction. This is a real concern as where reinforcement is applied close to the surface, any errors during construction are prone to cause failure. Also, the waterproofing properties of a geotextile are useful in cases where cracks penetrate from the surface.

Notwithstanding the general observation that most commercial reinforcing products do not reinforce pavements in the true sense of the word, they may be sufficiently strong to prevent material from breaking up. This may be the case on single track rural roads where edge breaks are common, for instance. Recommendations for these cases are made from reported observations, and are given in Section 11.4.

11.4 Guidelines from Literature

Although all design methods reviewed in Chapter 3 appear to have merit in certain situations, a universally accepted approach has not been found. Most of the design methods encountered are intended to prevent reflection cracking occurring on pavements carrying 'significant' traffic volumes (greater than 10msa, for instance). On the other hand, other situations exist where light pavements carrying low numbers of vehicles with high loads need to be reinforced. In these cases design methods used for the more conventional reflection cracking situations may not be suitable, and the best manner of dealing with these situations is to use empirically-derived relationships together with general understanding of asphalt-reinforcement mechanisms and pavement engineering principles. The following paragraphs give guidelines on which products have been found to be suitable in particular situations.

Pavements on low strength foundations - Traffic-induced distress
In cases where 'excessive' deflections under wheel loads are expected, i.e. where calculated strains in the asphalt suggest that the pavement is only likely to withstand a thousand or less loadings, a grid that can undergo large strains without rupture is required. As it is unlikely that reinforcement is sufficiently stiff to reduce deflections, the function of reinforcement is to help disperse cracks through the asphalt, and to provide continuity across cracks that develop, thus helping to maintain interlock across the cracks. Grids that have been shown (in the field) to provide these properties are typically polymers, such as polypropylene or polyester. Alternatively, woven steel grids have been found to work well in these situations, probably due to the toughness of steel, the (typically) good interlock with asphalt, and having joints that allow movement to occur between wire strands.

Rural single track roads
Where rural single track roads are subjected to relatively few, but often heavy, wheel loads, edge breaks often occur. A solution is therefore required that can strengthen the asphalt and prevent (or slow the rate of) lateral movement of material into the soft verge. Steel grid reinforcement has been found to work well in these situations. Similarly to pavements on weak foundations, the flexibility of the twisted wire joints, toughness of the steel, and good interlock with asphalt are believed to be the key factors that combine to help reduce the problem.

Similarly, rural roads built on embankments over soft foundations are also prone to edge breaks unless a substantial strip is provided alongside both sides of the pavement. Steel grids have also been found to be effective in these cases, probably for the same reasons given above. Both steel and polypropylene grids have been successfully applied in 'moss roads' in Lancashire and roads on peat in Scotland. [11.7,11.8]. However, although reinforcement helps to keep these roads intact and useable, cracks still occur. Due to the nature of some of the sites in Scotland and Lancashire, i.e. reasonably 'remote' with few users, a cracked pavement may be tolerated more easily than would be the case in a heavily trafficked urban environment. This is probably true both for pavement materials and user perception. Therefore, in situations where a visually acceptable surface must be maintained, additional measures such as a modified surfacing that can tolerate movement may be needed. Various proprietary products are currently available that may fulfil this need.

Environmental loading - Shrinking/swelling foundations
Where pavements are constructed on subgrades that contain swelling clays, seasonal moisture changes can cause large movements in a pavement. This leads to cracking, further ingress of moisture, leading in turn to more cracking, and failure. In cases such as these, reinforcement can be used to help

spread wheel loads, and, probably more significantly, to reduce moisture changes in the foundation (if a geotextile or composite is used). This approach has been successfully used in Australia for twenty years or more. Also, it is noted that successful use of the geotextile-reinforced bituminous surfacings was not limited to roads subjected to low traffic volumes, and they have been successfully applied to highways carrying in excess of 110,000 vehicles per day (including 12% commercial vehicles).

As with all reinforcement types, however, certain conditions are required to promote the successful use of these materials. In particular a maximum crack width of 5mm is recommended to avoid loss of binder during construction (and to avoid local shearing and support problems). Some patching and surface regulation may also be required before the reinforced surfacing is applied.

Typically, where a pavement is built on an embankment, the waterproofing fabric or composite may need to be extended down over the embankment slope to help reduce moisture changes. The extent that this may be required will be determined by local site conditions.

Thermal Cracking

With overlaid rigid pavements or composite pavements, thermal movements are often the prime cause of reflective cracking. For example, in the south of England joints have been found to move up to 5mm annually, due to annual temperature variations of around 30°C [11.9]. This magnitude of movement clearly gives cause for concern when the guidelines below are applied, and as these movement are difficult to either prevent, or to accommodate with any surfacing, action must be taken to reduce these movements.

Muktar and Dempsey [11.10] recommend that for effective use of geotextiles, horizontal movements should be less than 1.78mm. As these horizontal movements are related to the length of concrete bay, smaller movements may be achieved by reducing bay lengths, or by decreasing temperature changes in the concrete by increasing the overlay thickness. A thicker layer also increases the time required for a crack to propagate through the asphalt.

Mukhtar and Dempsey [11.10] note that geotextiles tend to perform better on flexible pavements that exhibit distress via closely-spaced alligator (fatigue) cracking, rather than on pavements with large cracks and/or large deflections.

Recommendations on limiting values for use of geotextiles are given in Table 11.2.

Table 11.2 Recommended Limits for Geotextile use [11.10].

	Vertical Deflection (mm)	Horizontal Displacement (mm)	Maximum Crack or joint width[1] (mm)
Geotextile not required	<0.05	<0.8mm	--
Suitable Range	0.05-0.02	0.8-1.8	3.0 – 10
Geotextile not suitable	>0.02	>1.8	>10

Note 1. If cracks are greater than 10mm, it is recommended that they be filled with a 'rigid' filler.

A general rule for the equivalent thickness of geotextile reinforcement found in the USA is given in Reference 11.10. It was found that a geotextile could be taken as equivalent to approximately 30mm of asphalt for overlays of up to around 65mm thick. Above this thickness, the geotextile was found to be less effective. However, it is noted that the geotextile was placed under the overlay, and if cracking was not initiated at the bottom of the overlay, then the geotextile would not have been effective. An example of the application of this 'rule of thumb' to the design of fully flexible pavements (as given in Volume 7 of the Design Manual for Roads and Bridges [11.11]), is shown in Table 11.3, using two equivalent thicknesses of asphalt – 25mm (1 inch) and 37mm (1.5inch).

Table 11.3 Applying the 'Rule of Thumb' to Fully Flexible Pavement Design [11.11]

Material	Equivalent Asphalt Thickness (mm)	Design Traffic (Unreinforced) (msa)	Total Bituminous Thickness (mm)	Design Traffic (Reinforced) (msa)	Ratio of design lives
DBM/HRA	25[1]	1.3	200	2.3	1.77
		2.3	225	4.5	1.96
		4.5	250	8.0	1.78
DBM50	25[1]	2.0	200	4.0	2.00
		4.0	225	8.0	2.00
		8.0	250	15.0	1.88
DBM/HRA	37[2]	1.3	200	2.8	2.15
		2.8	230	5.6	2.00
		5.6	260	11.0	1.96
DBM50	37[2]	2.0	200	4.6	2.30
		4.6	230	10.3	2.24
		10.3	260	21.0	2.04

Note 1 Approximately 1 inch

Note 2 Approximately 1.5 inches

As seen from the ratio of design lives, the increased thickness of 25 or 37mm of asphalt approximately doubles the design life. This is in general agreement with the findings of the beam test and Pavement Test Facility test results.

Alternative approaches should be used to estimate reinforced asphalt overlay thicknesses where reflection cracking occurs in situations 'more typical' than the cases described above. These situations include pavements with 'strong' foundations like, for example most pavements on the motorway and trunk road network. One possible approach is to use the design method proposed by Brown et a[11.5], (examples of which are given in Table 11.4) showing designs for reinforced and unreinforced pavements for two different traffic levels.

Three different situations have been selected for illustration:

Case a   Reinforcement at 30mm depth.
Design traffic = 50msa
200mm CBM (20 GPa stiffness)
3m crack spacing
300mm sub-base (150 MPa stiffness)
Subgrade (50 MPa stiffness).

Case b   Reinforcement at base of overlay
Design traffic = 50msa
200mm 'old ' asphalt (6 GPa stiffness)
0.4m crack spacing
300mm sub-base (150GPa stiffness)
Subgrade (80 MPa stiffness).

Case c   Reinforcement at base of overlay.
Design traffic = 1msa
200mm 'old' asphalt (6 GPa stiffness)
0.4m crack spacing
300mm sub-base (150MPa stiffness)
Subgrade (80 MPa stiffness).

Table 11.4 Asphalt thickness design (mm):- fully flexible cracked Pavement.

Reinforcement	Saving in Asphalt Thickness (mm)		
	Case a	Case b	Case c
Unreinforced	180 [-]	240 [-]	145 [-]
Polymer Grid (Tensar AR1)	153 [27]	208 [32]	110 [35]
Polymer Composite (Tensar AR-G)	156 [24]	208 [32]	110 [35]
Glass Composite (Rotaflex 833)	167 [13]	227 [13]	127 [18]
Steel Woven Grid (Road Mesh)	145 [35]	199 [41]	104 [41]

A shear stiffness across the crack of 1000 MN/m^3 has been assumed for each case.

It is noted that the estimated savings in asphalt thickness are similar to those estimated from analysis of field experiments in the USA and used in Table 11.3.

Rut Reduction
For permanent deformation within the asphalt layers (as opposed to structural rutting), grid reinforcement can help reduce rutting to around half or a third of the value expected in unreinforced layers. To work efficiently, grids need to be placed at a depth where shear strains are expected to be highest. This depth is determined by the load configuration and size of wheel area, and may be estimated using linear elastic theory. It has been estimated that this may be at a depth of around 20% to 25% of the width of the loaded area [11.2].

11.5 Guidelines from Laboratory testing and modelling

From the laboratory testing described earlier, and the modelling described in Chapter 9, the following guidelines are derived.

Interlayer bond
The Finite Element modelling in Chapter 9 indicates that the interlayer bond is an important factor for both traffic and thermal loading in determining resistance to reflection cracking.

For traffic loading it has been shown that a high shear stiffness between the layers is required to mobilise reinforcement properties and so reduce stresses at the crack tip. From the shearbox tests, a range of between 10 and 20 N/mm/mm^2 seems typical for the reinforcement and asphalt combinations chosen, although the values for both the Roadmesh and AR1 samples were significantly higher at shear stresses of around 250kPa. This value is calculated (using multi-layer linear elastic theory) as being reasonable for the maximum shear stress at a reinforced interface of a 'thin' pavement (i.e. bound layers of 160mm), under a standard axle load.

For traffic loading especially, a high shear resistance between layers should be provided, and to achieve this, careful consideration of the relationship between the grid aperture size and the aggregate size is required. A ratio of 3 to 4 has been recommended in literature, which is compatible with findings of laboratory tests.

To achieve a satisfactory bond, it is recommended that different combinations of aggregate and grid aperture be tried. To establish the relative value of bond from different combinations of materials, a shear test is required. Ideally, a test similar to that described in Chapter 6 should be used, but a more simple configuration such as the test used to measure shear bond on cores from the PTF may be adequate. In addition, with these trials, measures of asphalt density should be taken and note of any voiding around reinforcement made. Although voiding was not seen to be significant with the 14mm DBM mixture used, this might not be the case with a larger stone-size mixture and/or different grading.

In the absence of significant interlock, i.e. where fabrics and composites are used, adhesion plays a vital role in connecting asphalt to the reinforcement. This bond depends on a number of factors including the area of the reinforcement, and the nature of the asphalt to which it must adhere. It is critical to ensure that an appropriate quantity and type of bitumen is used, otherwise layers may debond. In this regard recommendations given by the manufacturer should be observed, and ideally, supplemented with tests.

Low temperature crack tests to determine the crack-suppression properties of reinforced asphalt under low temperatures were carried out as part of the overall project [11.5]. The results of these tests showed that stiff materials bonded by bitumen to both the concrete and asphalt overlay helped distribute strains along the interface, thus maintaining the integrity of the asphalt. Significantly wider crack openings were tolerated by the reinforced section than by the unreinforced section which helped prolong pavement life. Twisted steel and polypropylene grids were also found to prolong pavement life, partly through a different mechanism, i.e. by increasing the strain taken by the asphalt before failure occurred. How this occurs is not clear, but it is probably also due to distribution of strain along the asphalt-grid interface. Results showed that, the polypropylene grid also increased resistance of the asphalt to crack opening.

If the cracking mechanism in the field is similar to that in the laboratory, i.e. cracking being caused by movement of the 'existing pavement', then the bond between the reinforcement and the overlay should be stronger than the bond between the reinforcement and the existing pavement. If this is not the case, then (unless the reinforced layer is strong enough to prevent movement taking place), cracking will move into the overlay with little or no resistance from the reinforcement. To achieve this in practice on a concrete pavement, the bond between the reinforcement and the concrete reinforcement should be less than between the reinforcement and the asphalt. This may mean that the reinforcement should be laid directly on the concrete without being nailed or placed on a pad coat. More development of recommendations for site installation is required to be confident that appropriate bond is achieved. This needs to be sufficient for general stability between pavement layers, but low enough to allow slip at low temperatures to occur between the reinforcement and the existing pavement. A modified bituminous material (whose properties are less temperature-susceptible than conventional bitumen) may be appropriate.

Grid Selection

Although in a conventional reinforcement mode (e.g. in concrete), the strength and stiffness reinforcement is important, it is less so in most applications of reinforced asphalt. More important is the compatibility of the grid and the asphalt mixture to ensure that material can be compacted properly (and so avoid voiding), create a good interface bond (either interlock or adhesion), and to some degree, be able to accommodate movement of the asphalt layers.

Practical issues are important in the selection of reinforcement products, and in addition to in-service conditions, factors relating to transportation, storage and placement of the reinforcement need to be considered. In particular materials prone to damage (e.g. glass–reinforced materials) need to be protected. Also, where high differential movements are likely in service, such as across joints between concrete slabs, for instance, brittle materials or materials susceptible to shear fracture should be avoided. Another important practical constraint to be borne in mind during the design stage is the experience of the contractor. Often, 'problems' with reinforced asphalt (as encountered in the desk study) relate to difficulties with installation. This is especially true with grids (as compared to geotextiles). For this reason, it is recommended that that a specialist sub contractor is used if grids are selected.

If **permanent deformation** of the surfacing is expected to be a problem, then a reinforcing material with a distinct vertical profile may be considered. Grids with large nodes (where strands cross or join) such as woven steel nets or hole-punched polymer grids have been shown to be effective. Hughes [11.2] suggests that the most effective depth to place grids to prevent rutting is approximately 0.25 x width of loaded area.

It is noted that the guidelines for using grids to slow rates of rutting are, at present, based on a limited observations, unlike guidelines for crack suppression which have been developed from more extensive research over the past two decades. For confirmation therefore, more rigorous testing and analysis is required to confirm and quantify the proposed mechanism.

Fabrics

These products have been extensively used on a variety of pavements and seem to help prevent the incidence of reflection cracking.

In general unwoven fabrics have low stiffness with their main function being to provide a medium in which bitumen can be stored. The properties of the bitumen can help to blunt cracks and thus slow propagation rates, and also waterproof pavements if cracks do form. It is important to ensure that sufficient bitumen is applied so that adequate bitumen remains for adhesion with asphalt. The amount of bitumen to be applied varies with the absorption properties of the material and the surface of the existing pavement. To apply an appropriate amount of bitumen, manufacturers' recommendations should be used, or in their absence, the relationship given in Section 11.2.2 can be used.

Woven fabrics may incorporate strands of reinforcement (such as the glass-reinforced PGMG products, for instance), and be stiffer and stronger than unwoven products. Correct tack coat application is especially important for these materials as without good bond (in the absence of any interlock) the properties of the reinforcement will not be mobilised.

Detailed recommendations for grid selection can be derived using OLCRACK [11.5], although more in-situ calibration is required to increase the reliability of predictions.

11.6 Economic Appraisal

Unless it can shown that reinforced asphalt is an economic solution, a technical solution using reinforced asphalt has little relevance. Economics can and always should play an important role in the selection procedure, and normally require a more extensive analysis than a direct comparison of the installation costs of reinforced and unreinforced pavement layers. At present, and over the past few years, the economic assessment of any major maintenance treatment for trunk roads in England is, and has been carried out using whole life costing, which has been described in Chapter 10. With this technique, both works costs and user costs are taken into consideration over the expected life of a pavement. In Chapter 10, examples were given where whole life costs were calculated for three road types, each with three traffic flows for a thirty-year evaluation period. Results showed that economic viability is largely determined by user costs, incurred as a result of the maintenance regime followed and by traffic flow. Plots of grid cost versus 'savings' (the difference in costs between reinforced and unreinforced pavements) for different traffic flows served to illustrate this point. In the examples given, for reinforced asphalt to be economic traffic flows of around 200vph for single carriageways, 800vph for dual APTRs and greater than 400vph for 3-lane motorways. As mentioned in Chapter 10, whole life costs depend on many factors, not least being the type of traffic management employed during the works.

From the above, selection of reinforcement should therefore take into account both the initial costs as well as subsequent maintenance expected during the life of the pavement. In this respect, issues such as the suitability of reinforcement for recycling may also need to be examined in the light of the general move towards more sustainable construction practices, and the need to reuse materials where possible.

A simplified whole life cost procedure is at present being used by UK maintenance agents for bidding for capital maintenance funds. This format could be adapted for appraisal of alternative reinforced asphalt solutions by using assumptions for reinforced asphalt.

11.6 Summary

Guidelines for the selection of reinforcement products have been proposed following appraisal of laboratory test results, case histories (taken from literature), and economic assessment. The principles of the mechanisms of reinforced asphalt are given to assist in understanding the requirements of a reinforced asphalt solution.

When assessing possible reinforcements types for use, both economic and technical aspects need to be considered. To determine the economic viability of a proposed solution, an assessment of maintenance over the whole life of a pavement should be carried out. Heed of the disruption to road users (a consequence of maintenance works), must be taken into account, as without consideration of these 'user costs', grids may not be found to be economic.

This is crucial, as it is understood that regardless of technical suitability, a reinforced asphalt solution will not be accepted if shown to be uneconomic.

Insofar as technical issues are concerned, two of the main factors to be considered are pavement type, and identification of the cause of distress, i.e. whether traffic or the environmental is dominant. In this regard, site investigation is required, particularly visual inspections, where records of crack patterns are useful indicators of pavement behaviour. Another important site investigation task is to determine whether cracks propagate from the top or the bottom of layers, which can be achieved by coring through cracks. Knowledge of the crack mode will help a designer to place reinforcement in the most appropriate position.

Recommendations regarding reinforcement in 'thin' pavements constructed on soft foundations are given. In these cases where deflections are high, reinforcement should be able to strain with the pavement to some degree to avoid the development of excessively high stresses between the reinforcement and asphalt layers, and thus the likelihood of debonding. Where thin pavements on soft foundations are subjected to heavy loads, reinforcement may be an obvious cost-effective alternative to reconstruction, which might otherwise be required after relatively few repetitions of heavy loads. Selection of reinforcement for situations that are less extreme than the thin pavement case is not as straightforward and requires more design consideration. This may be carried out using design programmes such as OLCRACK[11.5] which takes into account reinforcement geometry, strength and stiffness, the reinforcement-asphalt bond and pavement support. As a spreadsheet-based programme, OLCRACK is easily used and, subject to more *in situ* calibration, a useful design tool. Other design approaches may include Finite Element Analysis (FEA), which although powerful, is often difficult to use. Both FEA, and OLCRACK have parameters that are awkward to determine, and will require special test methods to be developed.

Practical issues may determine the success or failure of reinforced asphalt solutions. Some reinforcement is more straightforward to install than others, and unless a specialist sub-contractor is to be used, it may be preferable to specify 'easy-to-install' materials.

11.5 References

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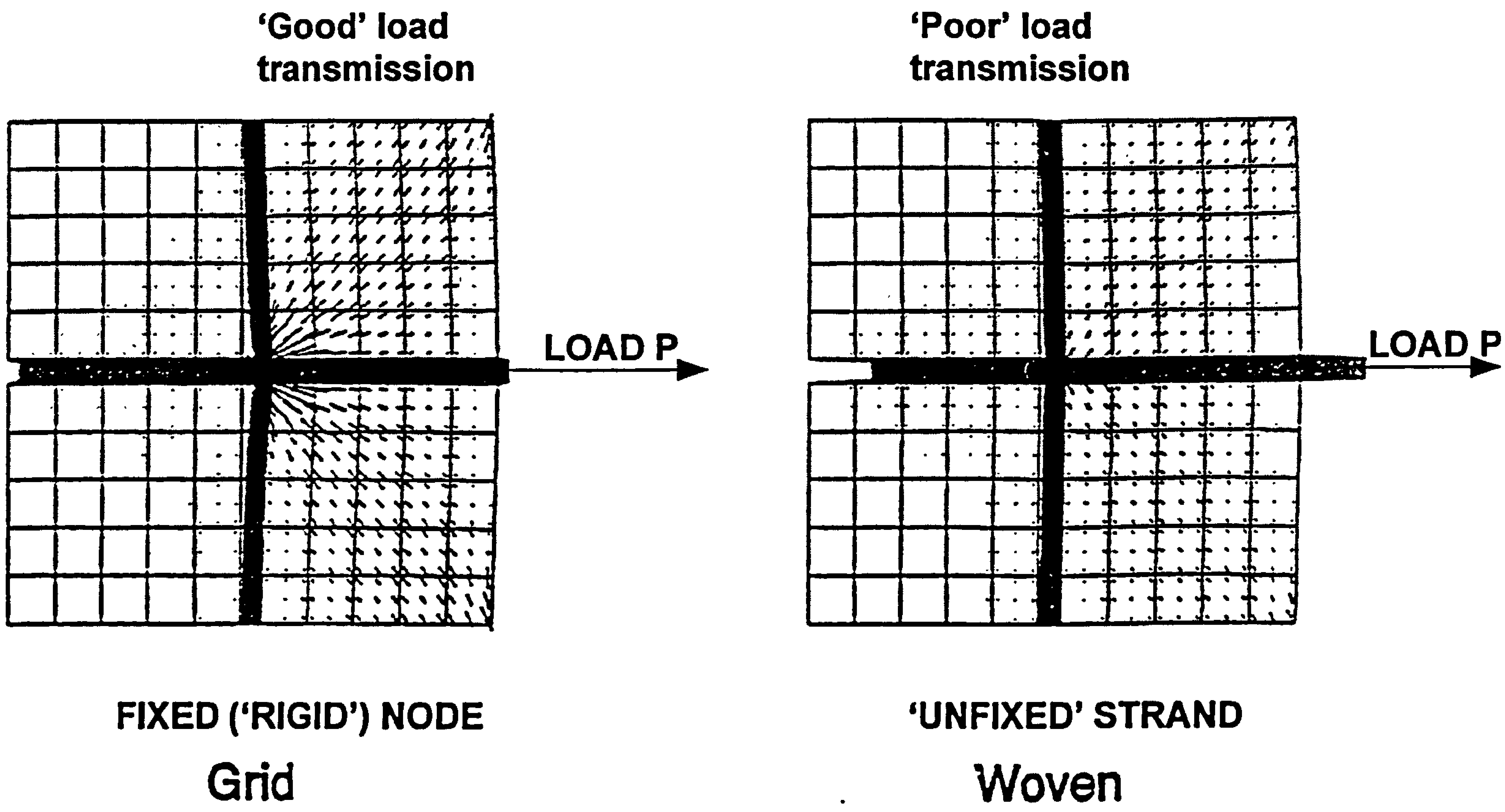

FIGURE 11.1

STRESS DISTRIBUTION vs NODE TYPE [11.4]

CHAPTER 12- SUMMARY AND OVERALL CONCLUSIONS

CONTENTS

CHAPTER 12 – SUMMARY AND CONCLUSIONS

12.1 General

An investigation into the mechanisms and uses of reinforced asphalt to counter cracking and rutting in asphalt pavements has been carried out using a combination of a desk study, laboratory testing and numerical modelling.

12.2 Desk Study

The desk study comprised

(i) a survey of organisations using grids in the UK to investigate the present ('local') use of reinforced asphalt, and

(ii) a literature review to obtain a wider perspective of the use of reinforced asphalt nationally and internationally, and to gain some insight into the mechanisms of reinforced asphalt.

The desk study showed that a variety of reinforcement products are available locally and internationally, although detail of how they perform is limited.

There was little indication that the mechanisms of reinforced asphalt were understood, or when reinforced asphalt would be cost-effective. The main points from the survey are given:

- Polypropylene grids tend to perform well on 'weak' foundations.

- Polypropylene grids and composites were perceived to be more effective than glass products where differential (vertical) movements at joints in concrete slabs were present.

- Geotextiles seemed to be effective over jointed concrete pavements. It was thought that this could be due to the bitumen-soaked geotextiles' ability to undergo high strains and reduce stress concentrations.

- For many, the perceived risks of using reinforced asphalt often outweigh the expected benefits, partly though the lack of a 'reliable' design method.

- The relatively high incidence of 'poor' performance appears linked to problems during installation.

Findings from literature relating to pavement performance largely complimented those from the UK survey.

Glass-reinforced products appear popular, due partly to their generally satisfactory performance, but also due to the ease of installation of the popular 'GlasGrid' product that uses an adhesive backing. In addition, this adhesive layer may provide a connection between the (much) stiffer glass reinforcement and asphalt which reduces the likelihood of debonding and thus improves performance.

Glass-reinforced products were reported to perform best where vertical deformations were limited and reinforcement was provided over the full lane or carriageway width, rather than in narrow strips. The stiffness of glass is often cited as a reason for its use, i.e. limiting strains within asphalt and helping to spread traffic loads over the softer foundation. However in practice it appears that the bonding between asphalt and reinforcement required to mobilise these properties can be difficult to achieve.

Geotextiles have been a popular choice for many organisations and have often performed well in practice. Typically these materials are non-woven fabrics that are placed between the surface course and the base. Similarly, (as for polymer reinforcement), these materials are not normally stiffer than asphalt, and therefore cannot behave in a traditional reinforcing mode. The way that these materials reduce cracking is similar to the way in which SAMIs (Stress Absorbing Membrane Interlayer) work, i.e. by absorbing and dissipating cracking energy. The SAMI effect is brought about by the thick bituminous tack coat that soaks the geotextile and fixes it to the asphalt, i.e. providing a 'reservoir' for bitumen.

For all reinforcement systems, there are limiting crack widths and deflections determining the limits of efficient use of geotextiles, and have been found to work best where cracks are typically closely-spaced and relatively narrow.

Good use can be made of the waterproofing qualities of geotextiles when soaked with bitumen, particularly on thin pavements on moisture-susceptible foundations. The flexible geotextiles are able to accommodate significant movements and remain waterproof, thus helping to stabilise moisture contents in the foundation. In some cases, geotextiles may be bonded to chip seals to provide a waterproof running surface. Geotextiles appear to have limitations in cold temperatures, which seems due to the binder becoming brittle and more susceptible to cracking.

Steel reinforcement has been used in one form or another since the 1950s with mixed success. The trend is now for thinner materials to be used in the form of hexagonal meshes with twisted wire nodes, and is generally reported as being effective in practice, particularly on overlaid concrete pavements. The reasons given for the good performance of these grids is that steel is inherently tough and interlocks well with the asphalt due to the wide apertures. Steel grids may be fixed to pavements with nails or bituminous slurrey, and if a slurrey is used, an additional waterproofing property is provided.

Design approaches for reinforced asphalt pavements vary widely, and none appear universally applicable. At the simplest level, reinforced asphalt is equated to a thickness of asphalt (often around 30mm) allowing an overlay to be thinned where property thresholds or bridge clearances, for instance, limit thickness. Conversely, more complicated approaches exist where each component of the reinforced asphalt composite is taken into account, i.e. asphalt, interface and reinforcement. This more fundamental approach is potentially applicable to any situation, provided that values for parameters used in the design are available. This unfortunately is not often the case, and sophisticated testing may be required to obtain the required values. Key unknowns in this regard are fatigue relationships for interface bonds and. values of appropriate values for interface bond and reinforcement stiffnesses.

Most design approaches are directed at reducing tensile strains in asphalt in the interface between the upper asphalt layer and the reinforcement. 'Top-town' cracking is not explicitly addressed by any method, although in practice, reinforcement may help reduce the rate of crack penetration in the same way as it appears to work for bottom-up cracking. Bitumen-soaked geotextiles may be particularly useful where top-down cracking occurs, as even if cracks penetrate through the reinforcement after being blunted and slowed down, the pavement still remains waterproofed.

The 'fundamental' design approach includes Finite Element Analysis which is useful to investigate the effects of variations in parameters, and in specific situations may be used to calibrate more simple approaches such as multi-layer linear elastic analysis.

Where a reinforced asphalt pavement has performed well on a pavement having high deflections, to be effective, the mechanism of reinforcement must be different to a 'normal' mode, where reinforcement is strong enough to reduce loading on the surrounding matrix.

Appraisal of the **economics** of reinforced asphalt in literature showed that often reinforced asphalt was not found to be cost effective. However this may have been due to poor construction practices and inappropriate application of reinforced asphalt rather than an inherent deficiency in reinforced asphalt.

Economic analysis using whole life costing showed that by assuming that reinforced asphalt increases crack resistance by a factor of three, and slows the rutting rate to 50%, reinforced asphalt can be cost effective providing that the volume of traffic is high enough.

12.3 Overall Approach of the Investigation

It was clear from the literature review and survey that an investigation into reinforced asphalt needs to take into account combinations of reinforcement, interlayer bonding, asphalt characteristics and loading. Consequently a combination of numerical modelling and laboratory testing was used where the characteristics of each component was to be measured and used as input for the CAPA-2D finite element programme. The programme was to be used to assist in analysis of the performance of beams of reinforced asphalt and then simulation of a full-scale field situation. The overall approach is illustrated in Figure 12-1.

12.4 Laboratory Testing

12.4.1 Interlayer Bond

To determine appropriate characteristics of the interlayer bond, a shearbox using cyclic loading was developed. The apparatus was used to measure the interlayer bond strength and shear stiffness between layers of 90 to120mm deep x 200mm wide x 320mm long samples. Results in Table 12.1 show how grid reinforced interfaces tend to have higher shear stiffnesses than composite reinforced interfaces and unreinforced interfaces. To calculate these stiffnesses, an interface stress distribution was assumed from results of a Finite Element Analysis carried out on a similar structure, found in literature.

Table 12.1 Interface Shear Stiffnesses

Reinforcement type	**Ranges of shear stiffness ($N/mm/mm^2$)**	
	100 kPa applied shear stress	**150 kPa** applied shear stress
Unreinforced	11	13
Grids	8-20	13-43
Composites	10-20	8-18

In the light of test results from the beam and PTF tests, the values in Table 12.1 imply that interlayer shear stiffness is only one component of reinforced asphalt determining performance; although grids provide a higher interlayer shear stiffness than composite materials, PTF results showed composite- and grid-reinforced asphalt to follow the same trend of resistance to cracking.

12.4.2 Grid Strength and Stiffness

The tensile strength and stiffness of reinforcement was measured at three rates of loading. Test results showed that the stiffness of Polypropylene reinforcement increased by between 20 and 40% as the test rate increased from 0.5 to 50.0mm/minute, whereas the change in properties of glass reinforcement was insignificant, at the same test rates – see Table 12.2.

Table 12.2 Test Results

Material	**Test Rate** (mm/minute)	**Max EA per strand** (kN/ε)
AR1	0.5 5.0 50.0	25.0 27.2 29.8
AR-G	0.5 5.0 50.0	20.2 25.7 27.9
ROTAFLEX 833	0.5 5.0 50.0	13.5 14.1 13.2

Notes: 1 Transverse direction
2 Longitudinal direction

The properties of steel grids were not obtained, as they could not be tested in the same way due to grid geometry and the nature of the wire nodes.

12.4.3 Asphalt

Asphalt density and stiffness were measured on material cored from test samples and pavements.

There were marked differences in material quality between samples compacted using the roller-compactor (beam and shearbox), and material compacted in the PTF. A summary of the densities is given in Table 12.3.

Table 12.3 Summary of average density and air void measurements

Compaction type	Material	Top Layer		Bottom Layer	
		Density (MG/m^3)	Air voids (%)	Density (MG/m^3)	Air voids (%)
Roller-compacted Samples (Beam and Shearbox)	AR1	2.47	4.2	2.44	5.2
	Road-Mesh	2.44	5.2	2.44	5.2
	Glass-reinforced	2.45	5.4	2.43	5.7
	Unreinforced	2.46	4.7	2.47	4.2
Pedestrian roller-compacted (PTF)	AR1	2.08	14.9	2.1	14.0
	Road-Mesh	2.22	13.5	2.21	14.3
	Glass-reinforced	2.21	14.3	2.19	15.0
	Unreinforced	2.23	13.5	2.24	12.8

12.4.4 Beam Testing

A beam testing apparatus was developed to investigate crack growth through reinforced and unreinforced layer interfaces. A configuration was developed where two-thirds of each beam was supported on rubber sheets and the middle third of the beam left unsupported. This allowed cracks to propagate from the bottom of the sample upwards but avoided the effects of permanent deformation found with the 4 point bending test configuration that was initially used. Crack patterns were recorded and showed that crack propagation varied with reinforcement type.

Overall, cracks took up to three times longer to propagate through a 20mm band around reinforced interfaces than was the case around unreinforced interfaces.

The strength of reinforcement was not seen to be the principal factor determining crack resistance, as polypropylene-reinforced interfaces performed well despite being less stiff than glass- and steel-reinforced products.

12.4.5 Pavement Test Facility Tests

Five unreinforced and ten reinforced 'half-scale' sections were built and tested under repeated 12kN wheel loading.

Reflective cracks were initiated using 600mm x 600mm x 60mm concrete paving slabs, on a Type 1 subbase.

The relative performance of the different sections were judged by rut development and the number of wheel loads taken before 'active' cracks appeared on the surface, i.e. cracks that could be seen opening and closing with wheel passes.

In the initial PTF test trafficking resulted in large shoulders along the wheelpath. which made identification of longitudinal reflective cracks difficult. Transverse cracks on the other hand were generally easily identifiable as they were located close to transverse joints and between paving slabs.

To reduce the permanent deformation (shoulders) and to encourage the development of reflective cracks, a 5mm rubber sheet was placed between the subbase and the concrete slabs, and the test temperature was reduced to around 13°C.

To compare the performance of all sections, single 'equivalent' deflection representing the behaviour of each section up to the appearance of surface cracking was calculated.

Overall, PTF trafficking showed that reinforced sections were able to withstand around two to three times the wheel loading taken by unreinforced sections. In addition, the results of all the reinforcement types used in the tests followed a similar trend.

Sections reinforced with materials that interlock well with asphalt were found to reduce rutting to around 50% of the unreinforced sections. This may be linked to the pavement structure used, i.e. with rigid paving slabs beneath the asphalt, a large component of deformation is horizontal, and it is this that is restricted by the reinforcement.

12.5 Numerical analysis of reinforced asphalt.

To aid the analysis of the beam test configuration used during the project, and to model a full pavement structure, the Finite Element programme CAPA-2D was used. The programme was particularly useful for assessing the effect of varying different parameters such as interlayer bond, asphalt and reinforcement stiffnesses.

The analysis showed that the bond between reinforcement and asphalt is important, and has a large effect on the behaviour of reinforced structures. This has implications in the design and construction of reinforced asphalt.

For more accurate modelling of the beam, better definition of parameters is required. In particular, realistic values for the interface bond stiffness, and the effective stiffness of the reinforcement when confined, are needed. To obtain these parameters, further development of test methods may be required.

A limited investigation showed that limited debonding has a potentially beneficial effect on slowing crack propagation. Additional modelling and calibration with test results is required to apply this in practice.

To model the 3-D effects of loading on a pavement, deflection bowls calculated using multi-layer linear elastic theory were simulated using FE analysis. Once a good fit was obtained, interlayer bond and asphalt stiffnesses were varied to calculate their influence on the rate of cracking. This exercise showed that without stiff interlayer bond, the stiffness of the reinforcement is largely insignificant.

12.6 Guidelines for the use of reinforced asphalt.

Details of proposed guidelines are derived in part from the laboratory investigation and partly from findings from the literature review. General points are given, together with some detailed recommendations on deflection limits and limiting crack widths, taken from literature.

To specify reinforcement, a site investigation to characterise the nature of existing distress must be carried out. This will probably include a visual inspection, deflection measurement across cracks and joints, and coring through cracks. This will assist in the selection of an appropriate reinforced asphalt solution, through use of the limits on crack widths and deflection given in Chapter 11. The basic factors to consider are now given:

For pavements subject to **large deflections**, effective reinforcement must either be strong enough to reinforce the pavement in a conventional fashion, or be sufficiently flexible to strain with the asphalt. This helps to maintain the bond between the different layers and hence retain pavement strength. If inadequate bond between layers exists, the asphalt will tend to act more as a succession of separate layers, and thus be prone to early cracking, due to higher tensile strains.

If a pavement is subject to large deflections which in turn are due to **changes in moisture content** of active clays, for instance, a bitumen-filled geotextile may help in reducing these changes. Furthermore, geotextiles bonded to chip seals have been shown to provide a durable waterproof wearing course.

For **thermal cracking** situations, reinforcement needs to be able to distribute strain caused by differential contraction or expansion over a length that results in strains being small enough to be accommodated by the asphalt. This means that a degree of slip between the reinforcement and the layer causing crack propagation is desirable.

For **thin pavements** that tend to crack from the lower asphalt interface, all types of reinforcement can improve the longevity of the pavement by a factor of between two or three if correctly installed, and if limitations of the various reinforcement systems are taken into consideration. Limitations may include relative deflections across cracks or joints being excessive leading to shearing of brittle reinforcement.

Reinforcement can be effective in **reducing rutting** if it is placed at the level in the pavement where horizontal shear stresses are highest. Laboratory tests show that to do this, reinforcement needs to have a good bond with asphalt to be effective, and empirical findings seem to show that interlock is more dominant than adhesion. In this regard grids with a large profile work well.

The laboratory investigation has been largely carried out with thin pavements in mind, where bottom-up cracking is thought to be the main mode of failure. In particular, PTF tests were configured to develop cracks that initiate from the bottom of the asphalt. Guidelines based on the results of laboratory testing may therefore only be applicable to similar configurations.

Taken in isolation, the results of tests on each component of reinforced asphalt (asphalt, reinforcement and interlayer bonding) are not usually good indicators of the overall performance of reinforced asphalt. Although the selection of suitable reinforcement might be defined quite simply in some cases, such as where waterproofing the pavement is important, or where a pavement is founded on a soft foundation, in practice situations are seldom straightforward. The interaction between factors determining reinforced asphalt pavement performance requires more definition to help identify the most important factors in any given situation. Once these factors are defined, a 'simple' index test or tests might be found that allow quick assessment of the likely performance of reinforcement.

Construction

The construction of reinforced asphalt pavements is critical to the performance of the final product. The most careful assessment of a pavement to be treated and adherence to design guidelines is of no consequence if reinforcement is installed incorrectly.

Poor installation can occur due to (a) the added complexity of additional pavement layers, and (b) a lack of understanding by the contractor of which aspects of construction are critical to the performance of reinforced asphalt.

It is recommended that only companies approved by reinforcement manufacturers are used to install reinforced asphalt.

12.6 Further Work

During the course of the investigation a range of questions have emerged that require answers. An extensive range of topics for further investigation have been listed in Chapter 13, from which the main points are given:

- **Definition of cracking mechanisms in the field**
  This is the most important issue of all and could be the main reason that reinforced asphalt is not regarded as a reliable option in many cases. If top-down cracking is widespread, then the effect of reinforcement on these cracks needs to be investigated, as the mechanism appears to be quite different to that which has been investigated during the course of the research.

- **Interlayer bond**
  The importance of interlayer bonding has been highlighted from the results of testing and modelling, and needs better definition. Theoretical modelling and testing will be required. Thereafter, means of achieving the recommended values in-situ will need to be defined.

- **In-situ reinforcement stiffness**
  The stiffness of reinforcement has been defined through unconfined tensile tests. These values of stiffness may not be appropriate for use in modelling and design, and the effect of the confinement on reinforcement stiffness, especially for steel grids with twisted wire mesh, needs to be determined.

- **Defects in Construction.**
  The effect of 'defects' in construction are unquantified, especially the effects of cutting and lapping reinforcement at bends and joins. Further work should be carried out in this area. Also, the sensitivity of performance to various degrees of interlayer bonding (especially tack-coat).

- **Compaction and voids.**
  The effect of reinforcement on compaction should be investigated, especially where stiff materials with relatively small apertures are used.

- **Modelling**
  3-Dimensional modelling should be carried out to investigate realistic loading conditions, and combine the effects of environmental and traffic loading. Furthermore, for all modelling, whether 2-Dimensional or 3-Dimensional, linear or non-linear, means of obtaining appropriate measures of input parameters need to be confirmed. Reliable means of achieving these values in-situ then need to be confirmed.

- **Permanent Deformation**
  Further empirical and theoretical validation is required to establish the mechanisms and quantify the effect of grids on permanent deformation.

- **Economic Appraisal/Whole Life Costing**
  More details on the costs and duration of installation, the effect (delay of crack and rut formation) and longevity of reinforced asphalt is required. More informed decisions will then be possible for clients and designers when recommending or commissioning reinforced asphalt.

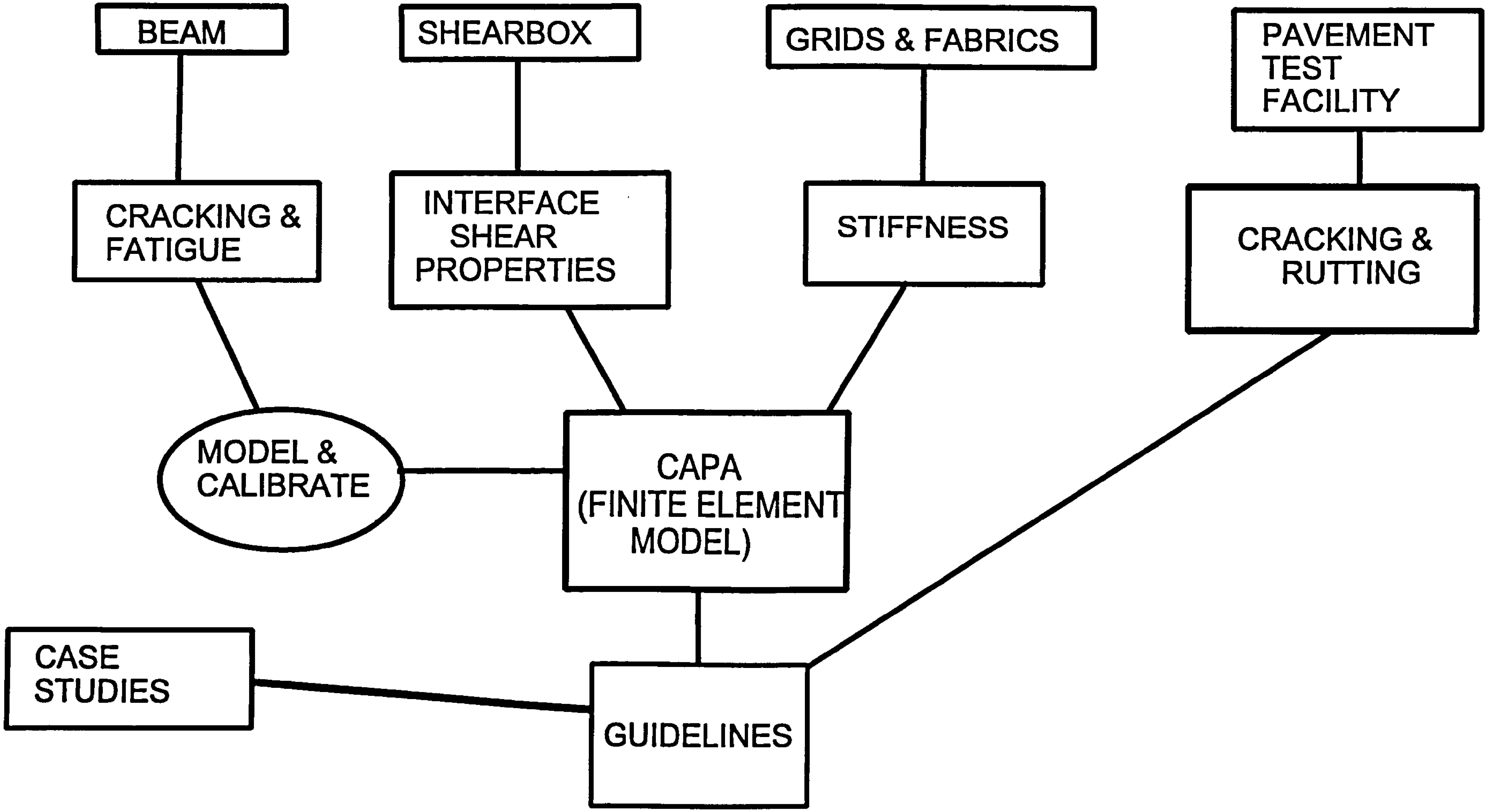

FIGURE 12-1
REINFORCED ASPHALT:OVERVIEW OF INVESTIGATION

CHAPTER 13

PROPOSALS FOR FUTURE WORK

CHAPTER 13

CONSIDERATIONS FOR FUTURE WORK

13.1 Introduction

From the results of the investigation described in preceding chapters, it is clear that many questions remain unanswered. This is true for each area of the investigation: theory and modelling, laboratory testing and the application of theory and laboratory test results to full-scale behaviour and site practice.

Overall, probably the most important overall issue that needs to be resolved relates to the application of test results from the laboratory investigation to practice. A key question in this regard is the suitability of the test modes used to date. In particular, the wheel tracking tests and the beam testing were both carried out at constant temperatures and loading frequencies, respectively. These test modes may have limited applicability to in-service pavements, as it seems that a significant percentage of cracks encountered on trunk and principal roads are largely due to environmental (particularly temperature) effects. These cracks are predominantly top-down, and in the main seem to be a result of combinations of temperature changes and wheel loading. Little evidence is available to show the effect of reinforcement on this type of crack, either in literature or from the tests carried out. Apart from the low temperature tests, wheel tracking and beam tests were carried out with dynamic loading at 20°C (except for PTF test 3) to simulate traffic effects, and configured to generate bottom-up cracks. It is not obvious whether reinforcement has the same effect on cracks from the slower temperature-induced loads as it does on (faster) traffic loading. The combination of environment and traffic loading on cracking should therefore be investigated under realistic temperature regimes, i.e. repeated heating and cooling cycles. Also, the effect of temperature may have a significant effect where the bond between reinforcement and asphalt is due to bitumen adhesion rather than grid-asphalt interlock. In winter temperatures the bond would be expected to be stiffer and more brittle than in summer temperatures. This may mean that the reinforcement–asphalt interface is more susceptible to debonding and cracking in the winter, and that slip is more likely in the summer. Trials using modified binders whose properties are less temperature susceptible than straight run bitumen may be advantageous in these situations.

More specific areas of work that need further investigation are now discussed.

13.2 Laboratory Testing-General.

The amount of samples tested during the investigation was small in relation to the number of unknowns. To verify findings and to characterise the variability of test results, therefore, more samples should be tested in each test configuration.

All numerical models (whether they be OLCRACK or a Finite Element Analysis, for example) require appropriate measures of input parameters. To obtain these parameters, standardised test methods need to be developed. These are required to measure (*inter alia)* reinforcement stiffness, (possibly under confinement) and shear properties of interface bond.

To provide parameters for a range of *in situ* conditions, tests should be conducted with different combinations of asphalt, reinforcement and interlayer bond, and at different speeds. As a limited number of generic reinforcement types and asphalt mixtures are typically used in practice, a set of design tables covering the majority of situations found in the field could be developed to reduce the need for testing.

The present investigation was largely confined to appraising the efficacy of reinforcement in retarding cracking and rutting due to repeated loads at 5Hz (the beam test) and around 8km/hour in the PTF. However, a limited investigation was also carried out using creep loading with the beam test configuration, and showed that the dead load used in the beam test did not have any noticeable influence on cracking. Further work is needed to confirm this observation with repeat testing and with tests on other types of reinforcement. The influence of reinforcement that relies on binder adhesion rather than interlock is of particular interest, as the visco-elastic bond may lead to different effects, especially under 'cold' and 'hot' conditions. The information provided is relevant for the design of reinforced asphalt for pavements under slow moving and stationary vehicles, such as car and lorry parks, for instance.

13.2.1 Interlayer Bond

The importance of interlayer bonding has been highlighted from the results of testing and modelling, but with the data at hand, the required values for the interlayer bond in practice are difficult to define. Interlayer bond needs to be able to connect layers sufficiently to resist traffic loading, especially at corners and gradients, for instance, but still permit a degree of slip between layers at low temperatures. To determine appropriate values of interlayer bond, therefore, additional modelling and testing is required. Furthermore, once appropriate values have been defined, means of achieving the recommended values in-situ will need to be found.

Further investigation of the **effects of debonding** between reinforcement and asphalt is required. Then, if a degree of debonding is found to be advantageous, methods of obtaining this condition in the field will be required. To do this, it seems likely that further testing and modelling will be needed, and will need to be calibrated with site trials.

Fatigue tests on unreinforced specimens in the **shearbox** show that the interface bond deteriorated at a slower rate than on samples with interlayers partly comprising a thick layer of bitumen emulsion. It is thought that this is due to a better bond (interlock) between the upper and lower layers of the specimens. More testing is required to confirm this possibility and to define the fatigue behaviour of the bonds between different reinforcement types and asphalt.

The **shearbox test** is not suited to routine measurement of interface shear values. A more practical interface shear measurement test would therefore be useful to provide appropriate values for design. If it is possible to reliably relate test results from the shearbox to a shear test like the simple shear apparatus used to test samples cut from the PTF, the simpler test might be used as a proxy for the shear box. Theoretical and practical work needs to be carried out to investigate possibilities in this area.

13.3 Sample size and shear tests

The results of the shear tests carried out on samples taken from the PTF are thought to be influenced by edge effects, which are linked to sample size. The effect of relatively high edge stresses is likely to be greater on the relatively small (100mm x 100mm x 60mm) blocks cut from the PTF pavement than on the larger (380mm x 200mm x 120mm) shearbox samples. Quicker initiation and propagation of bond failure is expected over the small sample area giving an over-conservative result (low value of shear). More investigation into the effects of sample size on stress distribution and the implications for bond failure should be carried out to help resolve differences in test results.

13.4 Monotonic versus cyclic loading tests.

Failure under cyclic loading occurred at lower applied stresses than under single failure loads. In addition it was also noted that the ratio of monotonic to cyclic failure loads was greater for unreinforced samples than for reinforced samples. There are various possible explanations to explain this which include: (1) natural variation of material properties (which was not defined with the limited number of test results), and (2) the interlock between grids and asphalt deteriorating more slowly than asphalt-bitumen bonds. Additional testing is required to define the mechanisms responsible for these apparent differences, as they hold potentially important implications for reinforced pavement design.

Comparisons of behaviour under monotonic and cyclic loading for other types of reinforcement are required, (especially fabrics and composite reinforcement), to see if similar reductions in loads required for failure exist.

13.5 Reinforcement Properties

The investigation was carried out only using reinforcement that is commercially available, and with a single asphalt mixture. Results show that not all types of reinforcement give the same performance in the same situations. It therefore seems likely that there is scope for more development of some types of reinforcement and, using results of the investigation, possibly a new product. However, to define more optimal properties for reinforcement in particular situations, additional numerical modelling and testing is required, followed by site trials.

For grids, the elasticity of the material and the geometry of the apertures are likely to be important. This seems particularly important where large pavement deflections are expected, and grids need to be well-bonded with asphalt but also able to move (stretch) as the asphalt layer deforms without causing high interlayer shear stresses.

13.5.1 Stiffness testing of reinforcement.

Although the stiffness of reinforcement may not be an over-riding factor determining the effectiveness of reinforced asphalt, the effective in-situ stiffness of reinforcement needs to be determined to model and understand the controlling mechanisms. In particular, the effect of confinement on the stiffness of products such as Road Mesh is likely to be significant, and may have significant bearing on the understanding of mechanisms of reinforced asphalt.

A test procedure should be developed where reinforcement is tested within asphalt. This is required for reinforcement that has a geometry which makes it unsuitable for unconfined testing, (such as Road-Mesh). To do this, reinforcement could be cast in a slab with a separating plate or membrane across the middle of the sample in both top and bottom layers (see Figure 13-1). This would leave the reinforcement as the only material connecting the two halves of the sample. Therefore, by applying load across the sample, the effective stiffness of the confined reinforcement could be derived from the relationship between load and deflection.

13.5.2 The ratio of aperture opening to aggregate size.

The question of optimal ratios between aggregate size and shape and aperture openings also needs investigation. A proposed ratio of between 3 and 4 was found in literature but it is not clear how this value may alter with different asphalt mixtures and aggregate shapes. This ratio has important implications in providing an asphalt-reinforcement combination that does not impede compaction, and provides in-service asphalt interlock. In addition to the minimum ratio of aperture and aggregate, which seems to influence construction-induced problems, a value for a maximum ratio would also be useful. This value will define a value for the ratio beyond which cracks propagate through apertures 'unimpeded', i.e. with reinforcement having little or no effect.

13.6 Beam Testing

Study of the crack patterns in the beam tests suggests that reinforcement can in some way influence cracking before cracks have reached the reinforcing layer. The reason for this phenomenon is not obvious and needs investigation. It is suspected that factors such as the stiffness of the grid and degree of interlock play an important role in suppressing early crack development, but this needs confirmation, and it is suspected that there may be more than one mechanism causing this phenomenon. When this, (or these) mechanism(s) have been defined, it (they) need to be incorporated into design methods to maximise the delay in crack development.

Once cracks propagate above the interface the delay in cracking in glass- and polypropylene-reinforced beams was particularly noticeable. Steel reinforcement, on the other hand had a larger influence on crack resistance when cracks were below the interface. These observations should be confirmed with more testing. Then, if this behaviour was found to be consistently repeated, the controlling parameters need to be determined. This is potentially important for the analysis of test results from the laboratory and field, and will have an influence on design.

13.7 Modelling

Modelling was carried out using a finite element analysis programme written for 2-Dimensional loading configurations with linear–elastic materials. The approach was generally found suitable for analysis of the beam test but had limitations in modelling wheel loads on pavements. 3-dimensional modelling should therefore be carried out to investigate more realistic loading conditions, and include both environmental and traffic loads. However, if non-linear models are used, means of obtaining appropriate measures of input parameters are required. This may mean that 'new' tests able to produce values applicable for in-service conditions are to be developed.

For the analysis of cracking in beam tests and for modelling cracking through an in-situ pavement structure the Paris law was used, which requires values for parameters A and n. In the absence of test data these values were assumed from literature. Testing needs to be carried out to confirm the applicability of the values used, and to explore the likely variation of these parameters.

Testing in the PTF has shown that the presence of reinforcement, particularly grids, can help to reduce **permanent deformation**. Further testing on different pavement structures and additional numerical modelling is required to justify this apparent benefit.

Reinforcement mechanisms for soft pavements.
Only empirical evidence exists to show polymer and steel grids to be effective when used in pavements on soft foundations. Both theoretical modelling and testing are therefore required to establish and verify the supposed mechanisms. In particular, the relationship between stiffnesses of asphalt, interlayer bond (whether adhesion or interlock), and reinforcement and performance is thought to be important.

13.8 Whole Life Costing

A more extensive whole life costing exercise should be carried out to establish in which situations reinforced asphalt is cost effective. This will require more information on the longevity of reinforced asphalt pavements and failure modes, and other issues such as their suitability for recycling.

13.9 Sustainability

Similarly, as for whole life costing, an investigation into issues relating to sustainability should be carried out for reinforced asphalt. The effect on 'sustainability' of reducing asphalt thickness, and thus energy consumption and the need for quarrying are aspects that need to be taken into account.

In considering sustainability, it seems likely that overall, reinforced asphalt will show benefits. If this is the case, the positive data relating to sustainability could be useful for promoting the use of reinforced asphalt.

13.10 Construction Techniques

The desk study identified poor installation of reinforcement as one of the main causes of poor performance. However, the most important procedures carried out during construction that ultimately determine the performance of reinforced do not seem have been identified or quantified. To help resolve this issue therefore, an investigation should be carried out where defects are systematically built into samples. For example, broken or twisted reinforcement, fabric with insufficient bitumen tack coat, or reinforcement with an inadequate thickness of asphalt overlay might be used.

The implications of cutting grids and fabrics to negotiate bends and other pavement anomalies is also required, and also the effects of 'lapping' successive lengths of reinforcement. These issues are particularly important where reinforcement is applied near the surface of the pavement, as (in the short term) mistakes in the surfacing are normally more critical than when defects are present deeper in a pavement.

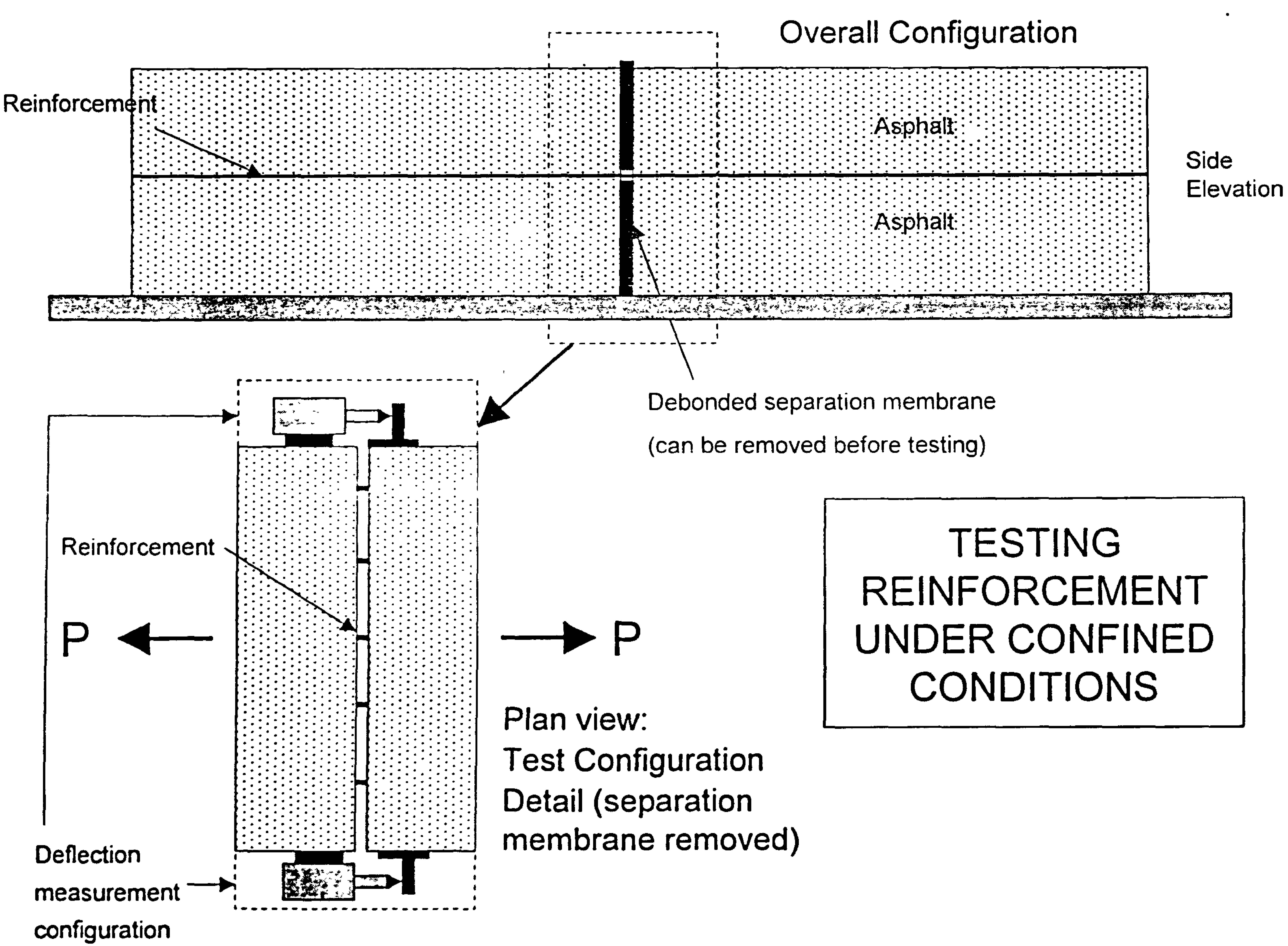

FIGURE 13.1
PROPOSED TEST CONFIGURATION