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Understanding Low-Volume Pavement Response to Heavy Traffic Loading

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PREFACE

This is a final report from Task B2 of the ROADEX III project, a technical trans-national cooperation project between The Highland Council, Forestry Commission Scotland and Comhairle Nan Eilean Siar from Scotland; The Northern Region of The Norwegian Public Roads Administration; The Northern Region of The Swedish Road Administration and the Swedish Forest Agency; The Savo-Karjala Region of The Finnish Road Administration; the Icelandic Road Administration; and the Municipality of Sisimiut from Greenland. The lead partner in the project is The Northern Region of The Swedish Road Administration and project consultant is Roadscanners Oy from Finland.

This report summarizes the study performed primarily at the University of Nottingham and at the Tampere University of Technology into the causes and development of rutting in low-volume roads and of the approach developed in this project to allow such pavements to be designed against rutting. The work has been carried out in close collaboration with Task B2 "Tyre Pressure Control on Timber Haulage Vehicles" the results of which are presented in a separate report. The report was prepared by Andrew Dawson of the Nottingham Centre for Pavement Engineering at the University of Nottingham (UK), Pauli Kolisoja and Nuutti Vuorimies of the Institute of Earth and Foundation Structures at the Tampere University of Technology, Finland, on behalf of the Task B2 project team which also included Ron Munro of Munroconsult Ltd, working under sub-contract to Roadscanners Oy and Frank MacCulloch of Forestry Commission Scotland.

The authors would like to express their gratitude to Heikki Luomala of Tampere University of Technology and Lelio Brito of the University of Nottingham who have performed many of the calculations that underwrite the information presented in this report and have acted as "guinea pigs" in testing out the concepts and procedures outlined herein. Their assistance is greatly appreciated.

Finally the authors would like to thank the ROADEX III Project Partners and the Project Steering Committee for their guidance and encouragement in this work.

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Chapter 1. INTRODUCTION

1.1. THE ROADEX PROJECT

The ROADEX Project is a technical co-operation between roads organisations across northern Europe that aims to share roads related information and research between the partners. The Project was started in 1998 as a 3 year pilot co-operation between the roads districts of Finnish Lapland, Troms County of Norway, the Northern Region of Sweden and The Highland Council of Scotland and was subsequently followed and extended with a second project, ROADEX II, from 2002 to 2005. and a third, ROADEX III, from 2006 to 2007.

The partners in ROADEX III "The Implementation Project" comprised public road administrations and forestry organizations from across the European Northern Periphery. These were The Highland Council, Forestry Commission Scotland & Comhairle Nan Eilean Siar from Scotland, The Northern Region of The Norwegian Public Roads Administration, The Northern Region of The Swedish Road Administration and the Swedish Forest Agency, The Savo-Karjala Region of The Finnish Road Administration, the Icelandic Road Administration and the Municipality of Sisimiut from Greenland.

Figure 1.1 Northern Periphery Area & ROADEX III partners

A priority of this Project was to take the collected ROADEX knowledge out into the Partner areas and deliver it first hand to practising engineers and technicians. This was done in a series of 14 seminars across the Partner areas to a total audience of 800. Reports were translated into the 6 partner languages of Danish, Icelandic, Finnish, Greenlandic, Norwegian and Swedish as well as English. ROADEX research continued through 5 projects: measures to improve drainage performance, pavement deformation mitigation measures, health issues of poorly maintained roads, road condition management policies, and a case study of the application of ROADEX methodologies to roads in Greenland. This report is a sub-report of Task B2 "Permanent Deformation" and has been done in close collaboration with the Task B2 sub-task "Tyre Pressure Control on Timber Haulage Vehicles" the results of which are presented in a separate report. All of the reports are available on the ROADEX website at www.roadex.org.

1.2. BACKGROUND

Roads are extremely important for users in the Northern Periphery, but the income to fund them and the locally available skills to achieve successful road construction and performance is usually limited. There is, therefore, an important requirement to provide reliable design and evaluation procedures that will allow local engineers to better support the operation of their pavements.

However, procedures that introduce demands of time and expertise that are, practically, impossible to obtain or maintain by the engineers who are responsible for road construction and maintenance – probably alongside many other duties in their rural situations – are unlikely to be successful. For a procedure to be adopted and utilised, it must be

- a technical advance on what has gone before – otherwise it makes no useful contribution,
- simple and quick to use and apply – otherwise it will not be adopted,
- provide an improved understanding of real pavement behaviour – so that users are encouraged in their adoption of the new procedure and so that they can appreciate the procedure's limitations.

This last point is believed to be particularly important as unthinking application of a procedure is likely to bring the whole procedure into disrepute, thus leading to loss of the genuine benefits that it could have delivered.

1.3. INTRODUCTION

This report describes, in greater detail and following further work, a procedure originally developed, in outline, during the ROADEX II project (Dawson & Kolisoja, 2005). The original method set out the basic engineering approach and the framework for design of low-volume road pavements either incorporating a chip-sealed surface or no surface at all. The chief structural layers were considered to be aggregate and the whole placed on a natural or imported fill foundation.

This scope of the design procedure discussed in this report has not changed. What has developed is the quality of the detail underlying the design approach and the presentation of the method. In addition this report describes the use of the method in more detail than was possible in the earlier report. In particular, this report sets out a practical and useable method of designing pavements against rutting which, it is hoped, will meet the goals set out in Section 1.2.

Chapter 2. FUNDAMENTAL UNDERSTANDING OF, & SOLUTIONS FOR, RUTTING

2.1. INTRODUCTION

Low-volume roads where the structural layers are granular in nature – and this type of structure represents the vast majority of roads in the Northern Periphery area – experience deterioration mostly in the form of rutting. Ultimately pavements may completely fail, even becoming impassable, due to excessive rutting, potentially requiring costly reconstruction if the cause is deep-seated. Even rutting due to near-surface effects will result in the need to re-grade unsealed roads or to reconstruct / overlay the surfaces of sealed pavements. In the meantime, problems of increased rolling resistance, steering difficulties, water collection, etc. may manifest themselves, making undesirable the distress from which they come.

Of course, there are other forms of distress – for example too high a resilience giving rise to excessive energy needs to travel along the road (and hence to unnecessary carbon emissions) and too great a longitudinal unevenness causing driver discomfort and speed limitations. Often the latter is, in effect, the result of localised rutting so design to prevent rutting will be likely to address this form too.

When rutting occurs it does so because the materials of which the road is constructed are inadequate for the purpose of carrying repeatedly applied, vehicle imposed, loads without plastically deforming. Sometimes the structural, aggregate, layers are of poor quality material that does not have sufficient resistance to deformation under such repeated stressing. More often the material could have sufficient load-carrying capacity, but that has been lost due to a degradation in property due to some environmental effect (wetting or frost loosening due to previous impact of rain or cold).

Thus a design against rutting can address a fundamental failure mode and, also, provide the means to withstand failure contributed to by other agents than wheel loading.

2.2. UNDERSTANDING RUTTING

If rutting is such an important means by which low-volume pavements suffer distress and fail, then it is important that engineers have a clear understanding of the types of rutting that might be seen in practice and an understanding of the reasons for their occurrence and of the factors that control them.

If there is no clear understanding of the mechanism(s) of rutting, what it influences, nor what it is influenced by, then remedial or avoidance approaches will be, at best, inefficient and, at worst, misguided. Therefore a first step is

- to analyse the modes of rutting,
- to explain the differences between the modes and
- to identify the key influences on each mode.

Then each mode can be modelled, as appropriate, and its contribution to overall rutting assessed. In the same manner the effect of possible remedial treatments or of new designs can be studied and reliable comparisons of alternative options may be achieved.

For this reason the new design approach has, as a fundamental goal, the aim of providing a knowledge framework for engineers in the Northern Periphery to design and assess road pavements.

2.3. DEFINITION OF MODES OF RUTTING

The earlier report (Dawson & Kolisoja, 2005) introduced four rutting modes as a means of understanding. They are as follows:

- Mode 0 = Vertical depression only, due to compaction,
- Mode 1 = Shear in the aggregate layer(s) only,
- Mode 2 = Shear in the subgrade only,
- Mode 3 = Vertical depression due to particle wear and loss.

Each mode is described more fully in the earlier report, and illustrated there in detail. Only a conceptual illustration of Modes 0 to 2 is shown here together with the following brief descriptions. Readers seeking a fuller explanation should refer to the earlier report. In practice rutting will usually be a combination of these mechanisms.

Mode 0: Rutting of this type is seen as a narrow depression relative to the original surface. The material affected is mostly near the wheel.

Mode 1: Local shear close to the wheel will give rise to heave immediately adjacent to the wheel path. This rutting is mostly a consequence of inadequate shear strength in the aggregate relatively close to the pavement surface. The maximum shear movement tends to occurs at a depth of approximately 1/3rd of the width of the wheel. Observations suggest that this mode is the most common of those considered here.

Mode 2: The whole pavement will rut if the subgrade undergoes shear deformation, with the granular layer(s) deflecting bodily on it (i.e. without any thinning). This results in a surface deflection comprising a broad rut with slight heave remote from the wheel.

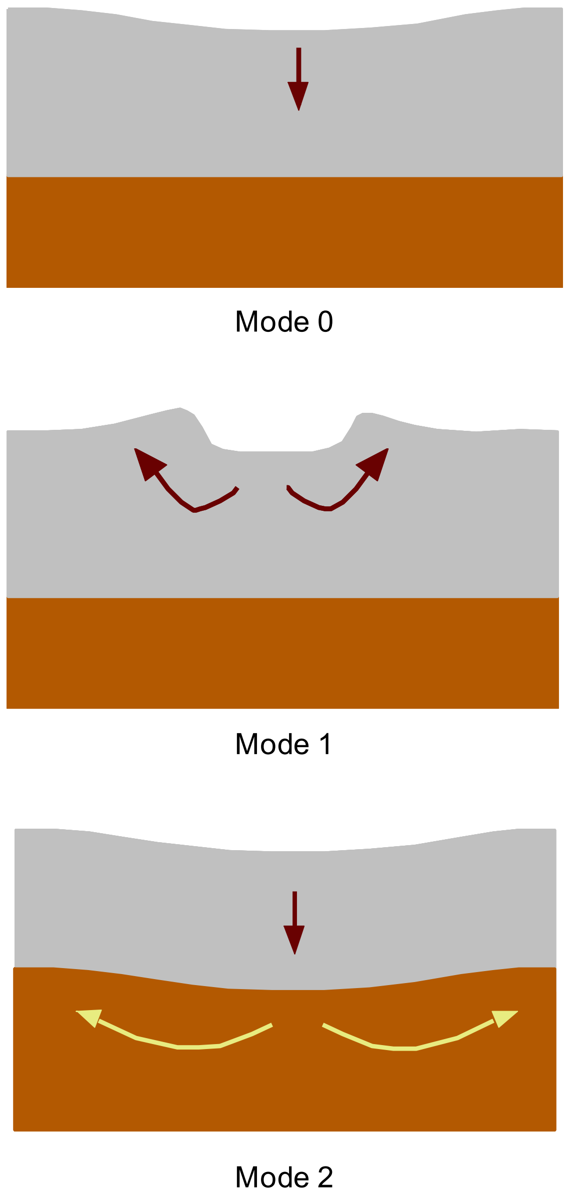

Figure 2-1 ***Schematic of Rutting Modes***

Mode 3: Particle damage and wear, principally of unsealed roads, can cause a similar surface rut as for Mode 0 rutting, with the “lost” material either being taken into existing voids in the aggregate or dispersed through dust, by wash-off or by displacement by wheels away from the trafficked zone.

2.4. FACTORS INFLUENCING RUTTING

In the earlier report (Dawson & Kolisoja, 2005) it was shown that rutting of granular pavements has, essentially, two ultimate causes:

- Volume reduction of the road construction or subgrade materials below the wheel, and
- Shear displacement of the road construction or subgrade materials below the wheel to a position marginal to the wheel path.

In principle, the more proximal cause can come in many varieties – for example:

- particle wear,
- compaction,
- particle breakage,
- inter-particle shear,
- inadequate stress distribution resulting in over-stressing of an element of the construction,
- inadequate material shear-strength,
- pavement overloading,
- high pore water pressures due to traffic loading that results in material weakening,
- material disruption due to frost-heave,
- thaw weakening,
- as well as general construction defects such as inadequate compaction, thin parts of the pavement, etc.

In practice, rutting often occurs when more than one of these causes combine to provide a pavement layer or material that is weak compared to the demands put upon it by the traffic loading.

It follows from the above list that the condition of both the subgrade and the aggregate are likely to be major factors that affect rutting.

2.5. BASIC OPTIONS FOR REDUCING RUTTING

Solution strategies can be grouped into two, principle, types:

- Improve the aggregate – this is the appropriate solution where defects in the aggregate quality are allowing shear or damage within the granular layer(s) of the pavement.
- Thicken the construction – this is the appropriate technique when the aggregate layer thickness is inadequate to prevent the subgrade (or lower aggregate layer) from being overstressed by traffic loads.

Improvement of the aggregate can take many forms – compaction; reinforcement; stabilisation by addition of a binding agent like cement; replacement by an alternative aggregate with better mechanical properties (e.g. as a consequence of better particle shape); blending with some particles of other sizes to provide a denser, more interlocked, aggregate structure; drainage to improve the pore suction or, even, improved confinement by addition of surrounding materials. Because supply of good quality aggregate may be difficult to secure at an economic price - and because additional thickness of aggregate is often, in practice, the default solution of many maintenance engineers responsible for low-volume granular pavements - therefore the presence of poor or marginal quality aggregate in the road construction is a common cause of pavement rutting distress.

2.6. PRACTICAL IMPLICATIONS FOR REMEDIES

When a road exhibits excessive rutting it is common practice to use the material at the site, possibly with additional material brought in from a supply pit, to fill the ruts and, thereby, to re-establish the road. Given that poor quality aggregate is the most common cause, this will not provide an adequate remedy. Merely moving back displaced material into the place from which it was earlier displaced due to its inadequate strength to stay there, is not going to solve the problem. Instead, it will be no better than in its original condition (and probably worse due to particle degradation) meaning that the rutting will rapidly be re-established. Bringing in new material of the same type will, similarly, not provide a remedy. Instead the aggregate must be improved in some way, as described above.

Conversely, if subgrade deformation has been the problem, due to a thin aggregate allowing over-stressing, then the placing of extra aggregate may solve the rutting problem. Nevertheless, earlier damage of the subgrade caused as it rutted, will now be “buried” at the bottom of the road’s construction and can lead to longer term problems – for example allowing water to collect in the pavement structure which cannot be solved by external drainage.

Chapter 3. REVIEW OF DESIGN APPROACH TO AVOID RUTTING

3.1. INTRODUCTION

Over the last 40 years, the procedures adopted by most nations for the design of major roads have moved from an empirical base to fully mechanistic approaches. It was soon realised that empirical procedures were only useful where the same traffic, climate, materials and construction applied. Once extrapolation was required beyond the experience on which the method was built, then the reliability of the procedure decreased substantially. To address this deficiency, analytical procedures were developed. These usually used a simple model of the pavement in which the upper (bound) layers are considered to be in bending and the strains incurred under trafficking are compared with the fatigue limits of the materials being considered.

Because the underlying mechanisms of rutting had either not been much understood, or if understood had not been much considered for use in the design process, the design of pavements for major highways against rutting has, almost until the present, relied on the computation of a very indirect index to assess rutting propensity. This index – the vertical resilient strain value at the top of the subgrade – is assumed to be related to the sum of all the plastic vertical strains in all layers. No mechanistic explanation is provided for this assertion, so the method, as far as rutting is concerned, can be considered to be analytical (analysis is required) but not mechanistic.

In recent years the AASHTO organisation in the US has published a design guide for major pavements which, if used at its most advanced level, does incorporate mechanistic models for plastic strain in the different layers of the pavement so that a fully mechanistic approach is now possible.

The design of low-volume road pavements lags considerably such advances, and perhaps this is not only inevitable but also desirable. The inevitability results from the amount of research and development that has gone into developing the fully mechanistic approach for major pavements compared to that which can be expected for low-volume pavements. However, that this state of affairs is desirable, stems from the complexity, expense and time required to perform the design process. This would hardly be reduced for low volume pavements, yet the support and funding available to design them is extremely limited. Therefore, no fully mechanistic design procedure against rutting has, so far, been developed for low-volume road pavements.

All pavement design procedures dealing with pavements in which the major structural layer is provided by aggregate seek, as their principle goal, to provide a design thickness of the granular base layer. Subsidiary goals may be to provide design thicknesses of sub-base and/or of an asphalt chip-seal layers and to provide sufficient drainage to ensure the continued performance of the pavement even in wet-weather or spring-thaw.

3.2. ALTERNATIVE DESIGN METHODS

The available design methods might be, simply, grouped as follows:

- Minimum requirement methods. These are the simplest of the available approaches. Pavements are built to a set thickness dependent only on the subgrade quality, being independent of anticipated traffic. Aggregate used much meet certain "recipe" requirements (which are not, directly measures of anticipated mechanical performance). Being an empirical approach, this method suffers from the limitations of empirical methods that were described in the previous section. Where traffic loads are small and environmental factors have a major impact on road pavement quality, then this approach has something to commend itself if the empiricism is "tuned" to the same climatic zone. One disadvantage, amongst several, is that the aggregate actually used may be either just adequate or substantially in excess of the minimum requirement, yet the method makes no allowance for this.
- Empirical thickness methods. These methods provide a design thickness of the granular base layer that is dependent on traffic and on subgrade quality. Typically they are based on trafficking studies performed by the US Army Corps of Engineers, perhaps adjusted for local conditions. The US Army trafficked different thicknesses of aggregate that had been placed on different subgrades (all clays but with varying CBR[1] value). Once again, aggregate quality doesn't normally feature in these design approaches. Traffic is typically aggregated using the familiar "fourth power law" although it is well known that this doesn't apply well to low-volume pavements, is based on fatigue-type failure response not that of rutting and, fundamentally, doesn't permit aggregation of different traffic loads.
- Semi-analytical methods. These methods typically implement a version of the resilient subgrade strain criterion to provide a design thickness of the base layer. In such a case a stress analysis must be performed in order that the resilient strain at the top of the subgrade may be computed under a set wheel load. The general unease about using a calculated resilient strain at one place in one material to predict a plastic strain over a range of materials remains. The use of the "fourth power law" to sum the affect of different loads must also be invoked. For practical implementation by local engineers, parameter determination must be simplified (as, indeed, the present document will propose). Despite their limitations and the concerns expressed, these methods are generally considered to be the most advanced, at present.

3.3. ORIGINAL ROADEX PROPOSAL (ROADEX II)

The earlier report tried to develop a fully analytical version, yet simplified for application by the target users – busy regional road engineers without a high level of technical support in-house. It exploited the observations, made by several authors in recent years, that there is an envelope of repeated stresses for which plastic strain doesn't develop in a granular material (so-called Range A) and another, higher, stress field for which plastic strain accumulates but slowly (so called Range B). Therefore, it was decided to adopt a design

[1] California Bearing Ratio

philosophy in which a pavement stress analysis would be performed and granular materials used which had either Range A or Range B behaviour (as desired by the designer) at the calculated stresses. This would allow materials to be selected based on their capacity to perform successfully, rather than on their ability to meet a particular specification. Used “in reverse”, such a method could be used to compute the thickness of an upper pavement layer so as to keep the stress levels beneath the Range A limit values (or the Range B limit values, if preferred) for a particular material occupying a lower layer.

The ROADEXII approach involved two design stages, the stress analysis being used to compute two stresses:

- one at the top of the subgrade – to address the possibility of Mode 2 failure (see Section 2.3) and
- the other in the aggregate layer – to address the possibility of Mode 1 failure (see Section 2.3).

Mode 0 and Mode 3 failures (see Section 2.3) were both discounted as being of concern, the first being self-stopping once adequate compaction has taken place and the second being addressed by particle strength requirements independent of the stress analysis.

The first of the two stresses was to be compared to the failure strength of the subgrade. The thickness of the granular base layer is increased until the stress is sufficiently less than that which would be sufficient to cause static failure. This approach was adopted on the basis that, in most of the pavements observed, rutting is not due to subgrade over-stressing, so an advanced design approach is not warranted. The critical strength was stated in terms of an undrained unconfined compressive stress, $4c_u$ being permitted – which was believed to be an acceptable simplification given that poor subgrades are nearly always cohesive – and the computed deviatoric stress was then compared with the value of $4c_u$.

The second stress was computed at a location in the aggregate where it was expected to be reasonably near the maximum value. This immediately introduces some empiricism as the position of maximum stress cannot be determined except by assessing the stresses over the complete zone of loading introduced by the vehicle. In this case the stress calculated needed to be a ratio involving the deviatoric to normal stress (in effect a kind of mobilised angle of friction which more appropriate for a granular material). The value of this ratio was then compared to the stress locus for monotonic failure in the aggregate layer – another frictional line. Provided the imposed ratio was less that 70% (55% in very wet conditions or where trafficking in spring-thaw conditions is required) then the onset of significant rutting should be avoided.

Although a finite element approach was adopted to compute the stresses when developing the ROADEXII approach, it was readily recognised that such a technique would not be available for local engineers. So, accordingly, a rather large simplification was introduced with Boussinesq stress distributions produced graphically for both deviatoric and all-round stress. These charts were then used to compute whether the stresses in the granular layer were less than that which would cause excessive plastic strain. The advantage of using the Boussinesq charts being that the stiffness of the material isn’t required. For the subgrade a layered elastic analysis by software was needed.

3.4. LIMITATIONS OF ORIGINAL PROPOSAL

There are several limitations with the ROADEXII proposal. Firstly, as mentioned above, there were two main simplifications concerning Mode 2 rutting. A cohesive subgrade with a simple strength parameter was assumed which will not always be the case. Furthermore rutting is assumed to commence when the subgrade stress exceeds a certain percentage of the failure stress value. There is some evidence to support this, but the exact value that is appropriate almost certainly varies with soil type and condition, so setting one value for all situations will introduce some uncertainty which is managed by ensuring a conservative percentage is adopted. In fact, this approach is not thought to be too much of an issue given that, in practice, few pavements fail in this manner.

A much greater issue is the use of the Boussinesq distributions to compute the stress level in the granular layer. Boussinesq theory applies to a semi-infinite half-space – which is NOT a good approximation for a layered pavement system. Thus, the computed stresses are probably significantly in error if computed in this manner. Furthermore, for the subgrade, the need to use layered elastic software would be likely to hinder take-up by many engineers. More significantly, the software would need stiffness values for aggregate and subgrade which are not readily available and would need relatively sophisticated tests to determine their value. This need would also be likely to hinder take-up of the improved design approach.

3.5. POSSIBLE IMPROVEMENTS TO ORIGINAL PROPOSAL

In addition to addressing the limitations just mentioned, the original proposal was not drafted with great consideration of wheel arrangements or tyre pressures. Therefore the procedure would be improved if these options could be easily addressed. Furthermore, assessment in this way would help to show the least damaging means of trafficking low-volume granular pavements. Variable tyre pressure systems (known as Tyre Pressure Control Systems – TPCS) are becoming more common (see separate report "Tyre Pressure Control on Timber Haulage Vehicles") and a design method that allowed for this aspect could be a valuable development of the design approach, enabling users and authorities to have a tangible means of deducing the likely benefit to the pavement of the TPCS approach.

If the outputs can be provided in tabular or graphical form then the benefits of the new approach will be likely to be much more usable by the target stakeholders.

Chapter 4. ASSESSING & APPLYING ANALYTICAL TOOLS

4.1. INTRODUCTION

As will be evident from the previous section, a key element of a fundamentally acceptable design approach is the acceptable computation of the stress strain state in the pavement. Unless very advanced (and for the present purposes, inappropriate) computational techniques are to be employed, then a full calculation of incrementally developed plastic strain is impossible. Even if such a method were to be employed, and the challenging demands of obtaining representative material parameter values could be achieved, the current state of international research doesn't give one any confidence that the results would be valid!

Therefore, in this section the computational tools considered in this study are reviewed and the basis for final selection is made.

4.2. AVAILABLE ANALYTICAL TOOLS

4.2.1 ELSYM

ELYSM is a linear elastic layered analysis program. As many as 5 layers can be analysed, each with a fixed resilient modulus and Poisson's ratio. The upper surface of the structure may be loaded with one or more circular loaded areas (each may have a different pressure on it), but each may only be loaded vertically. In common with all elastic layered programs, each layer is treated like a plate in bending. Frequently, under the influence of the load, this means that the upper surface is subjected to horizontal compression (a genuine effect) whilst the bottom of the layer is subjected to horizontal tension. If the material were a bound material, like asphalt, this could be a genuine effect, too. However, soils and granular materials can carry little, if any, tension. This is a serious limitation.

As the granular layer has a fixed stiffness (provided in the form of the constant resilient modulus value) the user must be very careful to select the correct values of stiffness for each layer. As granular material exhibits a stress-dependent stiffness, such selection needs to take into account the stress that will be applied via the traffic loading .... but this is, of course, dependent on the load spreading that the program is designed to compute. Thus selection of the appropriate stiffness value requires considerable experience and, even then, is liable to be mis-estimated.

4.2.2 KENLAYER

KENLAYER is a broadly similar program to ELSYM, but with the added advantage that a non-linear material model may be included. This allows the stiffness of the layers to be adjusted by the program in line with the stress applied. This feature allows the material to develop the stiffness appropriate for the actual application of load that it will experience. Despite this improvement the computational framework still introduces the limitation that each layer has the same stiffness properties throughout its width. In fact the real pavement will have a radially changing stiffness as the effect of the loaded area is ‘felt’ less by the material at a large distance from the loaded area.

KENLAYER also suffers from the possibility of non-representative tensions being recorded at the base of stiffer layers that lie on top of softer layers (especially a problem at the bottom of a granular base course overlying a soft subgrade). However, the problem is not so great as with ELSYM as the stress-dependent stiffness provided by KENLAYER will mean that the material’s stiffness drops where tension is computed, thereby allowing strain and some consequent re-distribution of the tensile stress.

4.2.3 FENLAP

FENLAP is a finite element program specifically designed to incorporate material non-linearity. A variety of non-linear material stiffness models are available within the program. The finite element approach overcomes the unnecessary limitation that the stiffness is constant with radius as the model now comprises finite blocks rather than layers. Finite element analyses do not automatically overcome the problem of tension at the bottom of stiffer layers, but FENLAP implements a tension cut-off approach so this problem is overcome.

4.2.4 Other Finite Element Codes

There are a large number of other codes available, e.g. ABAQUS, some of which have stress dependent granular material models incorporated into them, and most would implement tension cut-off options. However, the magnitude of the complexity of many of these programs means that the time to become familiar with their operation is considerable. Therefore they were not considered further for the current study.

4.2.5 Comparison & Selection of Tools

In principle, FENLAP was the preferred solution of those previously described. However, in a preliminary use of the program it was found to be difficult to obtain convergence. The program solves each problem iteratively, as the stiffness of each element has to be recalculated after the initial calculation of stress in the element. This is repeated until a harmonious set of stiffnesses and stress is obtained. In the case of FENLAP this is further complicated as the loading is not put on in one increment, but in a series of sub-loading steps which, thereby allows a more accurate estimate of resilient strains to be obtained.

For reasons that are not immediately apparent, in the initial computations performed, convergence to a harmonious set of stresses and stiffnesses was not achieved. Experience suggests that this is a consequence of a very high stiffness gradient at the surface for unsealed or chip-sealed pavements. However, the cause was not investigated in any detail in this study. Therefore, it was decided to fall back upon the KENLAYER code which, despite the limitations mentioned above, was capable of giving timely answers.

4.3. VARIABLES STUDIED

Having selected a computational tool, it was now necessary to provide materials data and loading arrangements that would cover the range of materials and trafficking situations to be encountered by users. In this section the materials and loading options selected for investigation are briefly introduced.

4.3.1 Materials

Three materials were selected from a database of existing materials held at the University of Nottingham. They are listed in Table 1. The three materials are selected as covering the range of likely behaviours that could be expected.

Table 1 ***Properties of Materials analysed in this study***

Code	Name	Bulk unit weight	Mohr-Coulomb Failure parameters		Shakedown range boundary A-B		Shakedown range boundary B-C		K-Θ modulus constants	
		ρ_b (kN/m³)	c (kPa)	φ (°)	d (kPa)	β (°)	d (kPa)	β (°)	k_1 (GPa)	k_2
NIG	NI Good	19	74	46	10	58	59	65	71.51	0,29
NIP	NI Poor	21	27	46	65	39	114	56	103.46	0,23
CAF	CAPTIF 2	22.8	0	61	0	45	0	62	3.2	0,77

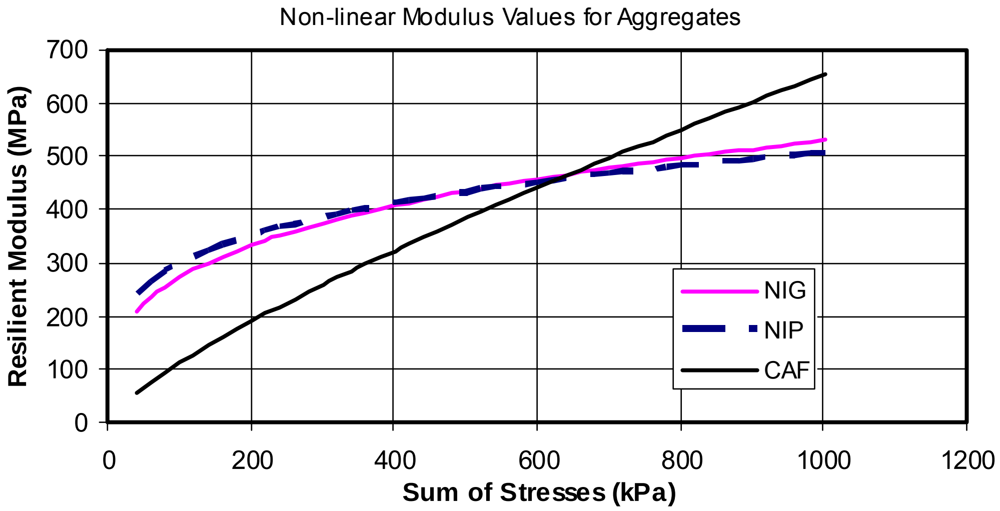

Figure 1 Resilient Modulus as a function of applied stress for three selected granular materials

The NI Poor (NIP) is a material used for a trial road in Northern Ireland, known to be of poor quality. It has relatively low cohesion (c) and a poor angle of friction (*φ*). It has moderate stiffness characteristics but a low non-linearity such that stiffness does not increase much in highly stressed areas (see Figure 1). NI Good (NIG) is a granular material of similar stiffness (see Figure 1) used in the same Northern Ireland trials. Although it has a similar stiffness its strength is higher than that of the NIP material and its “Range A” shakedown (see Section 3.3) stress envelope is substantially larger (see Figure 2a & b).

CAF, on the other hand, is a much cleaner aggregate with no cohesion, but a very high frictional behaviour (Table 1) and a stiffness that rapidly increases with additional imposed stress (see Figure 1). As shown in Figure 2c, its shakedown ranges are similar to those of NIP, but it has a much greater ultimate strength. The impact of a higher stiffness under traffic loading is likely to mean that the load spreading in the top of aggregate CAF will be far superior to that of NIP, therefore leading to a thinner design for the same performance. In the case of NIG a similar thickness of layer to that of NIP would be expected to protect the subgrade, but the in-layer rutting would be far lower.

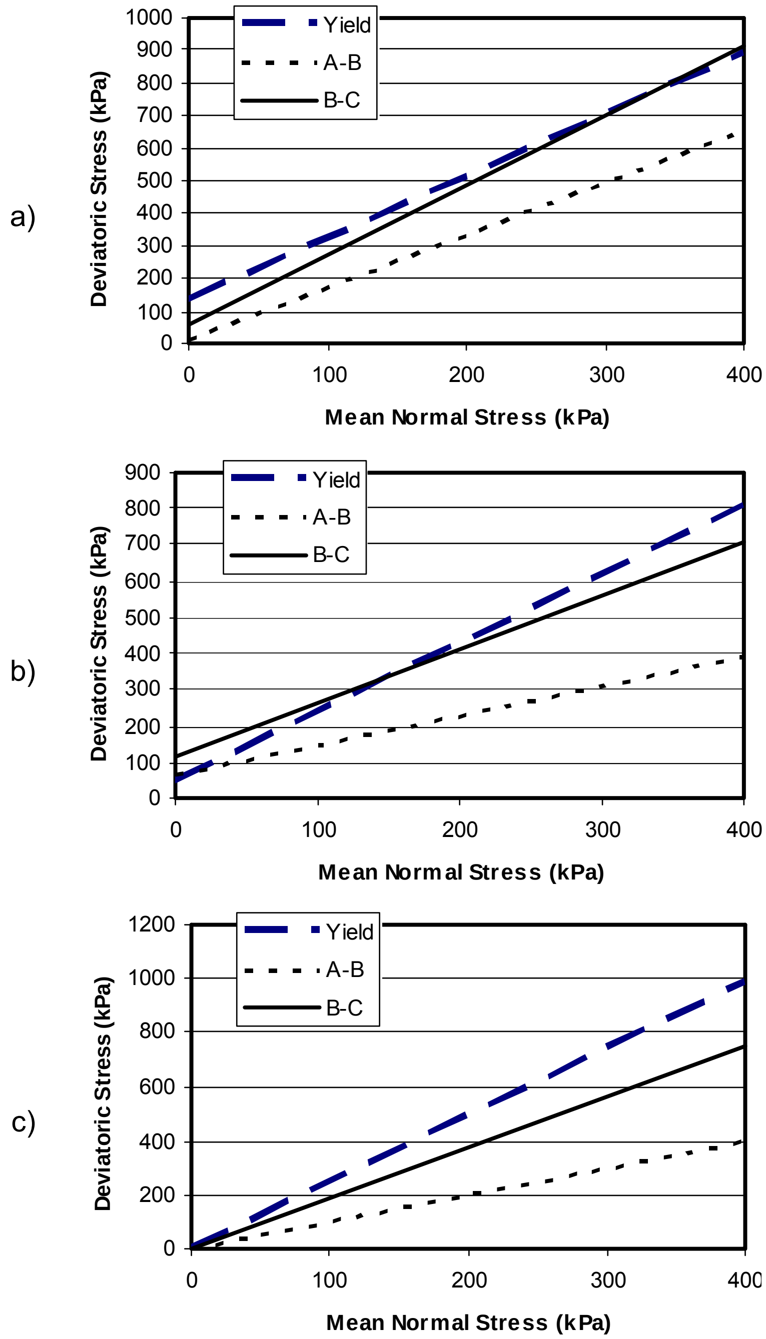

Figure 2 Stress loci for monotonic failure and shakedown regions. a) NIP, b) NIG, c) CAF

In fact, when the computations were performed (see Section 4.4) the computed stresses were found to be largely independent of granular material type, indicating that these stiffness non-linearities lead to very similar computed stiffnesses for the same loading and layer sequences.

4.3.2 Loading Arrangements

Trafficking of low-volume pavements in the Northern Periphery is, typically, either by trucks equipped with twin tyres on the end of each axle (excepting the steering axle) or by trucks equipped with “super-single” tyres. Measurements of the print width were taken at a site in Scotland for both types of tyres (Table 2) which suggested typical contact areas of 21cm wide and 29cm wide, respectively, as shown. The reduction on tread width compared to the nominal width is because the nominal width is wall-to-wall at normal inflation pressure, whereas the print width is due to the tread pattern with which the tyre is manufactured. For nominally identical sized tyres, the contact width does change a little (perhaps ±1cm). The length of print varies according to the tyre’s inflation pressure and according to the resilient depression. If the pressure in the tyre, and the load applied, are taken to be the only controlling factors on contact area, then the contact areas are as shown in Table 3 and the deduced length of the real tyre print is as shown in Table 2.

For computational purposes using KENLAYER all the loads have to be treated as circular. For this reason the radii giving an equal, but circular, area to those deduced in Table 2 are given in Table 3 and shown pictorially in Figure 3. Based on the measurements made in Scotland, the centre-centre distance between twin tyres on one axle is approximately 34.5 cm. Thus, for computational purposes dual tyres have been assumed to be two circular loaded areas positioned adjacent to each other as shown in Figure 3.

Table 2 Tyre arrangements measured and considered

Tyre type	Standard single	Super single
Typical tyre designation	295/80R22.5	385/65R22.5
Nominal tyre width (wall-to-wall) (cm)	295	385
Measured tread width (mm)	210	290
Estimated length (mm) of tread at 800kPa tyre pressure and 45kN half-axle load	134*	194

** assumes that the standard tyre is one of a twin, so each one only carries 22.5 kN*

Table 3 Equivalent tyre contact areas and radii for single tyres

Wheel Load (kN)	Tyre Pressure (kPa)	Area (m^2)	Radius (cm)
45	800	0.05625	13.38
45	400	0.1125	18.92
22.5	800	0.028125	9.46
22.5	400	0.05625	13.38

In this figure and these tables, two tyre pressures are used, 800 and 400kPa. These are towards the maximum and minimum tyre pressures that are used. The high value is not quite as high as normal maximum tyre pressures (usually between 800 and 900kPa according to manufacturer's data), nor is it as low as the lowest pressures used in on-board Tyre Pressure Control Systems (TPCS) when pressures as low as 240kPa have been reported. However, they were thought to give a value close to "normal" use and also show a much lower pressure so that the impact of lower tyre pressures can be clearly seen.

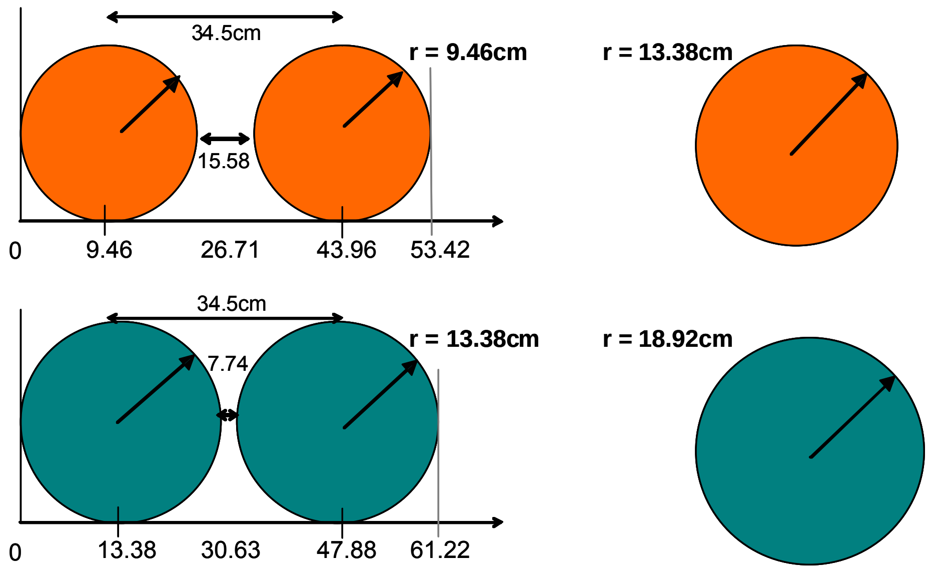

Figure 3 Equivalent, circular loaded wheel areas

Dimensions of loaded areas used to simulate dual and super-single tyres.
Top at 400kPa, bottom at 800 kPa

4.4. PRESENTATION OF COMPUTATIONS

4.4.1 Summary Results

Figure 4 is one of 180 plots obtained by computing the stresses in the various pavements considered under the various loads considered. The number of plots derives from the number of loading and material possibilities considered:

- 5 ratios of thickness of granular layer to radius of load,
- Whether super single tyres or dual tyres are to be used,
- Use of tyres at 400kPa (low inflation pressure) or 800kPa (high inflation pressure),
- 3 aggregate types (as described above), and
- 3 ratios of granular layer stiffness to subgrade stiffness.

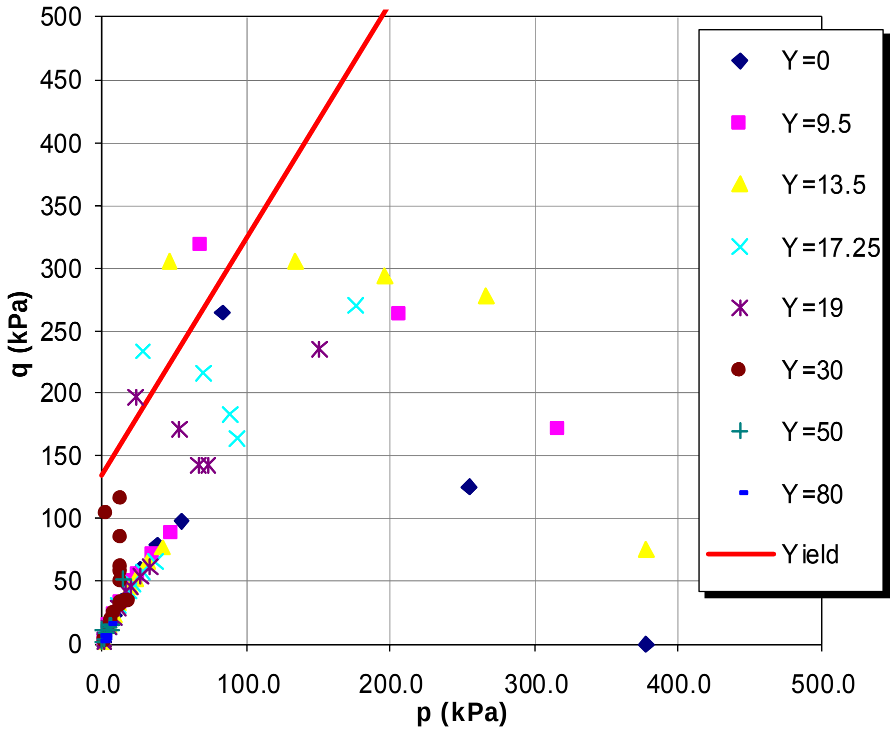

Figure 4 Typical plot of computed stresses in pavement

As can be seen in Figure 4, the stresses were calculated in terms of the mean normal stress, *p*, and the deviatoric stress, *q*. These stress invariants were evaluated at a large number of points within the pavement and plotted on the figures like Figure 4. The results show that there are some points (stress states) well beyond static failure, perhaps even in tensile stress state. Clearly these are impossible, and probably result as a consequence of the use of the layered elastic method that is provided by KENLAYER. It is known from other work that finite element computations with appropriate tension cut-off models can result in few or no stresses in this zone. The remaining stress points are scattered over the *p-q* space, but there is usually a fairly well defined locus of maximum stresses through which a line can be plotted as shown in Figure 4.

4.4.2 The summary stress variable, *S*

From the ROADEXII analysis it is known that, if the stress states in the pavement are kept a long way from failure than no rutting in the granular layer will occur, but that if the stresses approach the static failure envelope, then the speed of development of rutting in the granular layer increases. Therefore, to simplify the use of the computations, and to make them readily usable, each of the 180 plots was simplified to give one number:

- A stress variable, *S*, which is the distance along a line commencing at *p*=250kPa, *q*=0kPa and heading towards *p*=0kPa, *q*=250kPa but stopping when it reaches the locus of genuine maximum stress states found in the aggregate layer (see Figure 5 where S_{ss} is the value of *S* for the case of loading by a super-single tyre and S_{dt} is the value of *S* for the case of loading by a dual tyre).

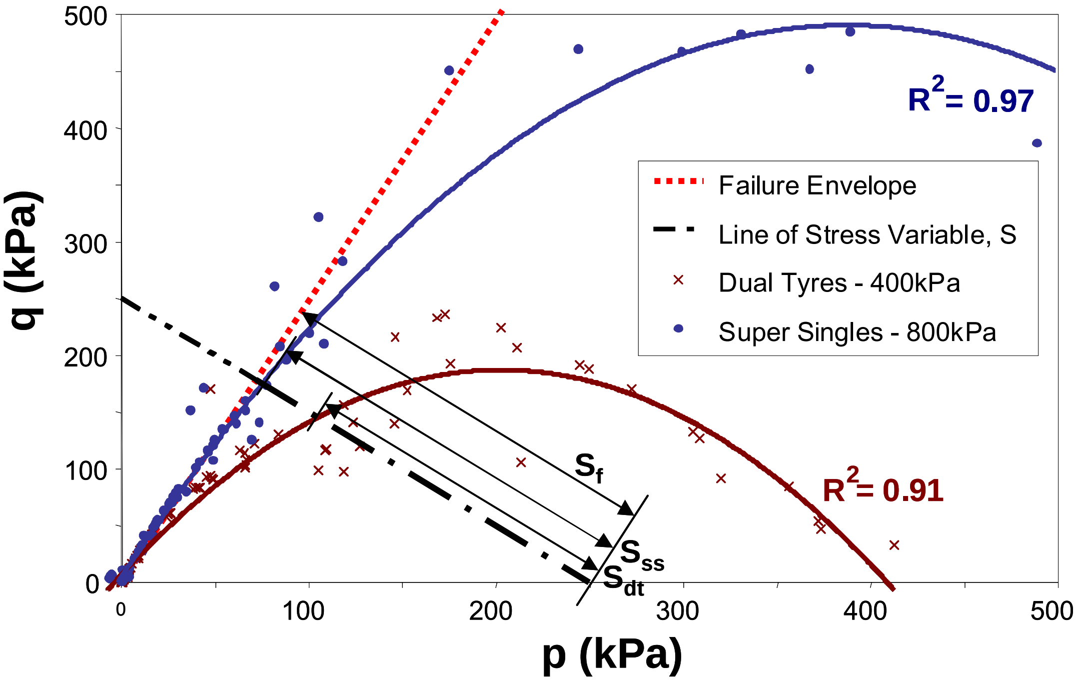

Figure 5 Typical plot of stress variable S on a stress diagram

The "250-250" line was chosen as it intersects most of the loci of maximum stress points when those loci are at their closest to the failure envelope. A different line could have been selected, but this line seemed a reasonably reliable one for the purpose of assessing "proximity to failure" (see below). Other features of Figure 5 will also be discussed below.

4.5. APPLICATION TO ROADEX DESIGN APPROACH

4.5.1 Design against rutting in the granular layer

In ROADEXII, a two-stage design approach was introduced, but the computations of stress in order to achieve the necessary computations were rather weak. The obvious ways of achieving better computations of stress (computational programs, for example) would have made the method of low attractiveness to users. But the approach just described helps to overcome this difficulty. The value of S may be compared with the distance along the line from p=250kPa, q=0kPa until the static failure envelope is reached (a distance to be labelled S_f, as shown in Figure 5), such that $S << S_f$ indicates relatively stable, slowly rutting conditions in the aggregate layer and $S \approx S_f$ indicates rapid rutting failure in the granular material.

Of course, there are some practical issues in obtaining values of S. It is not possible in this report to go through each, individually, but the important ones are:

- The p,q stresses calculated at individual points in the pavement sometimes give values that are not credible. This can be for a number of reasons, principal of which is the fact that a layered elastic model will often predict tensile stresses at the bottom of a stiff layer that overlies a softer layer. Of course, in a granular layer such a stress state is inadmissible, as granular material will separate between particles rather than carry tension. Stress states that are calculated as being in tension are, therefore, discounted in the generation of *S* values from the raw data. In Figure 5 these will be stress points that plot to the left of a line that rises at an angle of 71.5° from the origin.

- A further inadmissible stress state will be when static failure occurs – any stress state above the failure envelope in Figure 5. No limiting stress could be added in the computations so, at several points, stresses greater than tolerable were calculated. In reality the stiffness would change and plastic strains take place at these points so that the stresses are redistributed. In fact, for the aggregates considered in this study, most of the inadmissible stress states arise from this limitation rather than from the one mentioned above. For the purposes of this analysis, these stresses are taken to be at the failure envelope and not above it.
- In order to obtain a locus of admissible, maximum stress states, a "best fit" line is placed through the data. This has been achieved by using a hyperbolic function and discounting the low-stressed points (as we are only interested in those for which, by repeated loading, might deform causing rutting – and these are those towards the maximum). This selection of points is, of necessity, somewhat subjective. In Figure 5 the quality of fit is indicated by the coefficients of regression.

By reducing each analysis to (in effect) a single number, it is possible to tabulate the values of *S* or to plot them in a few summary graphs as seen in Table 4. As mentioned at the end of Section 4.3.1, the fact that the stiffnesses using all three materials become very similar when using the KENLAYER model, means that the values of *S* are also very similar for pavement with granular layers made from any of the materials. Therefore, the effective number of values of *S* reduces to 60 and it is these that are tabulated below.

Table 4 Summary of Values of Stress Variable, S, for all NIG analyses

Agg. Thick. / Load Radius Ratio	Aggregate Thickness	Stiffness Ratio (Ebas/Esub)	Tyre Pressure	Tyre Arrangement	S
	(cm)		(kPa)		(kPa)
1.0	13.5	2	400	Dual Tyres	**207.1**
1.3	17.0	2	400	Dual Tyres	**205.7**
1.7	23.0	2	400	Dual Tyres	**214.9**
2.5	33.8	2	400	Dual Tyres	**214.5**
3.5	47.3	2	400	Dual Tyres	**215.1**
1.0	13.5	4	400	Dual Tyres	**221.3**
1.3	17.0	4	400	Dual Tyres	**212.3**
1.7	23.0	4	400	Dual Tyres	**217.9**
2.5	33.8	4	400	Dual Tyres	**209.2**
3.5	47.3	4	400	Dual Tyres	**208.7**
1.0	13.5	8	400	Dual Tyres	**224.5**
1.3	17.0	8	400	Dual Tyres	**217.7**
1.7	23.0	8	400	Dual Tyres	**214.0**
2.5	33.8	8	400	Dual Tyres	**209.8**

3.5	47.3	8	400	Dual Tyres	**204.1**
1.0	9.5	2	800	Dual Tyres	**231.0**
1.3	12.0	2	800	Dual Tyres	**232.1**
1.7	16.0	2	800	Dual Tyres	**238.2**
2.5	24.0	2	800	Dual Tyres	**226.7**
3.5	33.5	2	800	Dual Tyres	**229.9**
1.0	9.5	4	800	Dual Tyres	**229.0**
1.3	12.0	4	800	Dual Tyres	**229.9**
1.7	16.0	4	800	Dual Tyres	**227.2**
2.5	24.0	4	800	Dual Tyres	**227.1**
3.5	33.5	4	800	Dual Tyres	**225.6**
1.0	9.5	8	800	Dual Tyres	**229.4**
1.3	12.0	8	800	Dual Tyres	**228.9**
1.7	16.0	8	800	Dual Tyres	**223.3**
2.5	24.0	8	800	Dual Tyres	**226.6**
3.5	33.5	8	800	Dual Tyres	**222.6**
1.0	19.0	2	400	Super Singles	**245.1**
1.3	24.0	2	400	Super Singles	**245.0**
1.7	32.0	2	400	Super Singles	**242.4**
2.5	48.0	2	400	Super Singles	**245.2**
3.5	66.5	2	400	Super Singles	**245.1**
1.0	19.0	4	400	Super Singles	**244.2**
1.3	24.0	4	400	Super Singles	**242.9**
1.7	32.0	4	400	Super Singles	**237.7**
2.5	48.0	4	400	Super Singles	**239.6**
3.5	66.5	4	400	Super Singles	**240.7**
1.0	19.0	8	400	Super Singles	**242.0**
1.3	24.0	8	400	Super Singles	**240.6**
1.7	32.0	8	400	Super Singles	**236.9**
2.5	48.0	8	400	Super Singles	**233.4**
3.5	66.5	8	400	Super Singles	**237.1**
1.0	13.5	2	800	Super Singles	**250.5**
1.3	17.0	2	800	Super Singles	**249.5**
1.7	23.0	2	800	Super Singles	**248.7**
2.5	33.8	2	800	Super Singles	**248.1**
3.5	47.3	2	800	Super Singles	**249.8**
1.0	13.5	4	800	Super Singles	**245.7**
1.3	17.0	4	800	Super Singles	**246.0**
1.7	23.0	4	800	Super Singles	**241.8**
2.5	33.8	4	800	Super Singles	**242.2**
3.5	47.3	4	800	Super Singles	**246.1**
1.0	13.5	8	800	Super Singles	**238.1**
1.3	17.0	8	800	Super Singles	**239.8**
1.7	23.0	8	800	Super Singles	**238.2**
2.5	33.8	8	800	Super Singles	**237.2**
3.5	47.3	8	800	Super Singles	**239.8**

NB: The value of S_f for these analyses is 297.1 kPa

The data for the shaded line is the data plotted in Figure 6.

The data in Table 4 have also been plotted against the granular layer thickness (expressed as a proportion of the load radius) and of the nominal aggregate stiffness (expressed as a multiplication of the subgrade stiffness). An example is given as Figure 6. Four such plots are needed, one for dual tyres, one for "super-single" tyres and then the same two plots again but for low tyre pressures. The set is provided in the Annex.

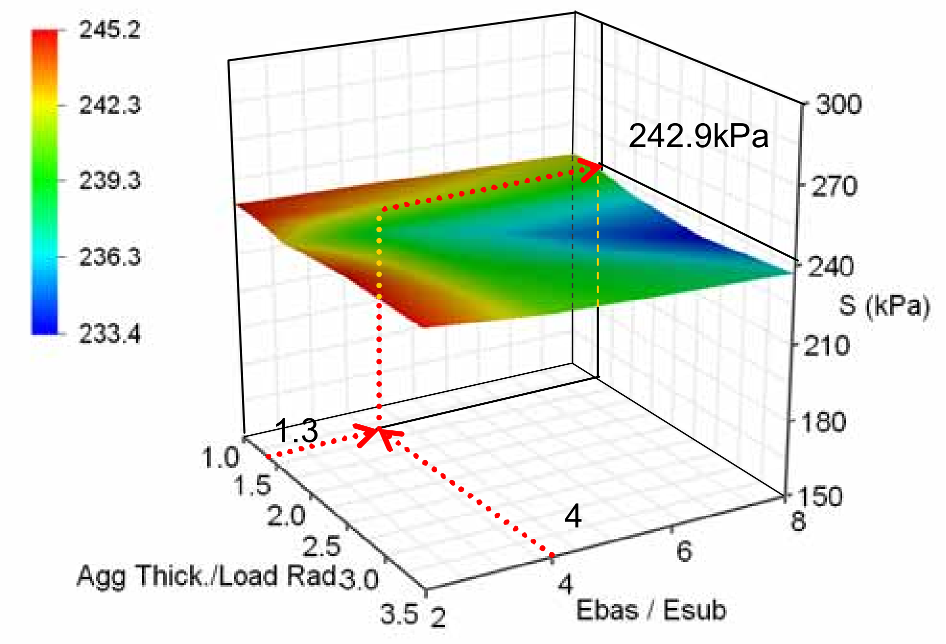

Figure 6 Value of stress variable S for loading by Super-Single tyre at 400kPa

Thus, knowing the tyre arrangements and pressures allows the appropriate plot to be selected and knowing the aggregate thickness and the layer stiffnesses allows the value of *S* to be obtained. This may then be compared with S_f which can be computed directly from a knowledge of the failure characteristics of the granular material being considered. The Annex gives an example. If the friction and cohesion characteristics of the granular layer are known (ϕ' and c'), then the value of S_f can be calculated using the equation:

$$S_f = \sqrt{2}\,\frac{250M - a'}{1+M} \quad \text{where} \quad M = \frac{6\sin\phi'}{3-\sin\phi'} \quad \text{and} \quad a' = c'\cos\phi'$$

otherwise it will need to be estimated from experience or from suggested values. Although the value of S_f given in these examples is 297.1 kPa, the actual value will be needed for the intended application.

In ROADEXII, the permissible stress to prevent rutting was set at 70 and 50-55% of failure depending on whether the conditions being considered were a) "normal" or b) very wet or thawing. Because the method of computing stresses has changed from ROADEXII and because of the use of a standard "250-250" line, we estimate that, for the present purposes, the following permissible stress limits should be set as equivalent to those introduced in ROADEXII:

- $S \leq 0.9 \times S_f$ to prevent rutting in the granular layer in normal conditions, and
- $S \leq 0.75 \times S_f$ to prevent rutting in the granular layer in wet or thawing conditions.

4.5.2 Design against rutting in the subgrade

By this means design of the granular layer against rutting in its own thickness is achieved. The second stage of design, as introduced in ROADEXII, is to design the thickness of the granular layer to ensure that rutting doesn't take place in the subgrade. In ROADEXII the criterion that the vertical stress on the top of the subgrade should be ≤ 2 × the Ultimate Compressive stress. This is the same as saying that the deviatoric stress on the top of the subgrade should be ≤ 4 × the undrained strength of the subgrade. It is not proposed to change this definition, but a new way of computing the stress is proposed. Once again, the stress computations have been graphed, this time into two key graphs one for dual tyres and one for "super-singles". The "super-single" tyre version is presented as Figure 7. A similar chart is available for dual tyres. Both are presented in the Annex.

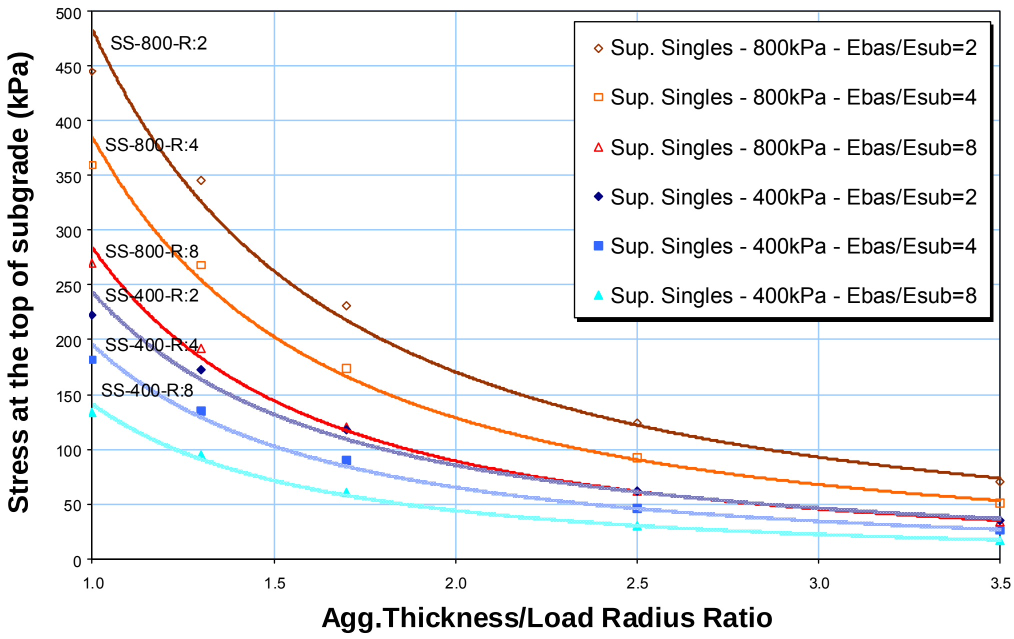

Figure 7 Vertical stress at the top of the subgrade as a function of loading and aggregate

Assembling an overall design approach

Figure 8. The Stage 1 chart leads the user to select an aggregate and the Stage 2 chart allows the user to compute the thickness of aggregate required to ensure no rutting in the subgrade. In order to compute the stresses in Stage 1 it is necessary to estimate the design thickness, so Stage 2, in which the thickness is designed, may undermine this assumption made in computing Stage 2. Hence it is necessary to use the two stages repeatedly until the results and the assumptions broadly match. In fact, one recursion is probably all that is necessary to achieve a workable solution. The design flow charts are on the next 2 pages.

4.5.3 Worked Example

The approach has been followed in the Annex to produce a worked example. The stages of the design process are followed from an unsuccessful preliminary design to a satisfactory final arrangement.

STAGE 1 – Design aggregate thickness to protect subgrade

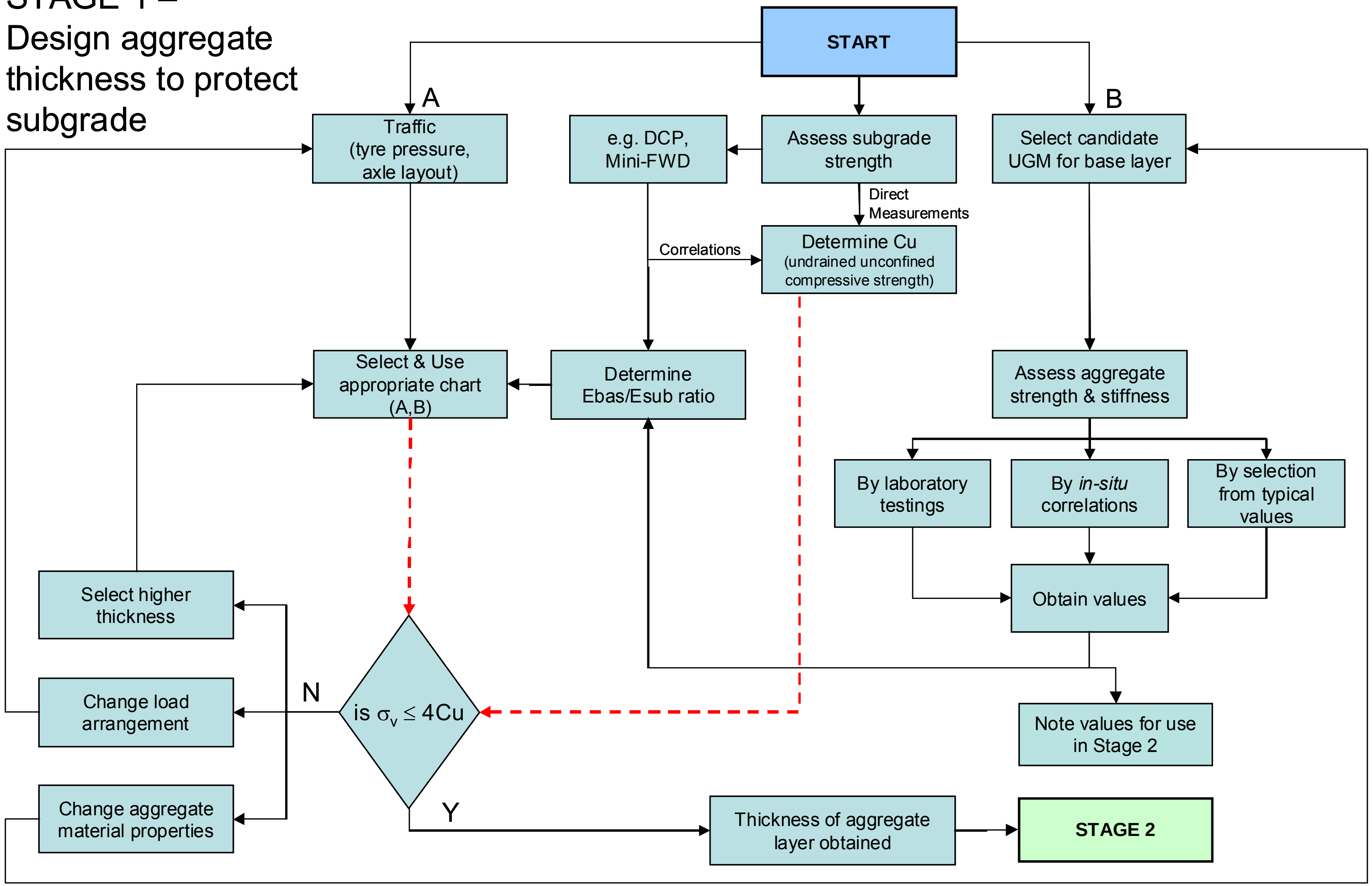

Figure 8a Stage 1 of flow chart to permit design

STAGE 2
Design aggregate quality to prevent rutting within aggregate

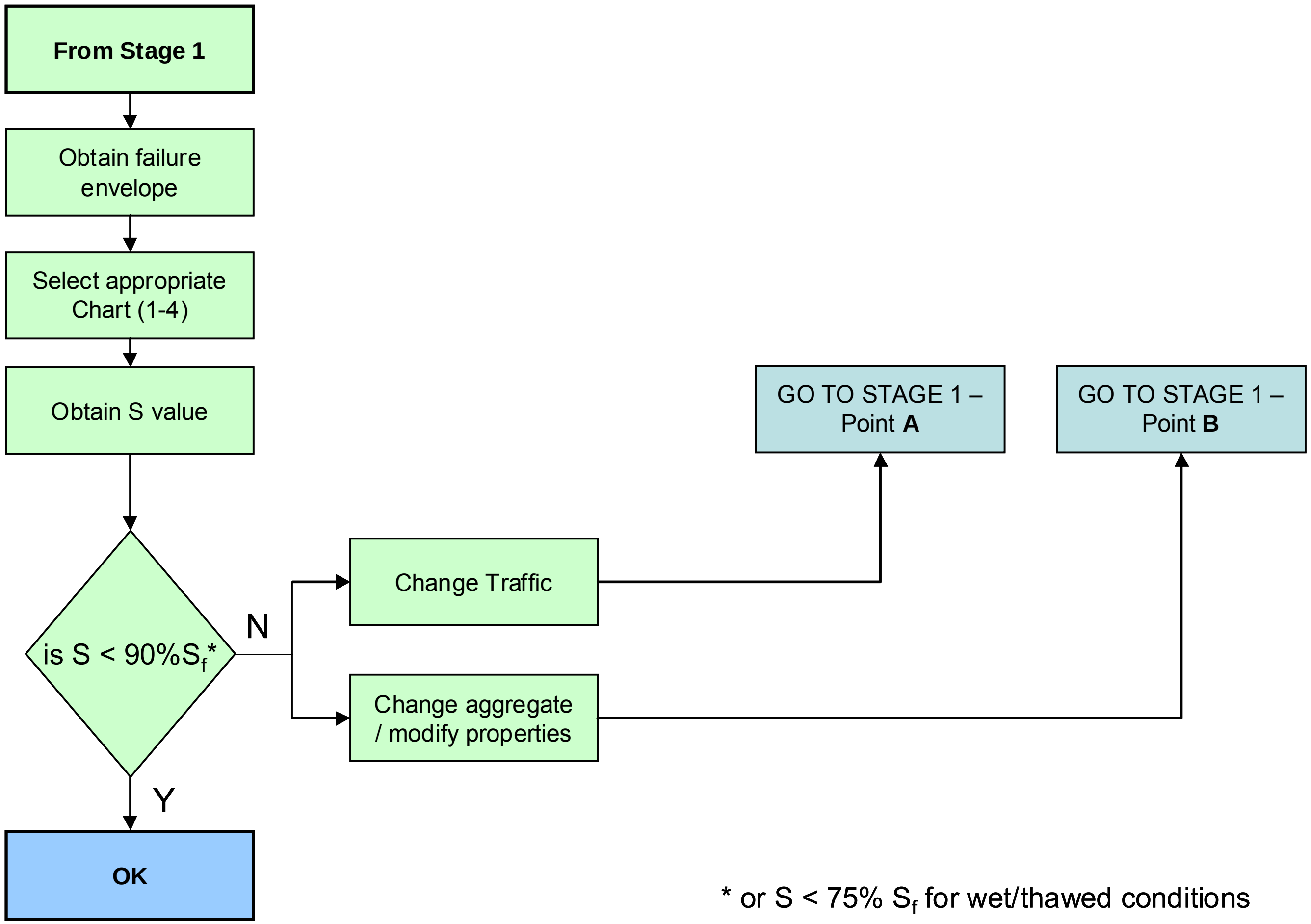

Figure 8b Stage 2 of flow chart to permit design

Chapter 5. OBSERVATIONS

5.1. INFLUENCE OF LOW TYRE PRESSURES

From the table of results (Table 4) and the plotted versions (see Annex) it is apparent that low tyre pressures in dual tyred systems are particularly effective at reducing the stress conditions imposed on the aggregate, but that the benefit of lower pressures is less pronounced for “super-single” tyre systems. Indeed, for stiff aggregates low tyre pressures are marginally *more* damaging in “super-single” configurations (this may just be a computational affect and is barely significant in any event). The benefit is most noticeable for low stiffness aggregates and especially for dual tyre arrangements.

5.2. INFLUENCE OF TYRE ARRANGEMENTS

From the table of results (Table 4) and the plotted versions (see Annex) it is apparent that “super-single” tyres always produce higher stresses, more likely to generate rutting (or stresses that will allow rutting to develop faster) than is the case for dual tyred loading. The difference is most pronounced for low stiffness aggregates (when load spreading is less and the stresses, therefore, more concentrated under the tyre) when the dual tyre provides significantly better load spreading itself.

5.3. INFLUENCE OF AGGREGATE

From the table of results (Table 4) and the plotted versions (see Annex) it is apparent that, except for the softest aggregates, greater aggregate thickness reduces the maximum stress experienced in the aggregate. However, this is not a very strong effect, and a change in tyre pressures or wheel arrangements is more likely to deliver a significant change in stress experienced and, hence, the likelihood of rutting (or its magnitude). The effect of changing aggregate stiffness, alone, on stress condition (and, thus, rutting) is mixed. No strong trend shows up and, in any event, the effect is rather insignificant.

5.4. COMPARISON OF DESIGN APPROACH WITH IN-SITU OBSERVATIONS

5.4.1 HVS TESTS PERFORMED AT VTT, FINLAND

Rutting of the aggregate material

Direct verification of the ROADEX analytical design approach turned out to be very difficult as the amount of well-documented experimental data available in literature proved extremely limited. The main reason for this is fairly obvious. Instrumented test sections are very seldom built using structures that so weak that they as to be damaged under a small number of load repetitions. One source of data that was close to the main focus of the ROADEX project were some of the results from the test series performed using the Heavy Vehicle Simulator (HVS) at the Technical Research Center of Finland (VTT) in the beginning of this decade. Even in these tests, however, the structures had some important differences from the typical very low volume / forest road type of structures considered by ROADEX:

- the structures were covered with a layer of 40 to 50 mm of asphalt concrete
- the total thickness of the aggregate layers was typically of the order of 500 mm
- in some of the tests the subgrade material was sand which has much better capacity to resist rutting than the most problematic fine grained subgrade materials.

The VTT tests typically involved some tens of thousands of wheel passages, resulting in rut depths of the order of 50 mm on top of the structure, which is of course much higher number than the typical number of passes on a forest road over a reasonable period of time.

The HVS test results reported by Korkiala-Tanttu et al. in 2002 are an example of test results that provide at least qualitative support to the ROADEX design approach. This test structure consisted of a 50 mm layer of asphalt concrete, a 200 mm layer of crushed rock unbound base course, a 250 mm layer of crushed gravel sub-base course and a saturated sand subgrade (Figure 9). The structure was loaded with a set of dual tyres with total loads of 70 kN and 50 kN, and tyre inflation pressures of 850 kPa and 700 kPa, respectively. The Mohr-Coulomb failure parameters for each of the materials, determined by means of multi-stage monotonous triaxial tests and the respective values of S_f, are shown in Table 5. In addition, the values of permanent axial strain after 70 000 load repetitions in each of the layers are also given in Table 5.

Table 5 Mohr-Coulomb failure parameters and permanent vertical strains in different layers of the HVS test structure after 70000 load repetitions (UP = upper part of the layer, LP = lower part of the layer).

Material	Mohr-Coulomb failure parameters		Parameter S_f	Load/tyre pressure	Permanent strain
	c (kPa)	φ (º)	kPa	kN/kPa	%
Crushed rock	43.0	43.1	267.4	70/850	UP: 4.2 LP: 3.7
				50/700	UP: 1.1 LP: 0.9
Crushed gravel	35.6	44.7	261.9	70/850	UP: 5.0 LP: 5.6
				50/700	UP: 1.5 LP: 2.2
Sand	12.9	35.5	223.8	70/850	UP: 8.3
				50/700	UP: 5.9

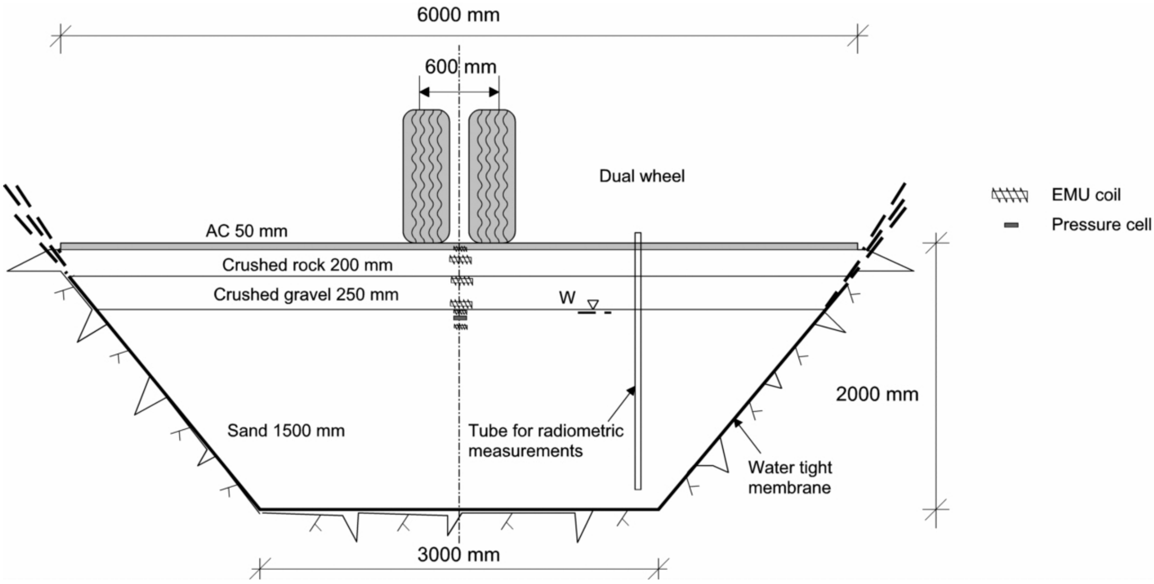

Figure 9 HVS test arrangement at VTT.

Because the ratio between the combined thickness of all of the structural layers above the sandy subgrade and the radius of wheel contact area (about 4.4 for the 70 kN load and about 4.7 for the 50 kN load) is beyond the ranges considered in the ROADEX design approach (where we have a maximum value of 3.5), a meaningful rutting analysis can only be made for the base course layer. Even in this case however, a problem is that the stiffness ratio Ebas/Esub between the crushed rock and crushed gravel is likely to be fairly low, possibly lower than the minimum value of 2.0 shown in the design charts. Keeping in mind the highly stress-dependent stiffness in both the base and sub-base course materials and the rapidly decreasing hydrostatic stress level alongside with increasing distance from the road surface on one hand, and the markedly low sensitivity of S on the Ebas/Esub ratio (see e.g. Figure 6) on the other hand, it is still quite reasonable to make the analysis for the base course using a Ebas/Esub ratio of 2.0.

By interpolating to the inflation pressure of 700 kPa for the 50 kN load, and slightly extrapolating to the inflation pressure of 850 kPa for the 70 kN load, we obtain S values of 225.3 kPa and 233.6 kPa, respectively. Comparing these figures to the S_f value 267.4 kPa of the crushed rock aggregate, the respective per cent values are found to be 84.3 % and 87.4 %. Even though the number of wheel passages required to reach the indicated values of permanent strain is as high as 70000, it is obvious that as the S/S_f ratio is approaching 90 %, the accumulation rate of rutting in the base course layer is rapidly increasing. Therefore, it seems inevitable that at an S/S_f ratio of 90 % a much lower number of load repetitions would be needed to create a marked amount of rutting into the structure and, consequently, this can be considered to support the assumption of the critical S/S_f ratio of 90 % made in the ROADEX design approach.

As already stated above, a similar analysis for the sub-base and subgrade materials is not possible at this time as the aggregate rutting analysis of the ROADEX design approach can only be applied to the topmost unbound layer of the structure. The increasing values of permanent axial strain indicated in table 5 can still be qualitatively explained by the lower values of S_f they have in comparison to the crushed rock base course material.

Stresses at the top of subgrade

In the test arrangement in Figure 9, earth pressure cells were installed at the top of the subgrade sand at a distance of 500 mm from the top the structure. Because the corresponding aggregate thickness/contact radius ratio is now > 4.0, the charts for computing stresses applied to the subgrade surface given in the appendix (page 46) cannot be directly applied. However, following the trend of the curve given for dual tyres, inflation pressure 800 kPa and Ebas/Esub ratio of 2 (DT-800-R:2), we can see that the actually recorded values of approximately 120 kPa for the 70kN/850 kPa load arrangement, and approximately 80 kPa for the 50 kN/700 kPa load arrangement, are slightly higher than the predicted ones but the prediction can still be considered as fairly realistic.

5.4.2 RINGOUR – "RUTT" TRIALS

The findings of the analytical work are supported by a recent series of trials undertaken in a joint study by the UK's Forest Civil Engineering and the University of Nottingham (Brito & Dawson, 2008) at Ringour, Dumfries and Galloway, Scotland. They trafficked pavements made of three different aggregates constructed to the same thickness over a rock subgrade such that all rutting, if it occurs, has to be in the aggregate layers. The pavements were artificially wetted (or not) and trafficked by a variety of vehicles with different axle weights, wheel arrangements and tyre pressures. The results are summarised in Figure 10.

For the pavements with low total rutting there is normally an initial high rate of rutting followed by a stabilising response (e.g. Trials 1, 9 & 10) This type of response is very common (Dawson, 2008). Other pavements show an on-going development of rutting. In some cases this development is very large and rapid (e.g. Trial 3) but, for the most part, it is moderately fast.

Figure 10 also clearly shows the beneficial effect of decreasing the tyre inflation pressure. Consider Trial 7 in which the tyre pressure was progressively increased. With the pressure at 75 and 100 lbs/in^2 (517 and 690 kPa) the rutting initially increased but then began to slow, yet the application of further trafficking at higher pressures caused the rutting to recommence and to accelerate. Similarly, comparing the two Trial 8 tests, the rutting damage due to the 110 lbs/in^2 (758 kPa) tyres is much greater than the rutting under 70 lbs/in^2 (483 kPa). Trial 4B – with the same vehicle, same aggregate and an intermediate mean tyre pressure, performs between the two cases of Trial 8. Trial 3, with the same aggregate material as Trial 8, but with "super-single tyres", has amongst the worst behaviour of all trials (it has the highest total rutting, 137mm). The use this type of tyre with a mean tyre pressure of 114 lbs/in^2 (786kPa) generates a condition for which such behaviour was to be expected.

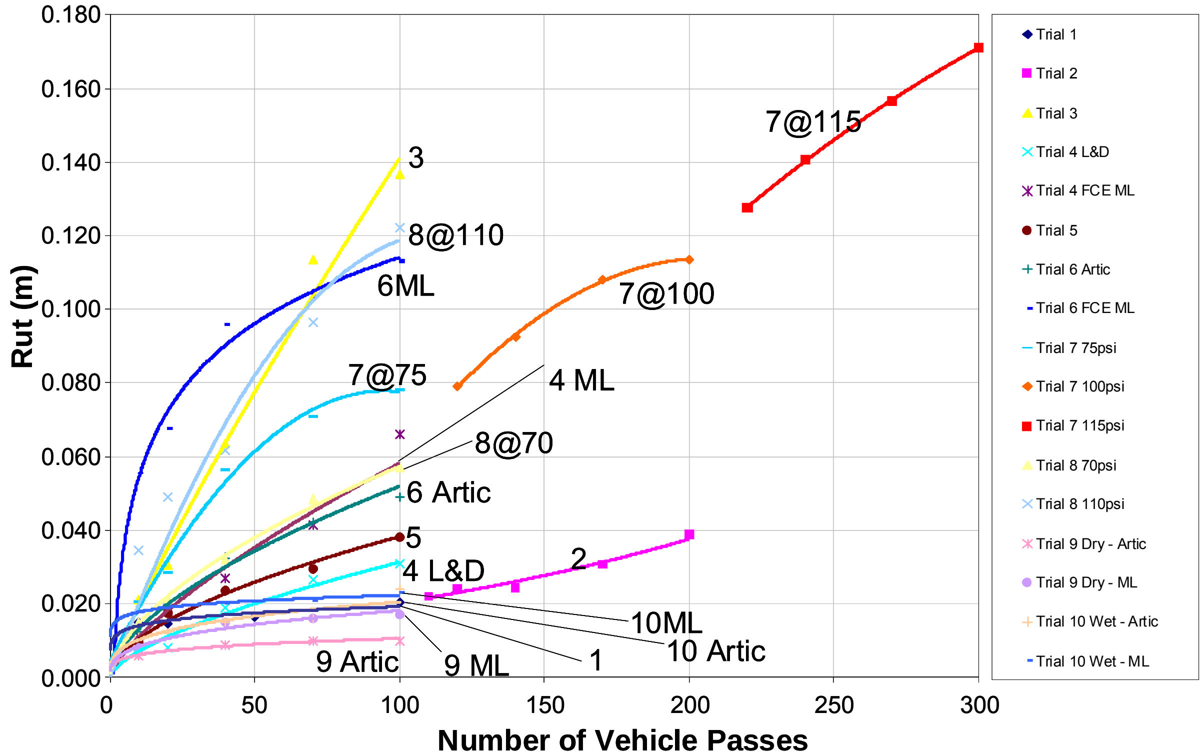

Figure 10 Summary rutting information from RUTT trials

The effects of "super-single" versus dual-tyre arrangements are not so clearly identifiable. Trials 6, 9 and 10 had parallel trafficking by vehicles with and without "super-single" tyres arrangements. In each case the "Artic" vehicle comprised a tractor with a dual-tyred drive axle, single tyres on steering and "tag" (liftable) axle and then a three-axle trailer all axles being equipped with "super-single" tyres. The "ML" vehicle was a rigid-bodied vehicle with four axles, the front two axles being steerable and equipped with single tyres and the rear two drive axles being equipped with dual tyres. It can be seen that, contrary to the analysis presented in this report, there is less damage from the vehicle with some "super-single" tyres. However, damage is also a consequence of axle mass and also depends on which axles do the most damage. In comparing the effects of these two vehicles, therefore, it is impossible to say whether there is a beneficial effect of the dual tyres over the "super-singles" because axle loads were not the same and, in general, incompletely known. Also, at least in some cases, the evidence suggests that a significant proportion of the rutting damage was caused by the steering axle – a loading case that has not been analysed in this report.

Unfortunately, stiffness values and ultimate strength values are not yet available for these pavements so direct comparison with the design strategy is not possible. Also, except for the low pressure "super-single" loading case, the aggregate was used at a much greater thickness than covered by the design cases in this report. Therefore, taking a low stiffness case (because the rock subgrade is undoubtedly stiff), the thickest aggregate layer possible the design information discussed earlier would show some benefit of lowering tyre pressure for dual tyres and only a small benefit in lowering tyre pressures in "super-single" tyre.

Qualitatively, this seems to be supported by the available data shown in Figure 10. Trial 8, trafficked by the predominantly dual-tyred vehicle ruts the pavement a lot less when its tyre pressures are reduced from 758 to 453 kPa (similar to the 800 and 400 kPa cases considered analytically). No directly comparative data is available for the predominantly “super-single” equipped vehicle, but the data from Trial 7 seems to show some, but less pronounced benefit as the computations lead us to expect.

5.4.3 VESILAHTI LOW VOLUME ROAD SITE

A second site was also identified in the Project for the verification of the ROADEX design approach. This site is located on a low volume road in Vesilahti some 30 km south from Tampere. The site has a heavily instrumented section of low volume road pavement with a total thickness of approximately 500 mm underlain by a silty subgrade soil (see more details in Luomala & Kolisoja 2008). Unfortunately the development of rutting in the test structure at the site, during the year and a half since installation of the instrumentation, has been marginal and well-controlled and as a result the planned test loadings using variable confining pressures could not be carried out by the time of writing this report. This work will be reported in the future.

5.5. CONCLUSIONS

In broad terms the approach laid out in this report can be seen to be in line with site observations. Without a dedicated study to investigate the issues directly, some subjectivity necessarily creeps into the interpretation, but the pattern of behaviour and such computations as are possible are confirmatory.

Chapter 6. DISCUSSION CONCLUSIONS & RECOMMENDATIONS

6.1. DISCUSSION

The work performed in this study has built on work performed earlier in the ROADEX II study (Dawson & Kolisoja, 2005). It is believed the approach has an a-priori advantage over other methods. This, first, benefit is the sound theoretical base from which it springs. While the authors and the whole research team in ROADEX recognise that practical solutions are needed if the roads in the Northern Periphery are to be improved, solely empirical procedures, as have been widely used in the past, are not able to provide the ability to handle the changing characteristics of available materials, traffic loadings and contractual arrangements.

On the other hand, a theoretically-based method, such as that defined here, can provide a sound basis for investigating the sensitivity of pavements to a host of changing inputs. Furthermore it allows the pavement owner to evaluate alternative strategies both not only with regards to materials for construction, but also with regards to defining trafficking restrictions / requirements. For the first time the pavement owner can assess the effects on rutting of providing a “low-tyre-pressure-only” restriction if he/she so desires. Taking this approach one step further, the new approach provides a basis for a pavement owner to establish an equable charging policy based on the damage likely to be caused by individual vehicles.

Despite these major advances, it cannot be said that all these benefits are yet in place or that the approach is robust in every respect. Thus the following sections suggest means by which the approach could be further improved.

Mention was made of a need for a simple approach and this, secondly, is the other chief advantage of the new approach provided here. From simple assessments and direct information on vehicles in use, a design approach is available using charts or simple look-up tables.

6.1.1 IMPROVEMENTS TO CALCULATIONS

Inevitably there are improvements that could be made. As readers will have observed earlier in this report, the “best” computational tools were not employable for a number of reasons. Therefore, at some future point, it would be very useful to repeat the computations but using a full finite element approach. This should incorporate material non-linearity, tension cut-off and a failure criteria to enhance the reliability with which the stresses in the pavement are computed. In addition, a plastic analysis could be made at the same time as a second level means of providing confirmatory support that the deformations being predicted are reliably being assessed.

6.1.2 IMPROVEMENTS TO LABORATORY AND *IN-SITU* ASSESSMENTS

With regard to the determination of strength and stiffness parameters, it would be desirable if the use of the simple equipment discussed in the ROADEX II report could be further evaluated and more reliably linked to laboratory testing. While material properties can be assessed at present and the assessment improved with experience, the full benefits of the new method will not be realised until these can be assessed simply and accurately at all stages of the design procedure. Given that most pavement works involve rehabilitation rather than new design, there is a rather great need for these tools to be used either in-situ or repeatedly on many specimens so as to assess inherent variability. Of the further developments and improvements discussed, this probably has priority.

Calibration of the outputs against other trials or, if possible, the establishment of trials specifically for the purpose, would also be immensely valuable

6.2. CONCLUSIONS

On the basis of the work previously performed under ROADEX II, and the supplementary research study performed in ROADEX III, it is concluded that:

- A stress-analysis can be used to obtain the stress distribution throughout an unsealed or chip-sealed pavement under a certain applied loading, if basic stiffness properties of the subgrade and aggregate base layers are known.
- The stresses so computed can be associated with rutting or non-rutting behaviour on the basis of the closeness to static failure conditions.
- This approach can be further simplified by considering the stress value *S*, being a single stress measurement indicative of the magnitude of the stress experienced in the pavement and, hence, by reference to the previous observation, an indicator of rutting propensity.
- If a similar value, S_f, is defined – being the value of *S* required to induce static failure, then a type of "factor of safety" against failure is available.
- On the basis of previous experience in ROADEX II and limited trafficking observations, permissible "factors of safety" have been established for occasionally and more regularly trafficked pavements or pavements that are particularly softened by excess moisture or spring-thaw.
- The overall approach combines a sound theoretical basis with a simplified application so that it should be useable by engineers in the Northern Periphery without access to equipment or a skill base that would be required for modern mechanistic design procedures used in more urban settings.
- While the framework is now fairly well established, there is room for some improvement, particularly in assessing aggregate characteristics in more reliable ways and in calibration of predicted performance with more experimental studies *in-situ*.

Chapter 7. REFERENCES

Brito, L.A.T. & Dawson, A.R., 2007, “Roads Under Timber Traffic”, Preliminary Research Report, December, NTEC Report No: 07016.

Dawson, A.R. (2008 – in press). “Rut Accumulation in Low-Volume Pavements due to Mixed Traffic”, paper accepted by *Transportation Research Record*.

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Korkiala-Tanttu, L., Laakosonen, R. & Törnqvist, J., 2003, ”Kevään ja ylikuorman vaikutus ohutpäällysteisen tien vaurioitumiseen, HVS-Nordic-tutkimus” (The effect of the spring and the overload to the rutting of a low-volume road, HVS-Nordic-research), Finnish Road Administration, Finnra Reports 11/2003, Helsinki (In Finnish)

Luomala, H. & Kolisoja, P., 2008, Effects of Seasonal Frost on Pavement Stiffness in Finland According to a Pavement Monitoring System, TRB 87th Annual Meeting, Washington D.C.

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Annex. ILLUSTRATIVE USE OF DESIGN APPROACH & DESIGN CHARTS

Illustration of use of design method

Subgrade – Dry silt

Base course – Crushed gravel (candidate material) *similar to NIP material*

Traffic – Lorries with super singles fitted ; 800kPa tyre inflation (116psi)

i. Assessment of the subgrade indicates Cu = 50kPa; Mr = 40MPa
ii. Assessment of base course – crushed gravel → DCP = 15mm/blow (From Figure 44 of the ROADEX Report Task 2.1), equivalent to CBR =15% (Mr approximately 150MPa)
iii. 800kPa of tyre pressure on a super single tyre → Radius =13.5cm
iv. Diagram A selected → Super Singles → Ebas/Esub ≅ 4
v. Subgrade allowable stress = 4Cu = 200kPa on Diagram A [below] (SS-800-R4 line). Hence pavement will require 1.5 Agg. Thick. / Load Radius Ratio (20cm of base)
vi. Failure envelope – Consider material as NIP (Sf = 255 kPa)
vii. Select Diagram B → Super Singles → Ebas/Esub ≅ 4 → Agg. Thick. / Load Radius Ratio = 1.5
viii. S = 244kPa; S/Sf=96% → Failed to prevent rutting in the base layer (90% is limit)
ix. Alternative 1 – Change Traffic – Super Singles with 400kPa (58psi)
x. Select Diagram C → Super Singles → Ebas/Esub ≅ 4 → Agg. Thick. / Load Radius Ratio = 1.5
xi. S = 240kPa; S/Sf = 94% → Failed to prevent rutting in the base layer
xii. Alternative 2 – Change Traffic – Twin Tyres with 800kPa (116psi) – Diagram D
xiii. S = 229kPa; S/Sf = 89%→ OK (now <90%)
xiv. {Another alternative would have been to change the material to make it stronger}.
xv. FINALLY – return to step i with new traffic (radius of tyre now 9.5cm) and use dual tyre version of Diagram A with the DT-800-R4 line to obtain 1.65 Agg. Thick. / Load Radius Ratio, i.e. base thickness should be 1.65 x 9.5 = 16cm thick. As thickness difference is small, 20cm is selected as the new base layer.
xvi. If 16cm had been selected, steps vii and following would need reassessment.

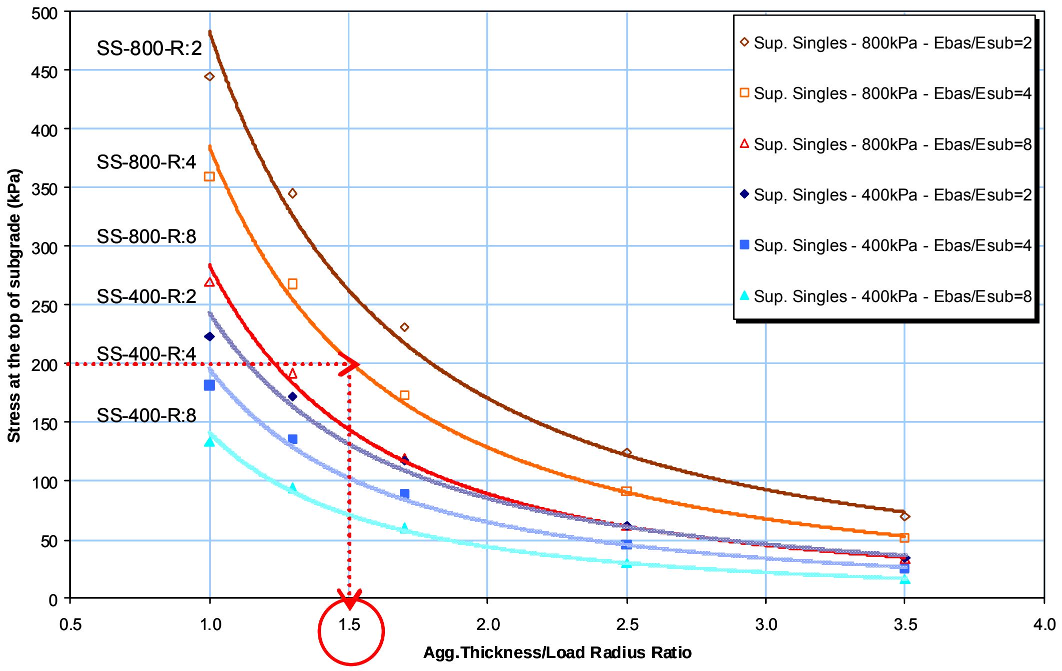

Stress at the top of subgrade (kPa)
Agg.Thickness/Load Radius Ratio
SS-800-R:2
SS-800-R:4
SS-800-R:8
SS-400-R:2
SS-400-R:4
SS-400-R:8
Sup. Singles - 800kPa - Ebas/Esub=2
Sup. Singles - 800kPa - Ebas/Esub=4
Sup. Singles - 800kPa - Ebas/Esub=8
Sup. Singles - 400kPa - Ebas/Esub=2
Sup. Singles - 400kPa - Ebas/Esub=4
Sup. Singles - 400kPa - Ebas/Esub=8
0
50
100
150
200
250
300
350
400
450
500
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

Diagram A

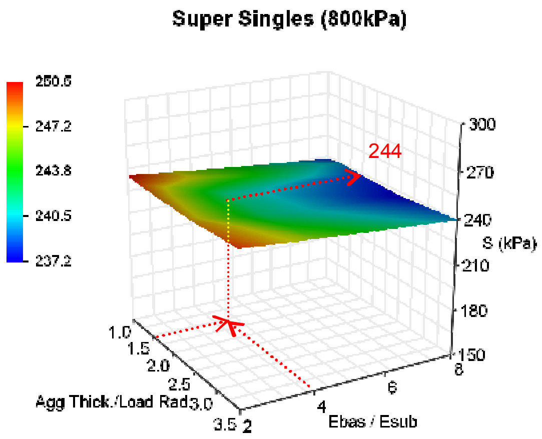


Diagram B

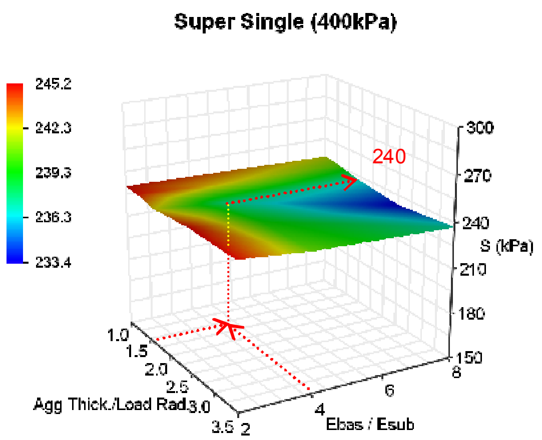


Diagram C

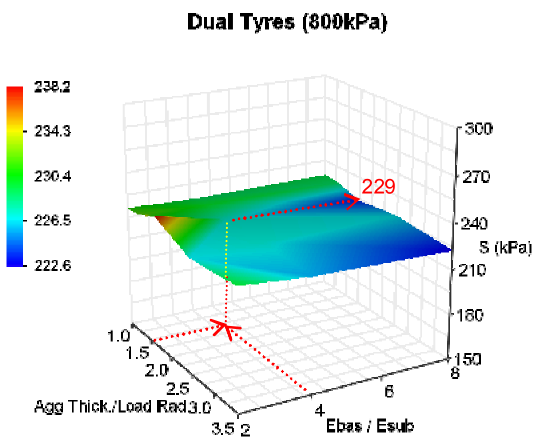
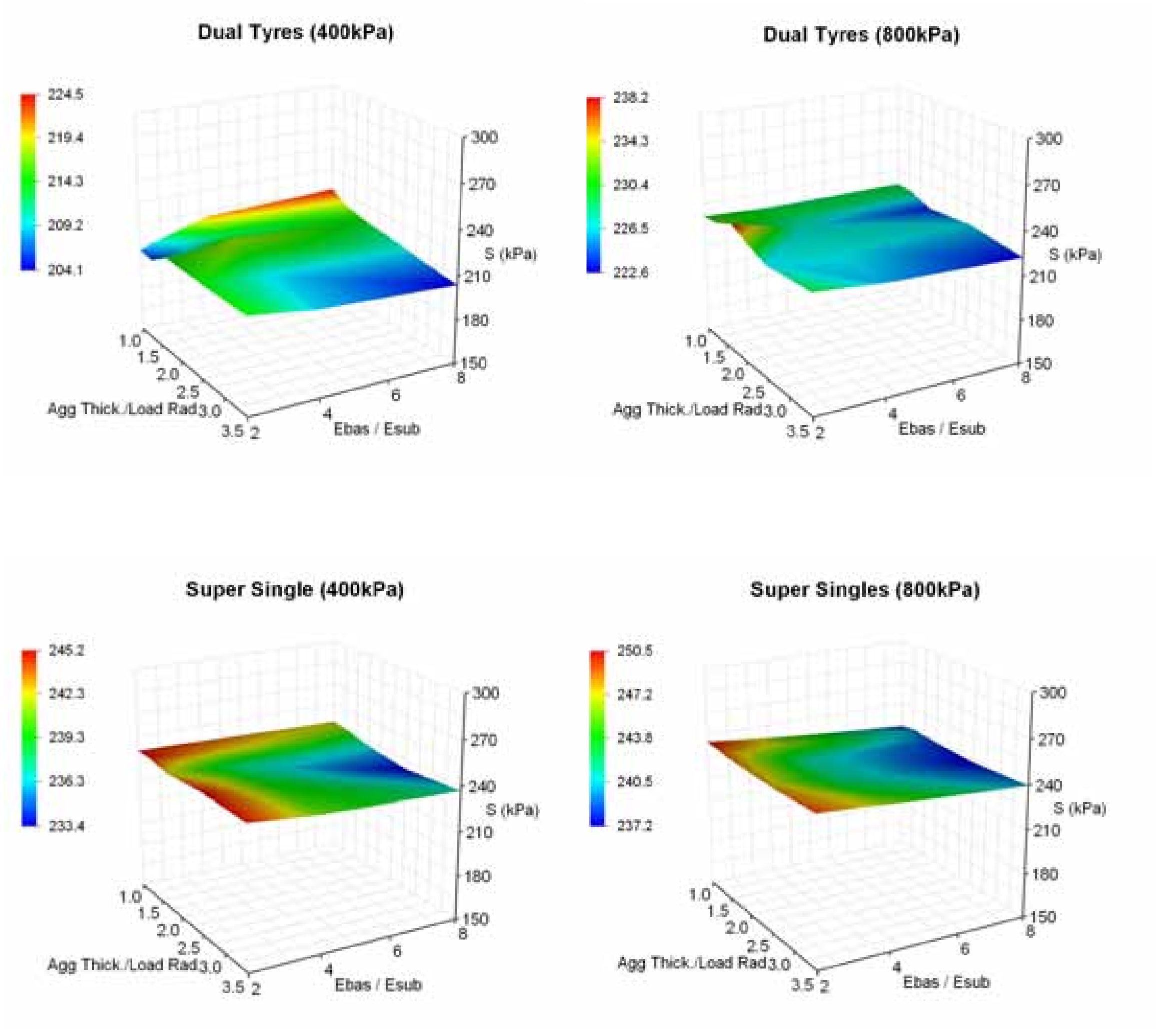


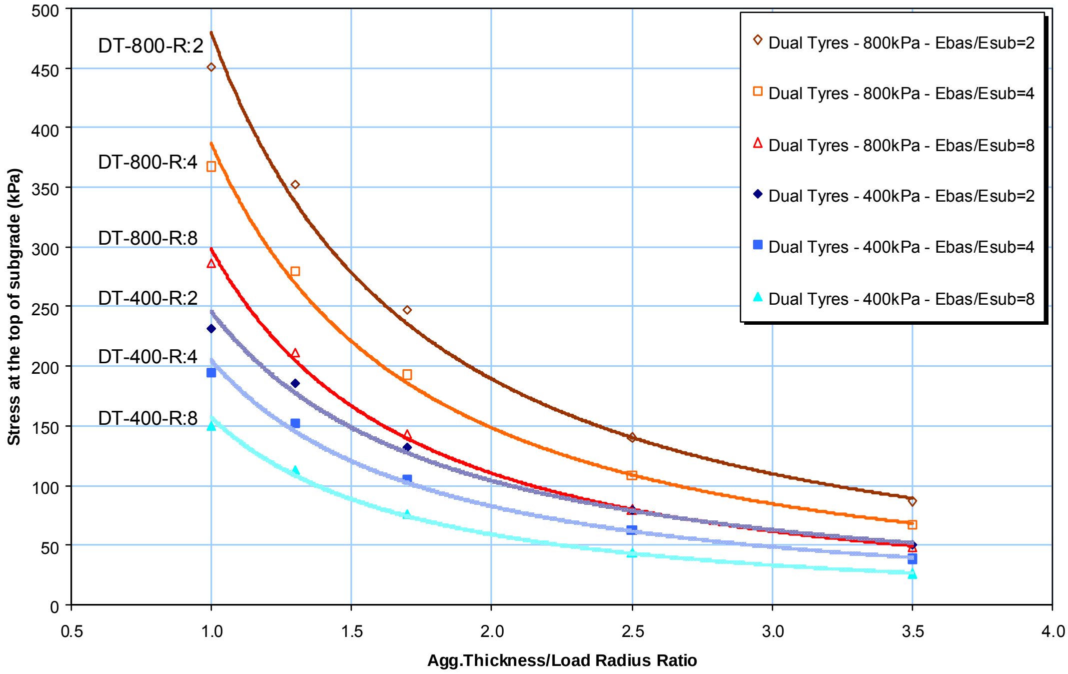
Diagram D

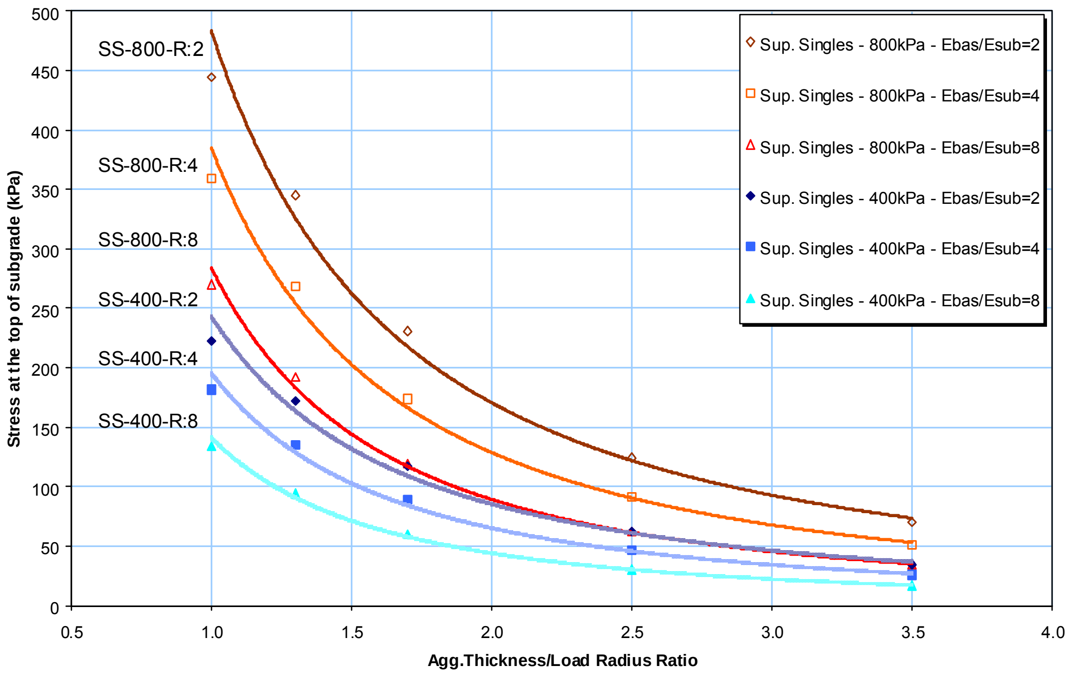
Four design charts for computing stress variable *S* in the aggregate layer



The data used for preparing these plots is tabulated in Table 4 and is available on-line at the ROADEX web-site to permit interpolation (see http://www.roadex.org).

Two charts for computing stress applied to subgrade surface





Top: Dual Tyres. Bottom: Super Single Tyres

ROADEX III PUBLICATIONS

Developing Drainage Guidelines for Maintenance Contracts

Tyre Pressure Control on Timber Haulage Vehicles

Understanding Low-Volume Pavement Response to Heavy Traffic Loading

Health Issues Raised by Poorly Maintained Road Networks

Road condition management policies for low volume roads – tests and development of proposals

Policies for Forest Roads – Some Proposals

Road Construction in Greenland - The Greenlandic Case