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Design of low-volume pavements against rutting – a simplified approach

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\textbf{ABSTRACT.}

Roads that connect remote communities to each other and to urban centers are essential for community survival, yet (often) must be funded from a very small taxation base. Because of their thin, often unsealed, construction, the pavements forming these roads typically fail by rutting. Thus this paper seeks to identify the causes of rutting, to identify simple methods of material assessment suitable for use by local road engineers with limited resources at their disposal and, hence, to propose a simplified means of designing pavements against rutting that is usable by engineers in these remote locations. An advanced testing and analytical approach is reported using repeated load triaxial testing of aggregates and non-linear finite element analysis of chip-sealed pavement constructions. The results of this are used to develop a permissible stress approach for design purposes. This approach uses simple stress analysis, by chart and PC-based computations, with readily available in-situ evaluation of materials, in order that the simple-to-use goal is delivered.

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INTRODUCTION

The northern periphery of Europe is an area of sparse population. People are usually concentrated in small coastal towns and villages, with fishing, forestry and, in some locations, tourism, traditionally providing the main sources of employment and the backbone of the local economies. Whilst very self-sufficient compared to comparable towns closer to the large urban areas further south in Europe, yet these remote communities depend on efficient access to those urban areas. Fish and fish products, wood and wood products must be transferred to those the towns and cities of the remainder of Europe as these are the main markets for the rural products. Tourists from those urban centers must be able to access the remote areas. Health and education provision and supplies must all be brought in.

There is a strong political will to encourage these peripheral communities to prosper and for rural depopulation to be prevented, yet there are many hindrances to this being achieved. Not least of these hindrances is the economic provision of the road infrastructure. Despite only carrying low traffic volumes, their deterioration rate tends to be fairly high due to the economic initial construction, their expedient alignment giving rise to locations with poor drainage [1], the harsh weather and the high axle loads necessary to take out and bring in goods and services in as economic a manner as possible.

RoadexII, an EU financed research project, has been studying the construction and maintenance of the low volume road network in the Northern parts of the Nordic countries (Finland, Sweden and Norway) and in the Highlands and Western Isles regions of Scotland in the UK, all under the leadership of the Highland Council (Scotland). Figure 1 shows the RoadexII study areas. The overall aims of RoadexII are wide ranging whereas this paper has a more limited objective which is to define a simplified, expedient, method of pavement design for situations where rutting is the key distress and where there is no advanced design and testing capability – a common combination in Europe’s northern periphery.

RUTTING

An earlier study [2] concluded that the chief distress that is observed in the low-volume roads common in these peripheral areas of Europe, being unsealed or only having a thin seal, was rutting. Some develops year round, but far greater rutting is seen during spring-thaw when the aggregate in the upper layer thaws due to warmer weather but while the lower subgrade and margins remain frozen. Often low budgets in the past mean that the permanent deformations that developed in the road surface were repaired by reshaping the wearing course material on the road surface. Since these kinds of repairs have been done time after time it is common for the thin structural layers to have become mixed together and with the subgrade material. It is concluded that management of the risk of permanent deformations due to heavy traffic loads especially during the thawing period, but also during rainy seasons, is of vital importance. A better understanding of the mechanisms and factors influencing the development of permanent deformations in these extreme conditions is therefore required and is now discussed.
THE CAUSES OF RUTTING

Rutting can occur for a number of reasons. Fundamentally there are four contributory mechanisms that are here labeled, arbitrarily, as Modes 0, 1, 2 and 3.

Mode 0
Compaction of non-saturated materials in the pavements can be a contributor to rutting (Figure 2a). Normally compaction prior to trafficking is considered sufficient to prevent further compaction under trafficking. Furthermore this mode is self-stabilizing - i.e. compaction under trafficking hinders further compaction. It also causes the material to stiffen and hence to spread load better. Better load spreading leads to a reduced stress on the subgrade, thereby reducing the amount and risk of rutting at that level. Rutting of this type is seen as a narrow depression relative to the original surface (Figure 2a). For these reasons, a limited amount of rutting by this mode is beneficial. Good compaction minimizes it.

Mode 1
In weaker granular materials, local shear close to the wheel may occur. This gives rise to dilative heave immediately adjacent to the wheel track (Figure 2b) in which granular material has undergone large plastic shear strains and consequent dilation, leading to relatively loose material. This rutting is thus, largely, a consequence of inadequate granular material shear strength in the aggregate relatively close to the pavement surface. Evidence from trial pavements [3] and from theory (a consideration of the stress field using a Boussinesq approach) has demonstrated that the maximum shear is felt at a depth of approximately 1/3rd of the width of the wheel (or width of the wheel pair where twin tires are used). In pavements with significant traffic wander (wide lanes, roads with no markings and roads without existing ruts) the depth may be a little deeper.

In this mode, ideally, there would be no deformation at the subgrade surface. This type of rutting is frequently observed in the Nordic area that is affected by seasonal frost. There, in many cases, it is likely to be the main contributor to the accumulation of rutting when, near the surface, an aggregate of inadequate quality is loosing its load carrying capacity, for a short time, as it thaws in the spring and contains excess moisture. For the rest of the year the same material, re-compacted and drained, is likely to have an entirely adequate performance.

The only remedy for such rutting is to improve the aggregate or to reduce the tire imposed stresses. Subgrade treatment will have no effect on this mode of rutting. The granular material may be improved by compaction (within limits), by stabilization, by the use of a geosynthetic reinforcement or by improving the conditions which control its behavior - e.g. by drainage. If none of these is effective, the aggregate may have to be replaced.

Mode 2
When aggregate quality is better, then the pavement as a whole may rut. Idealized, this can be viewed as the subgrade deforming with the granular layer(s) deflecting bodily on it (i.e. without any thinning) – see Figure 2c. The surface deflection pattern is of a broad rut with slight heave remote from the wheel (as it is the displacement of the soil which causes this).
In regions affected by deep seasonal frost, the spring thaw problem discussed above can lead to Mode 2 rutting of the subgrade. In such situations excess Mode 2 rutting may be seen only in the spring when subgrades are softened for a few weeks by excess moisture consequent upon thawing. Figure 3 shows a Ground Penetrating Radar (GPR) record and interpretation with the damaged subgrade zone clearly shown. Although continual grading keeps the road surface reasonably in shape, the deformation of the layer boundaries at the bottom of the unbound wearing course and, particularly, at the base/sub-base course boundary and at the top of the embankment is very evident (note the uneven thickness of many of the layers across the pavement, the general depression of the constructed layers into the subgrade and the wheel path rutting at the top of the subgrade).

**Mode 3**

Particle damage (e.g. attrition or abrasion, perhaps by studded tires but more often due to heavy trafficking and turning movements) can be a contributor to the same surface manifestation as seen in Mode 0 rutting (Figure 2a) though, of course, the mechanism is very different.

In practice rutting will be a combination of all the above mechanisms. It is expected, and to some extent observed, that Mode 1 will be more evident with channelized trafficking (e.g. as is the case with many forest roads) where wheel wander is not available to displace-back, and generally compact, aggregate (Mode 0). Conversely, Mode 2 is expected to be more evident under wandering traffic with Mode 0 more likely to make a contribution in this case as the "kneading action" of a wandering tire is effective in achieving compaction.

**BASIC ASSESSMENT OF RUTTING PROPENSITY**

For this study, it is assumed that Mode 0 requires no assessment because it is beneficial and self-stabilizing. Mode 3 was not studied, either, assuming that particle wear can be assessed and prevented using classical particle-related testing. To study Mode 1 and 2 behaviors, initial resort was made to simple-to-perform Tube Suction tests [4] and more advanced repeated load triaxial tests. A wide range of geological origins, grain size, stone quality, shape, etc. were assessed in one or both tests. Table 1 summarizes the main properties of the most important aggregate types used. They were compacted at near-optimum moisture condition but some were then allowed to suck water in from a free supply at the specimen base and some were then frozen and then allowed to thaw. One of the aggregates was of moderate quality metamorphic aggregate from Scotland - known as “Quarriebraes” aggregate. The other two aggregates were taken from the site of an environmental pavement condition station (‘Percostation’ [5]) at Koskenkyla near to the town of Rovaniemi in Northern Finland. From that site the base course material was crushed rock while the sub-base material was gravel.

Most of the repeated load triaxial tests were performed according to a test procedure specifically adapted to simulate the effects of seasonal variations by including a freeze-thaw cycle [6, 7]. The applied permanent deformation tests procedures consisted of a sequence of load cycles at a number of different combinations of cyclic deviator stress. Figure 4 provides an illustration of the typical response in a test in which each part of the sequence comprised 50 000 cycles. On the basis of performing testing with varying numbers of applications,
repeated load failure was somewhat arbitrarily defined as being when >2% axial plastic strain is accumulated in 5000 cycles of loading at a certain stress level.

A loss of performance (i.e. increased deformability) was observed with samples after a freeze-thaw cycle. This appears to be mostly due to increased water content caused by suctions developed during freezing and only a little by de-densification due to ice formation. Thus, if both the freezing layer is non frost-susceptible, and the layer beneath is also non frost-susceptible, then little reduction in performance should be expected in the pavement.

It is not clear from the testing whether the fines always have a direct effect on increasing plastic strain (though sometimes they do), but they certainly have a secondary effect, allowing the aggregate to hold more water, due to capillary suction, and this leads to much more rapid build-up in plastic strain. This capillarity is undesirably exploited on freezing because cryo-suction effects, acting at the freezing front, are able to pull in more and more water, where available, from the edge or bottom of an aggregate layer.

Data such as that presented in Figure 5 shows how the rate of accumulation of permanent deformation increases with stress level – that is it accumulates more rapidly when the repeatedly applied stress state is closer to the static failure stress. Accordingly, it would be sensible, for design purposes, to ensure that the stress experienced by the pavement doesn’t exceed a certain fraction of the failure stress in order to reduce the rate of deformation accumulation to an acceptable level. Previous researchers [8] have suggested a ratio of $q/q_f = 0.7$ – i.e. the deviatoric (or shear) stress applied, $q$, is limited to 70% of that needed to induce static failure, $q_f$, under, in other respects, the same stress and material condition. The data obtained from the materials in Table 2 suggests that this may, indeed, be a sensible determination, but that, when the aggregate is very wet with fines preventing rapid drainage and also during spring thaw conditions, this value (generalizing greatly) should be reduced to 50-55% of that wet or fine material’s static strength if the amount of rutting is to be kept small. Further details of test results are available in [6] and [7].

The Tube Suction results (Figure 6), when compared with permanent deformation measured in the repeated load triaxial test, suggest that a dielectric value of <9 are required to ensure that rutting is limited in magnitude. Although the data on which to base this recommendation is limited at present, the dielectric boundary value of 9 is similar to that recommended by others [9] based on other testing and on performance observations.

ANALYSES

The resilient data obtained from the triaxial testing was used in a non-linear axi-symmetric finite element routine known as FENLAP [10]. This program incorporates several non-linear constitutive stress-strain models for soils and aggregates. Load is applied incrementally (typically in 10 steps) and the program iterates after each stage until it find a complementary set of stresses and stiffnesses according to the model(s) in use. The program redistributes stresses internally if plastic yield would otherwise be reached, using a non-stress-dependent associative flow rule for simplicity (as only the resilient behavior is being modeled). The plastic
strains that would result are not, however, calculated. For computational reasons the program requires an asphalt surface, so the analyses were performed with asphalt thicknesses varying from 1mm to 200mm. Only the results from the pavements with thin asphalt layers, being of relevance to low volume pavements, are discussed here.

The asphalt and subgrade materials were allotted typical, constant stiffness values – for the asphalt this was a stiffness modulus of 1.5 GPa whereas for the subgrade the value was 40MPa. A value of 0.45 was adopted for the Poisson’s ratio for both layers. Arguably this is rather high for asphalt concrete although, given the thin layers analyzed, the results are not expected to be sensitive to assumptions of Poisson’s ratio values for the asphalt. In the aggregate layer the model developed specifically at the Technical University of Dresden for modeling unbound granular materials [11], was implemented. The loading used in the analyses was a circular plate of diameter 334mm with a tire pressure of 650kPa.

The aim of the analyses was to cover a range of aggregate conditions, so aggregates at various moisture conditions and with various temperature histories were modeled (Table 1). From the analysis of the pavements it is possible to compute the stresses within them. Because the aggregate has a modulus which depends on the stress it experiences and the stress in turn depends, in part, on the modulus, it follows that an iterative analysis is needed. Once FENLAP obtains a converged solution, the value of stiffness and stress are in harmony within the aggregate.

Observations on Rutting in the Aggregate Layer (Mode 1)

Figure 7 shows the computed vertical and horizontal stresses in pavements containing the ‘saturated’ (moisture content is 11%) high fines Quarriebraes aggregate in their granular layers. The rapid diminution of both vertical stress and horizontal stress is evident. Note also how the increase of the asphalt above 40mm thickness significantly reduces the vertical stress – because it is then that the asphalt begins to act as a beam in bending taking a far greater role in load spreading than it did when thinner. Whether this boundary in behavior is exactly located at a thickness of 40mm is, however, somewhat open to conjecture. Nevertheless, the principle is clear: Asphalt needs to be greater than a threshold thickness if it is going to achieve effective load spreading and significantly reduce stress on the lower aggregate base layers.

Figure 8 presents the value of shear stress at a depth into the aggregate layer of around 1/3rd of the width of the loaded area (the depth is chosen for the reasons explained earlier – see ‘the Causes of Rutting’ (Mode 1). That would be approximately 100mm or greater beneath the pavement’s surface for the pavements with a 1mm, 10mm and 20mm thick asphalt cover.

The figure shows that, in general, the thickest asphalt cover leads to the lowest stresses in the aggregate, although this is not always the case as a thinner asphalt not only changes the stress but also, thereby, the modulus which in turn leads to different stresses. Thus the relative stress seen in the aggregate as the asphalt cover changes is also affected by the material’s stiffness characteristics. Figure 8 also shows that the material is closest to static failure at a depth of 100mm or 150mm into the pavement (around 90 and 140mm respectively
into the aggregate). It is this stress condition that the designer needs to keep away from the failure line in order to reduce the onset of rutting within the aggregate layer (Figure 5). This might be achieved by moving the failure line (e.g. by improving the granular material by, for example, drainage) or by placing more asphalt to move the aggregate stress condition lower.

This depth of maximum shear loading is entirely in-line with Boussinesq theory as described earlier, with maximum shear at a depth of between 1/3rd and a half of the width of the loaded area. For the analysis performed here that should be at a depth of between 110mm and 170mm in an un-sealed pavement – which is exactly the depth shown in Figure 8. A slightly greater depth would be expected for thin asphalted pavements as the asphalt will act as a partial load spreader, in effect making the loaded area a little wider.

Interpreting this data in a q/q$_{f}$ manner, as above, maximum values of the q/q$_{f}$ failure ratios for the Quarriebraes dry moderate fines aggregate are between approximately 65% (20mm asphalt) and 80% (10mm asphalt). These values indicate that these pavements may be expected to experience premature rutting under trafficking in wet and/or thawing conditions. It was observed in separate trafficking studies [12] that the Quarriebraes material does show rutting, particularly in wet weather and this gives some explanation as to why this should be.

**Observations on Rutting in the Subgrade (Mode 2)**

As the aggregate layer is thickened, so the stress on the subgrade reduces. Analyses were performed with varying thicknesses of the aggregate with an arbitrary target vertical stress of the subgrade set as 70kPa. In reality the subgrade would rut, or not, depending on the shear stress relative to its shear strength in much the same way as was discussed for the aggregate. However, as this analysis included only a very simple subgrade model and as no definitive testing was performed on subgrade materials, it is only important that some reference permissible stress is used to compare the ability of the aggregate to spread load. On this basis the thicknesses of the different aggregates to achieve the arbitrary 70kPa vertical stress were computed and are shown in Table 1.

It is immediately apparent that the reduction in moisture and in fines leads to an improved material which can therefore be used more thinly to achieve the same load spreading. The corollary is that a large increase in aggregate thickness is required to counteract the effect of the poorer load-spreading ability of an aggregate in poorer condition.

**SIMPLIFICATION**

Hence, positions in the pavement where a candidate aggregate would be overstressed (with respect to its permanent deformation behavior) can be identified. Then appropriate changes can be made to the structure or materials, as desired. But the above approach isn’t suitable for routine economic application in remote rural areas where advanced testing and/or computational tools are not available. Thus, this approach was simplified with a Boussinesq-based stress analysis (available in chart form in, e.g., [13]) as an approximate replacement of the FE method and in-situ testing to replace the triaxial test, as follows.
DESIGN AGAINST RUTTING

To prevent rutting in practice, the following strategy is proposed:

i. Check that the tube suction measured dielectric value of the compacted aggregate is less than 10, preferably less than 9, after drying the specimen at 40 - 50°C and then letting it absorb water from the base of the specimen until a constant mass is reached. This is to limit the moisture susceptibility of the base material.

ii. If the desired dielectric value cannot be obtained then use the material with special water protection provision. This should comprise a capillary break beneath the aggregate layer in seasonally affected frost areas and, if at all possible, a sealed surface in all locations. This sealed surface should extend beyond the carriageway to maximize the width of sealed shoulders (though full structural capacity is not required there) as research elsewhere [14] has indicated the significant benefit of keeping the wetting path as long as possible.

iii. Assess the strength of the aggregate in a wet condition in-situ. This can be achieved by wetting the aggregate in a compacted area (e.g. by use of a small trial pad constructed at the aggregate source prior to construction commencing) and then using the Dynamic Cone Penetrometer (DCP) - the DCP has been found to provide a rapid strength determination of aggregate layers and established correlations [15, 16] are available to allow estimates of strength, in terms of angle of frictional resistance (φ) or to a line on a p-q plot, to be made from the DCP’s results. In this way a specific version of Fig. 8 may be produced. The DCP is only usable with reliability if the maximum particle size is less than 40mm. [For coarser aggregates (e.g. as often used on forest pavements) there is no simple means of assessing the strength.]

iv. Design lines are now plotted at 70% of this limiting value for conventional conditions and at 55% for wet, fines-rich aggregates (and those suffering freeze-thaw), respectively.

v. Use Boussinesq theory to predict the magnitude of the combination of shear and mean stress for the traffic of concern which most nearly approaches the boundary lines set in Step iv. This will require a knowledge of:
  - The traffic imposed stress
  - The wheel print
but not of the stiffness of the layer as this is not part of Boussinesq’s requirements.

vi. Plot this computed stress against the stress limit(s) established in step iv. Check that the stress condition is not too high.

vii. If the stress is too great then either replace or treat the aggregate, or add a covering layer. If aggregate treatment is proposed, go to Step i. If added thickness will be the solution then Step v can be used to predict the new stress state. From a comparison of analyses with very thin asphalt layers and those with thicker asphalt, the authors have observed the approximate rule that an asphalt layer may be treated, for the purpose of simplified pavement analysis, as being aggregate with a thickness 1.5 or 2 times the asphalt thickness (asphalt thickness <60mm or >60mm respectively). This makes an allowance for the asphalt’s greater load-spreading ability.

viii. Having designed the aggregate and any overlying layers to prevent Mode 1 rutting, the aggregate thickness must be designed to prevent Mode 2 rutting. The minimum thickness can be calculated using a layered elastic analysis. Such computational tools are readily available (e.g. the public domain program ELSYM5). The input data for this, however, are a little more difficult to assess. Poisson’s ratio values may be assumed (0.35 for aggregate and 0.45 for subgrade soils are typical values). Stiffness modulus values will
also be needed. The simplest means of collecting such data would be by use of a lightweight falling weight device on trial constructions. The results of the linear elastic layered analysis will be an imposed stress on the subgrade.

ix. The computed vertical stress on the subgrade should be kept to a figure less than 2 times the unconfined strength [8]. The value of the unconfined strength may be determined (e.g.) from published relationships [15, 16] between in-situ measures and laboratory determinations by conventional geotechnical means. However, allowance must be made to ensure that the subgrade is in its worst condition which can be expected during the pavement life, perhaps by artificially wetting the subgrade or by testing during spring thaw. Unconfined strength measurements should be increased somewhat to allow for the confining effect which will be provided by the completed pavement construction which will ultimately lie on top of the subgrade. This allowance can be made using standard geotechnical bearing capacity depth factors.

x. The computations in Step viii are now repeated until the imposed stress onto the subgrade is less than the strength limit described in Step ix.

xi. In areas affected by seasonal frost Steps viii to x may not be sufficient. There the depth of aggregate placed to prevent excessive damage – especially unevenness – will almost invariably be greater than that required to ensure structural acceptability. For such pavements minimum thicknesses should be established, but this is beyond the scope of this paper.

The above procedure does not specifically address durability of aggregate particles. It is assumed that aggregates with very weak stones are not being considered, having been excluded by prior durability assessment using conventional particle assessments.

SUMMARY

A review of mechanisms of rutting in low volume road pavements has been undertaken leading to a design procedure that aims to avoid premature development of any such mechanism. Advanced testing and analysis has provided a means of relating aggregate permanent (plastic) deformation behavior to aggregate strength and in-pavement stresses. This has allowed granular base layers to be designed to resist rutting based on simple granular material testing. Hence, a much simplified and practical method of low volume pavement design has been proposed which, nevertheless, builds on sound engineering principles and provides the basis for a fundamental assessment of materials and their best use. The method has been prepared for adaptation by regional engineers who do not have advanced technical resources at their disposal.

ACKNOWLEDGEMENTS

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Centre of Lapland (Lapin Metsäkeskus), Stora Enso and Metsäliitto, Procurement Area of Northern Finland (see Fig. 1 for approximate locations). The project consultant/manager is Roadscanners Oy from Finland. The authors of the paper gratefully acknowledge the work undertaken by the other members that has stimulated this study. Thanks are also due to the European Union’s ERDF IIIB Northern Periphery Program for partly funding the study.
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List of Tables and Figures

TABLE 1 Aggregate materials used in triaxial and tube suction testing, modeled aggregate stiffness and computed aggregate thickness needed to protect subgrade

TABLE 2 Failure stress sensitivity of Quarriebraes materials

FIGURE 1 Study areas (hatched) of (Left-Right) Scotland, Norway, Sweden and Finland. Shaded but un-hatched areas are also parts of the EU’s Northern Periphery, but were not part of the project reported here.

FIGURE 2a,b,c Mode 0, 1 an2 rutting, respectively

FIGURE 3 A Ground Penetrating Radar record from Kemijarvi, near Rovaniemi, Finland showing interpreted depth of interfaces.

FIGURE 4 Typical result showing cumulative plastic deformation (strain) under a series of increasing axial load pulses and decreasing confining pressures, 5000 load cycles applied at each stress state. \( q = \text{deviator stress}, \ p = \text{mean stress} = \frac{\theta}{3} \)

FIGURE 5 Plastic strain rate information obtained from test on Quarriebraes Qc material (saturated, high fines).

FIGURE 6 Permanent axial strains of triaxial specimens as a function of the dielectric value of the specimen’s top surface. Values of permanent axial strain > 2% indicate early failure of the permanent deformation test (see Fig. 4).

FIGURE 7 Variation of vertical and horizontal stress in the granular layer of pavements with various thicknesses of asphalt. Aggregate is Scottish Quarriebraes material in ‘saturated’, high fines condition (Qc).

FIGURE 8 Stress condition at various depths (*) beneath the top of the asphalt. Data is for the Quarriebraes ‘dry’ moderate fines mix, Qa. Note only small q/p reduction 10 → 20mm asphalt. This is more marked for ≥ 40mm asphalt thickness.
### TABLE 1  Aggregate materials used in triaxial and tube suction testing, modeled aggregate stiffness and computed aggregate thickness needed to protect subgrade

<table>
<thead>
<tr>
<th>Material Description</th>
<th>Code</th>
<th>Moisture Content (%)</th>
<th>Fines Content (%)</th>
<th>Thickness (mm) required to keep vertical stress on subgrade &lt; 70kPa</th>
<th>M, at $\theta=200$ kPa (MPa)</th>
<th>M, at $\theta=500$ kPa (MPa)</th>
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<td>Qa</td>
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Notes: 1] * the Tube Suction Values for the wet materials indicated (albeit on materials with similar moisture values and slightly different fines contents), and obtained by the method described, were 11.5 and 15 for the base & sub-base materials, respectively.
2] $\theta$ is the sum of the three principal stresses, $\sigma_1 + \sigma_2 + \sigma_3$

### TABLE 2  Failure stress sensitivity of Quarriebraes materials

<table>
<thead>
<tr>
<th>Material Description</th>
<th>Code</th>
<th>Moisture Content (%)</th>
<th>Fines Content (%)</th>
<th>$q/p$ at static failure</th>
<th>$%$ of $q/p$ at failure</th>
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<td>Quarriebraes Moist</td>
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<td>2.5</td>
<td>1.65</td>
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<tr>
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<td>Q3f</td>
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<td>15.3</td>
<td>0.76</td>
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<td>Q4f</td>
<td>15.1</td>
<td>11.6</td>
<td>0.45</td>
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<td>Quarriebraes Moist</td>
<td>Q1m</td>
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<td>4.7</td>
<td>67</td>
<td></td>
</tr>
<tr>
<td>Quarriebraes Saturated</td>
<td>Q2m</td>
<td>16.1</td>
<td>2.7</td>
<td>67</td>
<td></td>
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<tr>
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<td>13.2</td>
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<tr>
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<td>Q4m</td>
<td>16.6</td>
<td>12.0</td>
<td>56</td>
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Notes: 1] Q1-Q4f tests are static failure test. The Q1-Q4m tests are repeated load, permanent deformation tests at approximately matching moisture and grading conditions to those of the Q1-Q4f specimens. By comparing the responses of these similar specimen pairs, the maximum q/p value which gives acceptably small permanent deformation has then been compared with the q/p required for static failure. “Acceptably small permanent deformation” is defined as strain ($\epsilon$) development which broadly matches the response $\epsilon=100\times N^{0.15}$, where N is the number of load applications, as this response had been observed in a range of tests to describe a stabilizing (‘shaking-down’) response, whereas greater power law values than 0.15 are usually associated with excessive and/or rapid plastic deformation build-up.
FIGURE 1 Study areas (hatched) of (Left-Right) Scotland, Norway, Sweden and Finland. Shaded but un-hatched areas are also parts of the EU’s Northern Periphery, but were not part of the project reported here.
FIGURE 2a,b,c Mode 0, 1 and 2 rutting, respectively
FIGURE 3  A Ground Penetrating Radar record from Kemijarvi, near Rovaniemi, Finland showing interpreted depth of interfaces.
FIGURE 4 Typical result (here on a high quality, crushed rock, graded, aggregate ≤40mm from Bandridge, Northern Ireland) showing cumulative plastic deformation (strain) under a series of increasing axial load pulses and decreasing confining pressures, 50 000 load cycles applied at each stress state. \( q = \text{deviator stress}, \ p = \text{mean stress} (= \theta/3) \)
FIGURE 5: Plastic strain rate information obtained from tests on Quarriebras Qc material (saturated, high fines).
FIGURE 6: Permanent axial strains of triaxial specimens as a function of the dielectric value of the specimen’s top surface. Values of permanent axial strain > 2% indicate early failure of the permanent deformation test (see Fig. 4).
FIGURE 7  Variation of vertical and horizontal stress in the granular layer of pavements with various thicknesses of asphalt. Aggregate is Scottish Quarriebraes material in ‘saturated’, high fines condition (Qc).
FIGURE 8 Stress condition at various depths (*) beneath the top of the asphalt. Data is for the Quarriebras ‘dry’ moderate fines mix, Qa. Note only small q/p reduction 10 → 20mm asphalt. This is more marked for ≥ 40mm asphalt thickness.