Analytical Evaluation of Unbound Granular Layers in Regard to Permanent Deformation

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ABSTRACT: For the purpose of Low Volume Road (LVR) pavement design, computations of the stress strain-state in granular layers, when performed, are often simplified so that only linear elastic conditions are assumed and only the vertical stress at the top of the subgrade is assessed. With changing loading conditions on these roads – e.g. as a consequence of the introduction of Tyre Pressure Control Systems (TPCS) and super single tyres – more detailed analyses are required, so that their effect can be analytically assessed. This paper discusses an analytical method for evaluating the stress-strain condition in thinly surfaced or unsurfaced pavements typically used in LVR structures. It aims to improve the understanding of the effect of tyre pressure and contact area in regard to permanent deformation. To achieve this, several scenarios were modelled using the Kenlayer software varying aggregate material, thickness, stiffness, tyre pressure & arrangement. The results usually show a fairly well defined locus of maximum stresses. By comparing this stress envelope with failure envelope, conclusions could be established about the more damaging effect of super singles over twin tyres and, likewise, the greater damage inflicted by high tyre pressures compared to that incurred by lower tyre pressures. Additionally, when applying the design approach in the analysis of some Heavy Vehicle Simulator (HVS) test results available from literature the results show promising opportunities for the development of a simplified method of design of low volume roads against rutting.

1 INTRODUCTION

In unsealed and thinly sealed roads, the behaviour of the pavement is mostly dictated by the granular layers. In such structures, the aggregate is responsible for spreading the load received by the tyres to the road foundation. Depending on the modular ratio and thicknesses among the pavement component layers, the unbound granular layer(s) (UGL) and the subgrade will be responsible to bear varying proportions of the stresses imposed. According to the level of stress each layer will suffer, consecutive vehicle passes will cause certain damage, triggering permanent deformation to accumulate throughout the pavement’s life.

Growing use of new tools to aid transportation roads that are largely constituted of unbound granular material (UGM) – such as tyre pressure control systems (TPCS) – has motivated research about the difference tyre pressure would cause in regard to permanent deformation in UGL. The principle involved in TPCS is that by inflating (deflating) tyres, a smaller (bigger) “footprint” will cause a different tyre-pavement contact to occur as well as a different applied pressure on the pavement. This will directly affect the stress distribution in the pavement and, therefore, influence the permanent deformation induced.

TPCS is of growing use in some countries in the forest business (Douglas et al, 2003, Munro & MacCulloch, 2007). At lower pressures the same vehicle may cross a softer pavement without causing rutting or wheel spin. The technique also seems to result in much less tyre wear (Munro & MacCulloch, 2007) and, perhaps, fuel use. The drawback is that the vehicle cannot travel at speed on conventional pavements without safety concerns, thus requir-
ing the pressure to be increased to conventional levels in such situations. This system is credited in reducing the rutting damage in unsealed roads and also in improving ride quality for drivers.

In addition to TPCS, the increasing use of super single tyres over dual tyres in all range of goods transportation is also a reality. The reduced tyre size directly affects the pavement’s rutting rates, as the smaller contact areas have a more concentrated load with consequent increased damage rates.

This paper explains the derivation of a simplified method to assess the likelihood of pavement rutting when subjected to different loading conditions. An analytical method is explained that assesses both the impact of lorries equipped with TPCS running on dual tyres and super singles in regard to permanent deformation. To achieve this, 180 conditions were simulated using the Kenlayer software and results from while an HVS experiment in Finland provided data for validation.

2 PERMANENT DEFORMATION IN UGM

As rutting is the main distress mode in unsealed and thinly sealed pavements (Dawson et al, 2005), it is desirable that it be analytically approached rather than empirically as many design methods do (Austroads 1992, TRL 1993, AASHTO 1993); it is only by understanding each material’s behaviour that engineers will be able to anticipate the liability of a given structure, subjected to certain traffic and environmental conditions, to rut. If only empirical assessment tools are employed, then different conditions than those which provided performance benchmarks, could be never reliably accounted for.

Some design methods advocate that rutting only occurs within the subgrade provided that the unbound granular materials comply with materials specification. Hence, by limiting the vertical elastic strain at the top of the subgrade, it would be possible to assure a new pavement structure against rutting. This assumes minimally two things:

a) Recipe-based specification of UGM which typically include criteria for aggregate strength, durability, cleanliness, grading and angularity would provide some guarantee of a direct measurement of resistance to rutting.

b) The elastic properties of the subgrade material would have a direct relationship with its plastic properties.

Both of these conditions have very weak validity for granular and soil materials, if any. Such over-simplified methods for pavement design make it impossible to take full advantages of alternative materials and often provide a low reliability procedure.

Newer studies of plasticity in unbound granular materials for pavements such as Boyce (1980), Lekarp (1997), Werkmeister (2003), among others, have greatly advanced on the topic. Most of these models available for permanent deformation modelling are usually based on laboratory triaxial tests, what can be many times expensive tests for LVR projects.

Rutting can occur for a number of reasons, and basically follow four different types of contributory mechanisms, according to Dawson & Kolisoja (2004), namely:

- Mode 0 – Compaction of granular layers alone. No rutting in the subgrade. It is usually associated with inadequate compaction of the UGL during construction. It doesn’t represent a major threat to the pavement as it is usually self-stabilizing with traffic providing the missing compaction effort early in the layer’s life.
- Mode 1 – Shear deformation within the granular layer of the pavement, near the surface. A dilative heave adjacent to the wheel track is usually characteristic of this mode. It is largely a consequence of inadequate granular material shear strength and ideally there would be no deformation of the subgrade. It may lead to incremental collapse.
- Mode 2 – Shear deformation within the subgrade with the granular layer following the subgrade. This usually occurs in situations where aggregate quality is good. The pavement may rut as a whole and incremental collapse may result.
- Mode 3 – Particle damage (e.g. attrition or abrasion, perhaps by studded tyres) can be a contributor to the same surface manifestation as in Mode 0 rutting though, of course, the mechanism is very different.
- Combined mode – In practice rutting will be a mix of all previous modes. Each mode will have a certain contribution, although one mechanism may govern the overall rutting development.

As Mode 0 failure is self-stopping once adequate compaction has taken place and Mode 3 can be addressed by particle strength requirements, independent of the stress analysis, both this mechanisms have been discounted for this analysis. Hence, mechanisms of rutting such as Modes 1 & 2 are considered.

Permanent deformation could be accounted through a rational modelling of the plastic deformation generated by each vehicle pass throughout the pavement’s life or, in a more qualitative fashion, that would tell the likelihood of a specific structure to rut. This last rationale has been the option used in this study, as a means of promoting a readily accessible tool for engineers looking after LVRs. The proposed methodology still treats the pavement analytically, but permitting a more fundamental description of UGL behaviour than in simple linear elastic analysis but that simplifies elasto-plastic analysis for routine use, thereby reducing demands of material characterization and for computational skills.

3 ANALYSISYS

For the analysis of the proposed study three different materials were selected and modelled, using the Kénlayer software (Huang, 2003), totalling 180 different combinations of load arrangement, tyre pressure, unbound granular material, stiffness ratio between base (in this case the trafficked layer) and subgrade (semi-infinite layer) and the ratio between base thickness and loaded radius, which is a function of the tyre pressure. The three materials tested (Northern Ireland Good, or NIG, Northern Ireland Poor, or NIP, and CAPTIF material, or CAF) are detailed in Section 3. Figure 1 illustrates the experimental matrix.

The Ebas/Esub ratio represents the proportion between the resilient modulus of the base and subgrade while the “AggThick/LoadRadius” represents the ratio between the thickness of aggregate layer and loading radius. All the values were normalized so that the response of other structures could be easily interpolated from the results.

As is typical for LVRs, the UGL was taken to be the trafficked layer. The thickness of the UGL was 9.5-66.5cm and subgrade resilient modulus ranged from 45-350MPa. The rather extreme values for Esub and Ebas/Esub were chosen in order to allow interpolation of the results over a wide variety of situations. Although somewhat unrealistic at the highest values, to the values used allow the rationale of high stiffness materials to become evident.

Figure 1. Experiment matrix – total of 180 analyses
3.1 Materials

Three materials were selected from the University of Nottingham database (Arnold, 2004). All the necessary parameters for the analysis had already been previously assessed and are summarized in Table 1. The materials selected covered a range of likely behaviours expected for UGM.

Table 1 – Properties of Materials Analysed

<table>
<thead>
<tr>
<th>Name</th>
<th>Bulk Unit Weight</th>
<th>Mohr-Coulomb Failure parameters</th>
<th>Drucker-Prager (p-q stress space)</th>
<th>Shakedown range boundary A-B</th>
<th>Shakedown range boundary B-C</th>
<th>K-Θ Constants</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\rho_b$ (kN/m³)</td>
<td>c (kPa)</td>
<td>$\varphi$ (°)</td>
<td>d (kPa)</td>
<td>$\beta$ (°)</td>
<td>$d$ (kPa)</td>
</tr>
<tr>
<td>NI Good (NIG)</td>
<td>19</td>
<td>74</td>
<td>46</td>
<td>135</td>
<td>62</td>
<td>10</td>
</tr>
<tr>
<td>NI Poor (NIP)</td>
<td>21</td>
<td>27</td>
<td>46</td>
<td>49</td>
<td>62</td>
<td>65</td>
</tr>
<tr>
<td>CAPTIF2 (CAF)</td>
<td>22.8</td>
<td>0</td>
<td>61</td>
<td>0</td>
<td>68</td>
<td>0</td>
</tr>
</tbody>
</table>

Range A, B & C are fields of stress in which permanent deformation under repeated loading is stabilizing, incrementally increasing or de-stabilizing, respectively. The range boundaries are defined in terms of pseudo-Drucker-Prager surface values in p-q space; “d” being the q-intercept of the surface and $\beta$ the angle of the surface, both in p-q space.

NI Poor (NIP) is a material used for a trial in Northern Ireland (Arnold, 2004), is known to be of a poor quality. It has relatively low cohesion (c) and a poor angle of friction ($\varphi$). It has moderate stiffness characteristics but a low non-linearity such that stiffness does not increase much in highly stressed areas. NI Good (NIG) is a granular material of a similar stiffness 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 stress envelope is substantially larger.

CAPTIF material 2 (CAF) is a much cleaner aggregate with no cohesion, but a very high frictional behaviour and a stiffness that rapidly increases with additional imposed stress (highest $k_2$). Its shakedown ranges are similar to those of NIP, but has a much greater ultimate strength.

3.2 Loading Arrangements

The loading arrangements were those representatives of typical trucks on LVRs in the United Kingdom. Equivalent radii were obtained by distributing 45kN wheel loads over circular areas at the tyre pressure, circular loads being the Kenlayer input mode (Fig. 2). The distances used between tyres were based on measurements made in pavement trials in Scotland (Brito et al, 2008). Dual tyres are based on a typical tyre designation 295/80R22.5 and the “super singles” 385/65R22.5.

Figure 2 illustrates the radii and load position for all loading arrangements studied.

Although lorries fitted with TPCS may run with tyre pressures as low as 240kPa and as high as 900kPa, pressures of 400kPa and 800kPa were deemed to be closer to the operational values.

3.3 Methodology of computations

One advantage of the Kenlayer program (Huang, 2003) is its capability of treating UGMs as non-linear elastic using a k-θ model. This allows the computation of the elastic modulus of the layer according to the stress state it is subject to (due to external loading and geostatic forces).

Despite this benefit, the computational framework does not prevent the computation of tensile stresses which may lead to erroneous values at certain stress points. In addition, al-
though non-linearity is accounted for, each layer has the same stiffness properties throughout which, for the thicker layers, may result in some distortions.

The distortions caused by the computations of the tensile stress are minimized by the correction Kenlayer performs with the so-called “Method 3”, in which the negative or small horizontal stresses are modified according to the Mohr-Coulomb theory of failure, so that the strength of the material is not exceeded. This “method” is selected when the value of $\varphi$ is greater than zero and smaller than 90°. It also accounts for the geostatic stresses.

The output results were computed in terms of the stress invariants $p$ – the mean normal effective stress and $q$ – the deviatoric stress. The analyses were carried out by the following steps:

i. Stress and strain were computed at 360 points per structure.
ii. The ratio of deviatoric stress over mean normal stress ($q/p$) was calculated for each point analysed.
iii. If $q/p > 3$ or $q/p < 0$, the values were disregarded. For the first condition ($q/p>3$) this implies that a tensile stress was being reported, an impossibility for a UGM / UGL. The second condition ($q/p<0$) also indicates a negative stress being computed.
iv. $p$ and $q$ results were plotted and a polynomial of 6th order was fit to each stress plot.
v. The Drucker-Prager yield surface was then added to the plot and the proximity of the stress state to it was assessed by means of a variable, here called “S” and described further in the next section.

Figure 3 shows a typical stress plot resulting from the analysis performed. Figure 4 shows a plot in which the polynomial fits are displayed for two different cases: Dual tyres at 400kPa and Super Singles at 800kPa (CAF; Agg. Thick/Load Rad=1.3; Ebas/Esub=2).

4 RESULTS

Presenting all the results for the 180 analysis in this study would be cumbersome. As the main goal of the exercise is to promote a more qualitative study of the likelihood of the UGM developing permanent deformation, it is expected that the proximity of a certain stress state in a UGM to its yield failure envelope will provide a basis for the assessment of the rutting likelihood of rutting of the structure due to plastic deformation in the UGM.
Figure 3. Typical plot of computed stresses in pavement.

Figure 4. Example of two p,q plots. $S_{yield-surface-failure}$ in this plot is the same as $S_f$ in the text.
From previous studies (Dawson and Kolisoja, 2004) it is known that, if the stress states in the pavement are kept a long way from failure then no rutting in the granular layers will occur, but that if the stresses approach the static failure envelope, then the speed of development of rutting in the granular layer increases.

Hence, in order to simplify all computations performed, the stress variable S is introduced to provide a single parameter that will allow a comparison between the most damaging stress in the pavement and the corresponding limit stress state (Fig. 4). The line drawn from “p,q = (250,0) to (0,250)” was chosen as the basis for defining S as it intersects most of the loci of maximum stress points when those loci are at their closest to the failure envelope. The length (in kPa) of the line from p,q = (250,0) to the stress loci is then termed “S” and the length from p,q = (250,0) to the failure envelope is “Sf”. Different lines could have been selected, but this line seemed a reasonably reliable one for the purpose of assessing proximity to failure. In addition, after a careful study of the results, 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 sequence. This fact 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.

Arguably, the p,q line selected for this study as a reference parameter to assess proximity to the yield surface could be changed. Although the three materials investigated covered to some extent the differences in mechanical behaviour exhibited by aggregates traditionally used in LVRs, different p,q lines may be required to assess the S variable for other materials. Even though a certain amount of empiricism is introduced in the assumption of a “p,q = (250,0) to (0,250)”, the computations performed show this line to be a good reference point. Figure 5 presents a surface plot containing the results for all 90 exercises modelled using dual tyres. Figure 6 illustrates the results for super single tyres. It is evident from Figure 4 that the value of S is sensitive to wheel loading arrangements.

The S values for the failure envelopes, “Sf” in Figure 4, are evidently different for each material. For the three materials tested, the values are:
- NG = 297.1kPa
- NIP = 254.9kPa
- CAF = 251.8kPa

A “S” condition that approaches the Sf means that the pavement is likely to rapidly develop permanent deformation. In effect, there should be a limit to which these values could be considered acceptable and not. Potentially, two boundaries could tell if this “proximity” to failure is either far distance from developing permanent deformation (Range A behaviour, explained beneath Table 1), very likely to collapse after few loadings (Range C) or if it could develop a rutting acceptable for a certain number of vehicles passes (Range B). This follows the same criteria suggested by the Shakedown theory although considerably simplified.

Figure 5. S values (kPa) for the dual tyres at both pressures
5 VERIFICATION

Direct verification of the suggested design approach turned out to be challenging as the amount of suitable, well-documented experimental data available in the literature proved extremely limited. The main reason for this is obvious. Instrumented test sections are very seldom built using structures that are so weak that they are to be damaged under a small number of load repetitions. However, one source of data that was close to the intended application area of the design approach were some of the results from a test series performed using the Heavy Vehicle Simulator (HVS) at the Technical Research Center of Finland (VTT) (Korkiala-Tanttu et al. 2003). Even in these tests, however, the structure had some important differences from the typical very low volume / forest road type of structures mainly considered here (Fig. 7):

- the structure was covered with a 50 mm layer of asphalt concrete
- the total thickness of the aggregate layers on top of a sandy subgrade was 500 mm

In the HVS tests different sections of the test structure were exposed to 70,000 load repetitions 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 corresponding values of permanent axial strain in both upper and lower part of the unbound base course are presented in Table 2 together with the Mohr-Coulomb failure parameters determined by means of multi-stage monotonous triaxial tests and the respective value of $S_f$.
Table 2. Mohr-Coulomb failure parameters and the respective value of $S_f$ for the base course material, values of $S$, $S/S_f$ and permanent deformation after 70 000 load repetitions with different tyre arrangements.

<table>
<thead>
<tr>
<th>Mohr-Coulomb Failure parameters</th>
<th>Parameter</th>
<th>Dual wheel load</th>
<th>Tyre pressure</th>
<th>Parameter</th>
<th>$S/S_f$</th>
<th>Permanent strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c$ (kPa)</td>
<td>$\phi$ (°)</td>
<td>kPa</td>
<td>kN</td>
<td>kPa</td>
<td>%</td>
<td>Upper / lower part</td>
</tr>
<tr>
<td>43.0</td>
<td>43.1</td>
<td>267</td>
<td>70</td>
<td>850</td>
<td>234</td>
<td>87.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>50</td>
<td>700</td>
<td>225</td>
<td>84.3</td>
</tr>
</tbody>
</table>

For the rutting analysis of the unbound crushed rock base course the following assumptions and simplifications were made:
- the thin AC surfacing was considered as a part of the unbound base course layer
- the unbound sub-base course was combined with the sandy subgrade
- the stiffness ratio $E_{bas}/E_{sub}$ was considered as 2.0. Keeping in mind the highly stress-dependent stiffness in both the base and sub-base course/subgrade materials and the rapidly decreasing hydrostatic stress level alongside with increasing distance from the road surface, this may be considered as a reasonable assumption.

Then, by interpolating to the inflation pressure of 700 kPa for the 50 kN dual wheel, and slightly extrapolating to the inflation pressure of 850 kPa for the 70 kN dual wheel, and using the respective values of Aggregate Thick/Load ratio of 2.17 and 2.35, $S$ values of 225 kPa and 234 kPa were obtained (Table 2). As Table 2 indicates the accumulation rate of permanent deformations in the unbound base course rapidly increases alongside with the $S/S_f$ ratio. Even though the available data is too limited for firm conclusions to drawn, it seems that a critical limit value of the $S/S_f$ ratio for very rapid Mode 1 rutting (Range C) of the unbound base course to take place might be of the order of 90 % as has been suggested earlier (Dawson et al. 2007).

6 CONCLUSIONS

The analytical evaluation carried out seemed to produce reliable results and seems to be easy to use. Benefits and limitations could be summarized as follows:
- It is simpler than complex permanent deformation modelling that requires several input variables.
- It is neither time consuming nor requires major computational resources.
- It applies a logical rationale of permanent deformation development. That is, it suggests, that the proximity to a certain failure envelope is related to rutting likelihood.
- As the calculations demonstrated little sensitivity to the materials’ resilient properties (within the range evaluated), the proximity to failure could be simply computed by determining the $S_f$ parameter for a given material and comparing it to the $S$ value of the structure.
- It analytically verifies the UGL against rutting rather than by performing a simplified empirical assessment.
- simplifications are inherent in the computations due to the software used, in particular:
  - tensile stresses are manually filtered (by doing so, the software doesn’t recalculate the stress state of the pavement into a match where no tensile stresses exist; hence, there may be some minor inaccuracies in the results),
  - rutting is only considered in the granular material (Mode1). It is assumed that none comes from the subgrade (Mode 2). This assumption is in line with a study of LVR made by Tyrrell (2004) but could be unfounded on pavements with very thin structural layers.

The presented verification example gives at least qualitative support to the suggested design approach. It must, however, be recognised that so far the verification is based on a very limited amount data, because well-documented experimental results from rutting tests performed with low volume road type of structures are extremely scarce.
REFERENCES


