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Routes for exceptional loads: a new soil mechanics perspective

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Abstract

Offsite prefabrication can bring cost, quality and programme benefits to construction projects but often requires transportation of large, indivisible loads (in the order of 1,000-10,000 tonnes) on temporary routes which can cross soft soils. Through simple numerical modelling, this paper demonstrates the fundamental behaviour of the ground supporting these large loads can differ significantly from that expected in conventional road design practice; interaction between many closely-spaced wheels means the vehicle's influence depth and failure mechanism are significantly deeper. Surface soils are less influential.

Deeper soil is found to be more prone to local yield, developing large localised strains at low proportions (10-30%) of ultimate capacity. Instead of designing temporary roads to avoid yield and degradation under cyclic load, significant savings may be possible if limited degradation is permitted, with recovery through consolidation between loads. Investigation and monitoring of deep subsoils during operations is recommended for real-time evaluation of geotechnical risk.

Keywords chosen from ICE Publishing list

Pavement design, Roads and highways, Foundations

Symbols

B – loaded width (m)

D – pavement depth (m)

E – Young's Modulus (MPa)

E_0 – Young's Modulus at top of layer (MPa)

K_0 – coefficient of lateral earth pressure at rest (dimensionless)

$K_{0,NC}$ – normally consolidated coefficient of lateral earth pressure at rest (dimensionless)

$K_{0,OC}$ – overconsolidated coefficient of lateral earth pressure at rest (dimensionless)

m_1 – increase in Young's modulus with depth (MPa/m)

N_{Su} – shear strength ratio of clay subgrade, $S_u/\gamma'_p D$ (dimensionless) OCR – overconsolidation ratio (dimensionless)

p' – mean normal effective stress (kPa)

q – deviator stress (kPa)

s_E – settlement for linear elastic model (m)

s_p – settlement for Mohr-Coulomb model (m)

S_u – undrained shear strength (kPa)

x – lateral distance from vehicle or wheel centreline (m)

z_{NC} – depth to normal consolidation (m)

γ'_p – effective unit weight of pavement fill (kN/m^3)

γ_p – total unit weight of pavement fill (kN/m^3)

γ_s – unit weight of subgrade (kN/m^3)

Λ – utilisation of ultimate capacity, i.e. ratio of applied load to failure load, ω/ω_{ult} (dimensionless)

ν – Poisson's ratio (dimensionless)

σ'_z – vertical effective stress (kPa)

$\Delta\sigma'_z$ – increment in vertical effective stress from wheel load (kPa)

ϕ' – angle of internal shearing resistance (degrees)

ω – uniformly distributed wheel load stress on pavement (kPa)

ω_{ult} – ultimate ω at failure applied on a single wheel (or for each separate wheel in the case of multi-wheel models) in Mohr-Coulomb model (kPa)

1. Introduction

Demand for offsite prefabrication in the construction, mining and oil and gas industries (see www.mammoet.com/en/cases; Cronin, 2015) can necessitate transportation of large indivisible loads to remote locations on temporary roads. These loads may be in the order of 1,000-10,000 tonnes; transported by large, multi-wheeled mobile platforms (for example 6m wide by 60m long, supported on 88 wheels). Economic limitations due to the road's short design life, coupled with logistical constraints, may preclude the use of ground improvement where soft soils are crossed; in such extreme conditions the risk of cyclic degradation must be managed. Subgrade soil behaviour, under these unconventional conditions, is poorly understood and requires novel research for efficient road design.

Conventional pavements are designed to allow only a small amount of local yielding under each passage, otherwise cumulative strains render the pavement unserviceable. Unbound temporary roads need not be so robust (i.e. cumulative strains could be tolerated), but they must avoid deep-seated slip failure. This can form under repeated loads as localised yielding in the subgrade spreads. Understanding local yielding behaviour of heavy haul road subgrade soil is fundamental for successful design; this paper uses a parametric finite element analysis study to investigate how it varies with:

- dimensions of the vehicle
- proportion of capacity mobilised
- differences in pavement-subgrade system properties
- assumptions made in modelling soil behaviour

Simple analysis methods are purposefully used to allow yielding behaviour to be isolated and examined. It is demonstrated that the practice of minimising local yield, when applied to large vehicles, can be prohibitively expensive for temporary assets: a procedure which instead monitors real-time degradation may yield cost benefits. Implications for site investigation and monitoring of heavy haul roads are also discussed.

2. Literature review

2.1. Transmission of loads through pavements

2.1.1. Domain of conventional road research

Conventional pavement design aims to maintain low subgrade stress, well below its shear strength, by spreading wheel loads through an engineered fill layer (Brown, 1996; Frost et al., 2004). The resulting small strains are almost entirely recoverable; irrecoverable strain is only measurable after multiple cycles (Brown, 1996). Within this small-strain regime (i.e. <0.5% strain), elastic behaviour and constant post-cyclic shear strength can be assumed with acceptable accuracy (Díaz-Rodríguez and López-Molina, 2008; Wang et al., 2014). Tannant and Regensburg (2001) present a resilient modulus design method which prescribes mine road fill depths to specifically limit strains to this regime (1500 to 2000 microstrain). Subjecting subgrades to larger strains causes irrecoverable strain and degradation of shear strength (Brown et al., 1977; Wang et al., 2014).

Conventional pavement design is typically based on the response to passage of a single wheel (Sharp and Booker; 1984, Brown, 1996; Boulbibane et al., 2005), as wheels on conventional vehicles are considered to be sufficiently separated to avoid interaction between wheel pressure bulbs. Individual pressure bulbs from closely-spaced wheels can interact to form wider, deeper pressure bulbs (encountered by Gräbe and Clayton, 2009, when investigating deformation in railway

foundations, where sleepers are more closely spaced than conventional road wheel loads). Interaction between closely spaced wheel loads (or railway sleepers) also potentially changes the failure mechanism from being localised under a wheel to acting over the whole vehicle (Lehtonen et al., 2015). This interaction and deeper stressed zone must be considered in heavy haul road design.

Conventional pavement design generally presumes the upper subgrade layers are the most influential over development of progressive failure (Little, 1992; Brown, 1996). Even heavily loaded mine and forestry haul roads use this approach; pavement design thickness is often based on either the California Bearing Ratio (CBR) or the resilient modulus of the subgrade surface (Kaufman and Ault, 1977; Tannant and Regensburg, 2001; Forestry Commission England, 2011). For interacting, multi-wheeled vehicles the dominance of the upper, most overconsolidated layers, will diminish and deeper, normally consolidated, saturated soils will become increasingly influential.

2.1.2. Composite fill-subgrade system behaviour

A pavement is often described as a 'composite system' (Hyde, 1974; Brown, 1996; Frost, 2000); strains are considered compatible at the interface and stress distribution is influenced by the ratio of the pavement stiffness to that of the subgrade (relative stiffness). The role of engineered fill is primarily to spread the wheel load to keep subgrade stresses sufficiently low (Frost et al., 2004). Load on the subgrade is more efficiently distributed if pavement relative stiffness increases, but above an optimum relative stiffness the pavement shears locally, reducing the composite capacity (Sharp and Booker, 1984).

Deeper pavements spread loads on the subgrade more widely, but the load-spreading angle decreases until a limiting depth is reached where behaviour is completely controlled by the pavement (Sharp and Booker, 1984; Burd and Frydman, 1997; Houlby and Burd, 1999; Boulbibane et al., 2005). This effect has also been observed in scale foundation tests (Laman et al., 2012; Ismail Ibrahim, 2016).

2.1.3. Influence of principal stress rotation

A moving wheel load continuously rotates the principal stresses (Brown, 1996). Cyclic principal stress rotation softens soil, causing significantly larger cumulative strains (Arthur et al., 1980, Xiao et al., 2014; Jefferies et al., 2015). Cyclic principal stress rotation was found to result in accelerated failure (Gräbe and Clayton, 2009) when compared to a cyclic load that does not move (Brown and Chan, 1996).

Self-weight stresses are low beneath a conventional road (Brown, 1996); therefore lateral stresses arising in front of (and behind) a moving wheel will cause large rotation of the principal stresses. Overlapping adjacent stress bulbs will to some extent 'cancel out' the lateral stress pulse between adjacent axles, causing these lateral stresses to occur at greater depths where principal stress rotation from a given lateral stress increment is reduced as a result of the increased self-weight stress. It is possible two 'regimes' exist; a shallow zone where principal stresses are rotated significantly by the passage of traffic, and a deeper one where stress rotation is small. This is a complex stress-strain environment: the shallower, overconsolidated zone experiences rapidly changing stresses (both magnitudes and principal directions) whilst the deeper zone, more prone to compressive yield, experiences smaller stress changes but is still likely to degrade quickly.

2.2. Progressive failure and shakedown

A stable equilibrium ('shakedown'), whereby only recoverable strain occurs under load, is reached if the pavement-subgrade system can effectively become 'prestressed' by a residual stress field to remain within elastic limits both under load and at rest (Sharp and Booker, 1984; Ponter et al., 1985; Zhao et al., 2008). Residual stresses are assumed to be mobilised through limited local yield during initial load passages. This suggests an additional reserve of resistance to repeated load slightly above the load which causes first local yield, but significantly below the static load causing collapse (Pande, 1982; Sharp and Booker, 1984). Constant strength and stiffness parameters are typically assumed (Sharp and Booker, 1984; Zhao et al., 2008; Boulbibane et al., 2005), characteristic of small to medium-strain soil behaviour (Díaz-Rodríguez and López-Molina, 2008), to mathematically model a limiting safe load or 'shakedown limit'. N.b. the shakedown limit differs from cyclic threshold stress observed experimentally (e.g. Heath et al., 1972); the former considers the whole pavement-subgrade system theoretically whereas the latter indicates loss of strength of a single soil element once sufficiently large strains occur (e.g. Wang et al., 2014). Full scale pavement tests (Sharp and Booker, 1984) and scale laboratory experiments on a range of soils (Juspi, 2007) compare well to theoretical shakedown limits. This suggests residual stresses are the dominant factor in achieving equilibrium and material strengths only degrade for loads above the shakedown limit. Overconsolidated surface soils in particular are predominantly elastic when loaded well below their shear strength (Schofield and Wroth, 1968; Brown, 1996): the conventional approach of limiting subgrade surface strains is clearly valid for roads resisting predominantly small, separate wheel loads.

If the residual stress field required to counteract heavier loads cannot be achieved without yielding at rest, strain accumulates with each load application and the road fails progressively, typically accompanied by rising subgrade pore water pressures which reduce the strength (e.g. Frost, 2000, Gräbe and Clayton, 2009). Avoiding such degradation by limiting strains for large, multi-wheeled vehicles crossing soft ground, where stresses extend to great depth, is likely to require a much thicker pavement layer. Such an approach would likely be unjustifiably costly and impractical for a temporary road. A better understanding of strength degradation from load and recovery from consolidation in the medium/large-strain regimes, as discussed by Krechowiecki-Shaw et al. (2016), could achieve a more economic and practical design.

2.3. Local yield

As a surface load increases from zero towards the failure load, the first local yield is reached and plastic deformation occurs at a single point. Further increase in load causes more widespread yielding, redistributing stresses until the shear strength is mobilised over the full slip circle at failure (Osman and Boulton, 2005; Madabhushi and Haigh, 2015). In studying local yield, it is convenient to consider utilisation, i.e. the proportion of capacity mobilised (Λ , see Equation 1).

$$\text{Equation 1:} \quad \Lambda = \omega / \omega_{ult}$$

Elastic-plastic finite element modelling and corroborating field tests by D'Apollonia et al. (1971), indicate first local yield depends heavily on the *in-situ* stress state and shear strength. Both of these are determined by overconsolidation which can be quantified by N_{Su} (see Equation 2, after Burd and Frydman, 1997). For normally consolidated soil, this may occur at $\Lambda = 12\%$ to 25% , whilst local yield in heavily overconsolidated surface soils may only occur for $\Lambda > 50\%$ (D'Apollonia et al., 1971). Berardi and Lancellotta (2002) made similar observations related to oil tank settlements.

$$\text{Equation 2:} \quad N_{Su} = Su / \gamma'_p D$$

Burd and Frydman, (1997) also found N_{Su} to influence the failure mechanism; normally consolidated subgrades mobilised yield over larger volumes of soil, particularly developing yield in the compression zone directly beneath the load.

At rest, a normally consolidated soil has a lower K_0 and thus requires a smaller deviator stress increment to reach compressive yield. Whilst compressive yield in deep normally consolidated soil is not a common problem for conventional roads, below a heavy haul road it is a concern and is therefore investigated herein. This paper focuses on the development of local yielding in soft soil at depth below unbound heavy haul roads, rather than attempting accurate stress or settlement predictions. A number of simplifying assumptions have been made in the finite element analyses to allow a 'first-approximation' of the problem to be developed.

3. Analysis method

3.1. Modelling philosophy

Simple linear-elastic and linear-elastic, perfectly-plastic ('Mohr-Coulomb') models are used for the soil layers. Whilst being unrealistic descriptors of element-level behaviour, over a soil mass they can:

- 1) offer good agreement with yield development and failure mechanisms of site trials (D'Appolonia et al., 1971; Ismail Ibrahim, 2016).
- 2) allow study of local yielding effects in isolation (D'Appolonia et al., 1971).

Single wheel loads are compared with multi-wheeled vehicles to identify changes in behaviour dependent on interaction between wheels. The vehicle considered is long relative to its width and wheels are closely spaced in the longitudinal direction (Figure 1), thus the three-dimensional layout is simplified to a two-dimensional, plane strain analysis based on the section view. Whilst a single wheel is more accurately represented by an axisymmetric (circular) load, conventional haulage vehicles often have multiple axles with small longitudinal spacing; some longitudinal interaction is likely to occur, making the response more similar to a strip load, hence single wheel loads are also simplified to a plane-strain strip load. Sharp and Booker (1984), Boulbibane et al. (2005) and Juspi (2007) similarly analysed moving single wheel loads as plane-strain strips.

A range of subgrade models are simulated to represent normally consolidated to lightly overconsolidated, soft to very soft deposits with a high water table, i.e. areas with low bearing capacity that are likely to present difficulties to temporary roads, such as alluvial deltas. Soil parameters are chosen to reflect a low plasticity (PI = 10-15%) silty to very silty clay. Overconsolidation is represented using a simplified model, presented by Foye et al. (2008), in which the overconsolidated layer is modelled with a constant stiffness and shear strength, whilst deeper normally consolidated soil increases in stiffness and strength with depth (Figure 1). The properties of the pavement layer are also varied to investigate its influence on single and multiple wheel loads.

Whilst principal stress rotation is known to influence the soil stress-strain response (Jefferies et al., 2015), such effects are not modelled herein for the sake of simplicity and due to difficulty in obtaining realistic parameters. General trends in stress rotation are discussed, as this can have a large influence on the subgrade degradation environment. The MIDAS GTS NX finite element software was used for all finite element analyses.

3.2. Soil models

3.2.1. Linear-elastic models

Three separate linear-elastic clay subgrade models (representing varying degrees of overconsolidation) and a single granular pavement model, with properties as per Table 1, are analysed. Below the depth to normal consolidation, z_{NC} (Figure 1), the subgrade soil is assumed to be normally consolidated and hence the Young's Modulus is described by Equation 3:

$$\text{Equation 3:} \quad E = m_1 \cdot p'$$

The Case 1 subgrade model is such that z_{NC} coincides with the top of subgrade, to provide a theoretical minimum strength and stiffness.

The clay subgrade K_0 is assumed to take the normally consolidated value, following Brooker and Ireland (1965), (Equation 4). The granular pavement follows Jaky (1948), (Equation 5):

$$\text{Equation 4:} \quad K_{0,NC} = 0.95 - \sin(\phi')$$

$$\text{Equation 5:} \quad K_{0,NC} = 1 - \sin(\phi')$$

For the purpose of determining $K_{0,NC}$, for the low plasticity clay subgrade $\phi' = 30^\circ$ (BSI, 2015)

Several road thicknesses are considered, overlying each subgrade model. Subgrade effective stresses vary as a result, thereby changing z_{NC} (Table 2).

3.2.2. Mohr-Coulomb models

The granular pavement is modelled as drained material with zero effective cohesion and $\phi' = 40^\circ$ (unless otherwise stated as 32°), corresponding roughly with typical values for a granular sub-base and general earthworks fill respectively (Sharp and Booker, 1984; Burd and Frydman, 1997; BSI, 2015). Other parameters are unchanged from the linear-elastic models. The clay subgrade is modelled as undrained cohesive material, i.e. with $\phi' = 0$ and undrained shear strength set such that $[E/S_U = 1000]$, in agreement with typical values for low plasticity, normally consolidated or lightly overconsolidated clay (D'Appolonia et al., 1971; Jamiolkowski et al., 1979).

To calculate ω_{ult} , Strength Reduction Method (SRM) analysis was used with a nominal surface load. SRM reduces (or increases) strength parameters (i.e. S_u and $\tan\phi'$) by a SRM factor until equilibrium is met (MIDAS 2016). By modifying the surface load accordingly, a SRM factor of 1.0 is achieved, which is taken to correspond to the ultimate pressure. Bearing capacities of single material models (drained and undrained) obtained in this manner were compared to classical closed-form equations of Brinch Hansen (1970) and found to differ by less than 4%, confirming reasonable accuracy of the finite element model.

To improve computational efficiency, artificial symmetry is imposed through the centreline of the wheel or vehicle. This approach is commonly used in modelling single loads (Burd and Frydman, 1997; Ismail Ibrahim, 2016), but is not necessarily applicable to a multi-wheeled vehicle as an asymmetric slip may arise between groups of wheels. To test the applicability of this simplification for closely spaced wheels, a Mohr-Coulomb model with artificial symmetry about the centreline was compared to a full-width model. Differences in bearing capacity and surface settlement profiles were negligible (<1%). The failure mechanism was found to remain symmetrical up to 95% of bearing capacity. Above this load the solution struggled to converge and asymmetries noted are most likely as a result of convergence algorithms rather than an asymmetrical failure mode.

For multi-wheel models with 1.5m pavement depth, large, localised movement of nodes at the pavement surface adjacent to wheels caused slow convergence at low utilisations. By including a small effective cohesion of 5kPa over the uppermost 0.25m of the 1.5m thick pavement layers,

convergence times were improved. Comparison of bearing capacity and settlements indicate a negligible effect on global behaviour (Figure 2).

3.2.3. Mohr-Coulomb with overconsolidated earth pressures

The previous Mohr-Coulomb models only considered changes to strength and stiffness resulting from overconsolidation. The further refinement of including overconsolidated lateral earth pressures is incorporated here, allowing comparison of the relative effects of lateral earth pressures on the composite response. A constant K_0 is assumed from the base of the pavement to z_{NC} . Whilst the actual K_0 for overconsolidated soil varies from a maximum near surface to approach $K_{0,NC}$ at depth, using a mean K_0 over the layer produces reasonable correspondence with field response (Levenburg and Garg, 2014). The value of K_0 at the midpoint of the layer is used, calculated following Ladd et al. (1977), Mayne and Kulhawy (1982) and assuming an exponent of 0.8 (as per Burd and Frydman, 1997), (Equations 6 and 7).

$$\text{Equation 6: } \frac{\left(\frac{S_u}{\sigma_{z'}}\right)}{\left(\frac{S_u}{\sigma_{z'}}\right)_{NC}} = OCR^{0.8}$$

$$\text{Equation 7: } \frac{K_{0,OC}}{K_{0,NC}} = OCR^{\sin\phi}$$

Compaction of the granular layer will 'lock in' lateral stresses (Brown, 1996). Accordingly, analyses with varying K_0 in the granular layers were undertaken (using $K_0 = 1.0$ and $K_0 = 3.0$).

4. Modelling outcomes

4.1. Linear-elastic stress bulbs

A greater relative stiffness of the pavement (i.e. over a softer subgrade) distributes vertical stress more efficiently at a wider load-spread angle (Figure 3 and Section 2.1.2), particularly for thin pavements. The subgrade stress bulbs are also deeper for thin pavements, particularly with high relative stiffness.

Modelling the large multi-wheeled vehicle confirms that the closely-spaced stress bulbs beneath individual wheels join to form a resultant stress bulb on a scale of the whole vehicle (Figure 4). This acts similarly to a single load of the same width as the vehicle, meaning even the 1.5m deep pavement behaves as 'thin' in relation to the combined stress bulb, evidenced by the wide load-spread angle in the pavement and deep subgrade stress bulb (similar to a single wheel load applied to a thin pavement). Changes to relative stiffness and pavement depth do not significantly affect multi-wheel stress distribution patterns (Figure 4), although deeper pavements protect the subgrade from high localised stresses.

4.2. Local yield of Mohr-Coulomb models

The bearing capacity, ω_{ult} , improves at diminishing rates with increasing fill depth, eventually becoming limited by the capacity of the fill itself (Figure 5), in agreement with Burd and Frydman (1997) and Ismail Ibrahim (2016). Limited local yielding at the pavement surface is common to all models but is relatively small except at high utilisations (Figure 6). Local yield in the subgrade can be

identified when behaviour diverges from the 'pavement only' results. Normally consolidated subgrades show local yielding at lower utilisations, whilst overconsolidated subgrades show little local yield until larger utilisations, followed by rapid plastic settlement, similar to the findings of D'Appolonia et al. (1971). Differences are more pronounced for thinner pavements, indicating increased importance of the subgrade in these cases.

Single wheel analyses with differing subgrade strengths, groundwater levels and pavement depths exhibit similar behaviour if their N_{Su} ratio is similar (Figure 7); a deeper pavement or lower water table increases subgrade effective stresses, reducing the tendency for overconsolidated-like behaviour. Pavement depth itself also influences local yield; the magnitude of plastic settlement for a constant N_{Su} ratio reduces for a deeper pavement (compare Subgrade Case 1 results in Figure 6), but the tendency local yielding to begin at low utilisation (10-30%) is similar.

Load-settlement behaviour of multi-wheel analyses cannot be similarly normalised by N_{Su} at the base of the pavement (Figure 8). Following Foye et al. (2008), an influence depth for shear strength equal to one footing width (taken as the vehicle width, i.e. 6m) is used to compute N_{Su} . The resultant values are similar to those of normally consolidated single wheel models (Subgrade Case 1), indicating local yield behaviour is also determined by the size of the whole vehicle.

For single-wheel analyses, increasing plastic settlement coincides with shear stresses exceeding the subgrade shear strength in the compression zone at the top of the subgrade (Figure 9); as the load increases, the yield extent spreads laterally to the passive wedge zone. A similar tendency is apparent for the multi-wheel analyses, although yield occurs at low utilisations in the compression zone for all subgrade models, similarly to the normally consolidated single-wheel analyses (Figure 9).

4.3. Development of failure mechanisms

The form of the single-wheel failure mechanism is influenced by both the pavement thickness and N_{Su} . Thick pavements and overconsolidated subgrades develop extension strains in a passive wedge confined to the pavement layer, while thin pavements and soft subgrades tend to develop strain bulbs in the subgrade compression zone. Figure 10 shows progression of failure mechanisms with increasing load by indicating shear strains in excess of 1% indicating significant post-yield straining. Failure mechanisms extending into overconsolidated subgrades (i.e. through thin pavements, not localised to the pavement) are found to be smaller, similar to the findings of Burd and Frydman (1997).

The multi-wheel failure mechanism is governed by wheel load interaction: a single wheel load applied to a 1.5m deep pavement causes failure entirely within the pavement layer (Figure 10), but the multi-wheel failure is a deep global slip over the vehicle's full width (Figure 11). The large yielded volume of soil in the compression zone is also similar to the normally consolidated single-wheel response, validating the assertion that a whole-vehicle scale response is dominated by soil at depth and not the pavement layers (see Table 3 and Table 4)..

4.4. Influence of *in-situ* stress state

Burd and Frydman (1997) suggest lateral earth pressure coefficients have little effect on the ultimate bearing capacity; analysis from this study agrees. However, higher subgrade lateral earth pressures are found to reduce local yielding, in agreement with D'Appolonia et al., (1971). This is most pronounced for high overconsolidation and shallow influence depths (Figure 12), although the effect on load-settlement response is small when compared to that arising from changes in shear strength (Figure 6). For multi-wheeled models, the behaviour is less influenced by the upper strata and hence the impact of the upper overconsolidated layer is even less significant.

Increasing K_0 in the granular fill had a negligible influence on settlement. This may be due to the simplistic choice of material model, which assumes constant stiffness. More sophisticated models, e.g. Duncan and Chang (1970) or Wolff and Visser (1994), which account for stiffening under increasing mean normal effective stress and strain-hardening under increasing deviator stress, may indicate a greater influence of K_0 . This is important in understanding an unbound pavement response to a single wheel load (as suggested by Brown, 1996). It is likely such sophisticated modelling of the pavement is not as necessary for large, multi-wheeled vehicles, as the influence of the pavement layers on composite response is greatly reduced (Table 4).

4.5. Comparison of principal stress rotation

Significant rotation of the principal axes occurs within the pavement layers in front or behind a single wheel even at relatively low utilisations (Figure 13). Below the pavement, only small principal stress rotation occurs at low utilisation. This is more pronounced for weaker subgrade models, and can be attributed to two factors: firstly lower bearing capacity (and therefore lower wheel pressures for the same degree of utilisation) means smaller relative changes to the *in-situ* stress state. Secondly, the higher relative pavement stiffness causes greater load spreading (Figure 3), reducing principal stress rotation in the subgrade (Figure 13). Inclination of principal stresses reduces with depth, as self-weight stresses become dominant.

For the multi-wheel model at low utilisation, the subgrade principal stresses show little inclination, which reduces further with depth and increases with increasing utilisation. At high utilisation (80%), a zone of principal stress reversal is apparent directly beneath the pavement (Figure 14) as the passive part of the failure mechanism is mobilised. This coincides with development of large plastic strains throughout the entire compression zone and yield being initiated in parts of the extension zone (Figure 11).

5. Practical considerations for heavy haul roads

Simplified modelling indicates the response of an unbound pavement-subgrade system is fundamentally different when subject to loading from a large vehicle with many closely spaced wheels rather than a single wheel. The influence of the pavement layer and subgrade surface, dominant for the single wheel case, is reduced; behaviour is more strongly influenced by soil at depth (see Figure 4, Figure 8 and Table 4). Designing these roads by investigating the subgrade surface only, and specifying a granular layer thickness to minimise irrecoverable subgrade strain from single, separate wheel loads, as is done for conventional roads, is thus inappropriate.

A different site investigation and design approach is necessary for roads carrying large, multi-wheeled vehicles. Surface tests, such as the commonly used CBR test, dynamic probing or *in-situ* surface stiffness tests (Frost, 2000), will still be useful in understanding resistance to rutting on the scale of a single wheel but less useful in understanding the behaviour of deeper soils when exposed to repetitions of multi-wheeled vehicles. The risk of degradation on a whole vehicle-scale mechanism will require investigation to greater depths using investigation techniques more common to design of large foundations, such as percussive drilling or Cone Penetration Tests. Furthermore, monitoring of the pavement surface for rutting may not give an indication of deeper-seated strain development, which is likely to manifest over a larger area. Under large strains close to or exceeding yield, excess pore water pressures are expected to accumulate in the subgrade; monitoring pore water pressures at depth via piezometers in boreholes may therefore be more effective.

Finite element results suggest a large vehicle will generate local yielding at much lower utilisations (Figure 8, Figure 11). As shakedown theory indicates cyclic failure is expected at loads slightly in excess of those causing first local yield, a conventional design which aims to avoid cyclic degradation would need to limit the extents of yielding, either through a very deep pavement or high earthworks to spread transient stresses sufficiently, or by strengthening the ground with large-scale ground improvement. Both of these options are unlikely to be economically viable for a temporary road.

The excess pore water pressures generated by a single vehicle passage may initially reduce the strength of the subgrade, potentially making it unsafe for another vehicle to pass. However, consolidation of the subgrade is expected to increase its strength over time. By taking this into account, an economic observational design, similar in philosophy to that of the Cape Kennedy causeway to transport Apollo mission rockets described by Peck (1969) could be achieved; vehicles only traverse once the subgrade strength has been sufficiently recovered. For this approach, the following aspects of the subgrade behaviour, discussed further in Krechowiecki-Shaw et al. (2016), need to be understood:

- The rate at which consolidation occurs: monitoring *in-situ* pore water pressures during operations will be more effective than estimation from laboratory tests on recovered samples and provides a real-time indication of risk.
- Strength recovery as a function of excess pore water pressure dissipation: advanced laboratory testing will be required to determine the degree of consolidation necessary to fully recover the soil's initial strength. This can then be used as a trigger level to allow passage of the next heavy vehicle.

If this approach is used, large strains are likely to develop (at least initially); topping up of the pavement surface may be necessary to maintain design alignment.

An additional complication arises if this design approach is used to mobilise high proportions of capacity and the compression zone of the failure mechanism fully yields (Figure 11). Resistance to a failure mechanism is now provided by the 'passive' extension zone away from the loaded area. Not only is stability wholly dependent upon the strength of the extension zone being maintained; the greater rotation of principal stresses (Figure 13) will accelerate cyclic degradation. The response of soil to such an extreme combination of actions is not well understood and further research into this is merited.

6. Conclusions

The simple material models used in the finite element analyses presented are easy to understand and apply and the effects of yield can be isolated and investigated. It is recognised these models are not representative of real soil behaviour, which often exhibits features such as strain- and pressure-dependent stiffness. Further investigation, isolating the effects of these properties, will be useful in developing a more realistic picture of a soil's response. However, the modelling undertaken in this paper clearly indicates the composite pavement-subgrade response for a multi-wheeled large vehicle is fundamentally different to that of a conventional vehicle and thus design must consider the following:

- Interaction between wheels results in deeper soil, which is closer to a normally consolidated state and more prone to local compressive yielding, being mobilised and thus far more influential. The composite system responds on a whole-vehicle scale, similar to the response of a single wheel load applied to a thin pavement. Local yielding behaviour can be described by assuming an influence depth equal to the vehicle width, as proposed by Foye et

al. (2008) for shallow foundations. The pavement and upper overconsolidated subgrade layers are less influential for large, multi-wheeled vehicles.

- Principal stress rotation from moving wheel loads is known to significantly accelerate degradation. This will complicate the degradation regime under a large, heavy vehicle; the soil at depth is expected to be less affected by stress rotation than surface soils due to the higher self-weight stresses. However if loads close to the static bearing capacity are transported, a passive wedge begins to mobilise, resulting in significant principal stress rotations extending to depth. As the passive wedge provides the final restraint against failure, rapid strength degradation here could result in sudden collapse. Further research into soil under such extreme conditions is required.
- Limiting the subgrade to small-strain behaviour to avoid degradation under repeated transient load may be unfeasible for a temporary road; the large factors of safety necessary, in the region of 3 to 10, may not be justifiable economically.
- Conventional investigation and design, whilst useful for designing pavement layers to resist rutting along wheel tracks, is governed by the properties of the subgrade surface and does not give adequate understanding of deeper-seated risks.

In contrast to the conventional design method, it is recommended that deep investigation (similar to that for large shallow foundations) and in-service monitoring of the deep subsoils is conducted to understand the real-time risk of strength degradation. A design which allows repeated application of medium-to-large strains is expected to result in significant changes to the subgrade; if the interaction between degradation and consolidation-related strengthening over time is understood and risks well-managed, significant construction cost benefits are possible.

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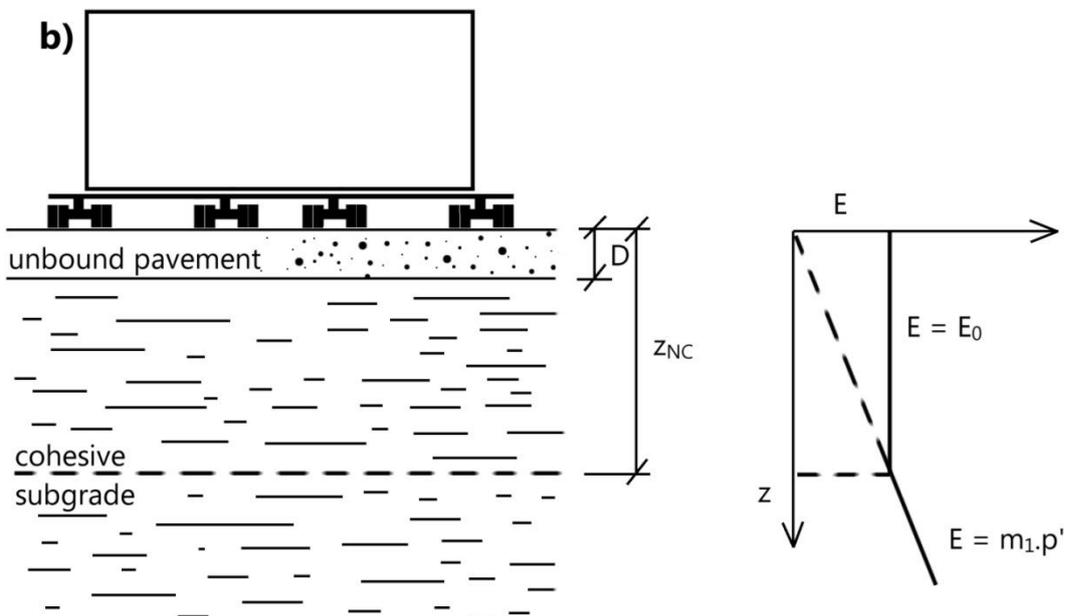
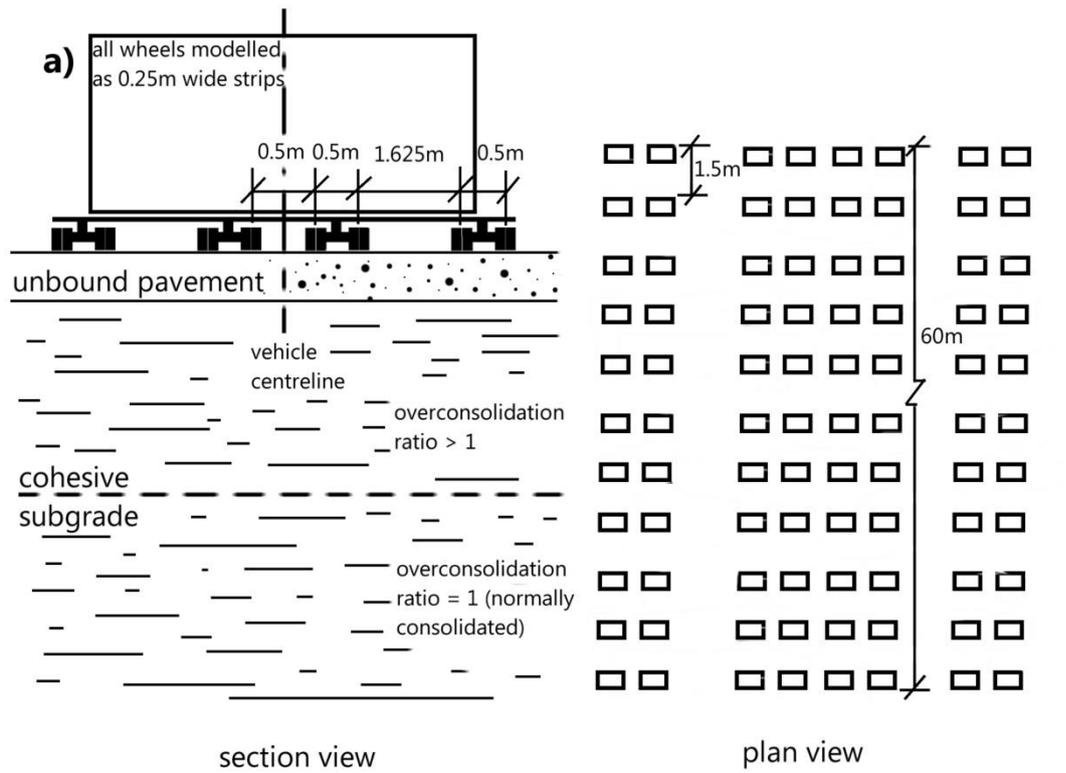


Figure 1: Problem definition: a) vehicle geometry and b) ground model for analysis. N.b. as a plane strain analysis is used, the model geometry is based solely on the section view; plan view is for information only.

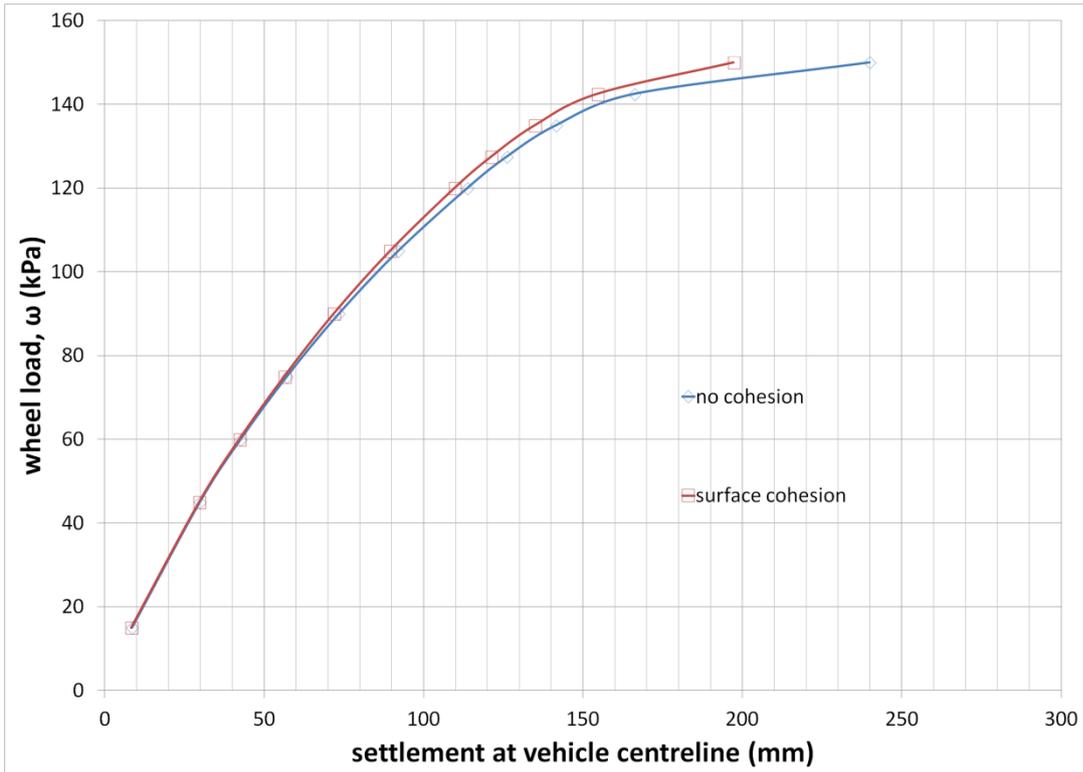


Figure 2: Load-settlement response at centreline of multi-wheel Mohr-Coulomb model for a 1.5m total pavement depth, showing impact of adding cohesion to the uppermost 0.25m of the pavement layer.

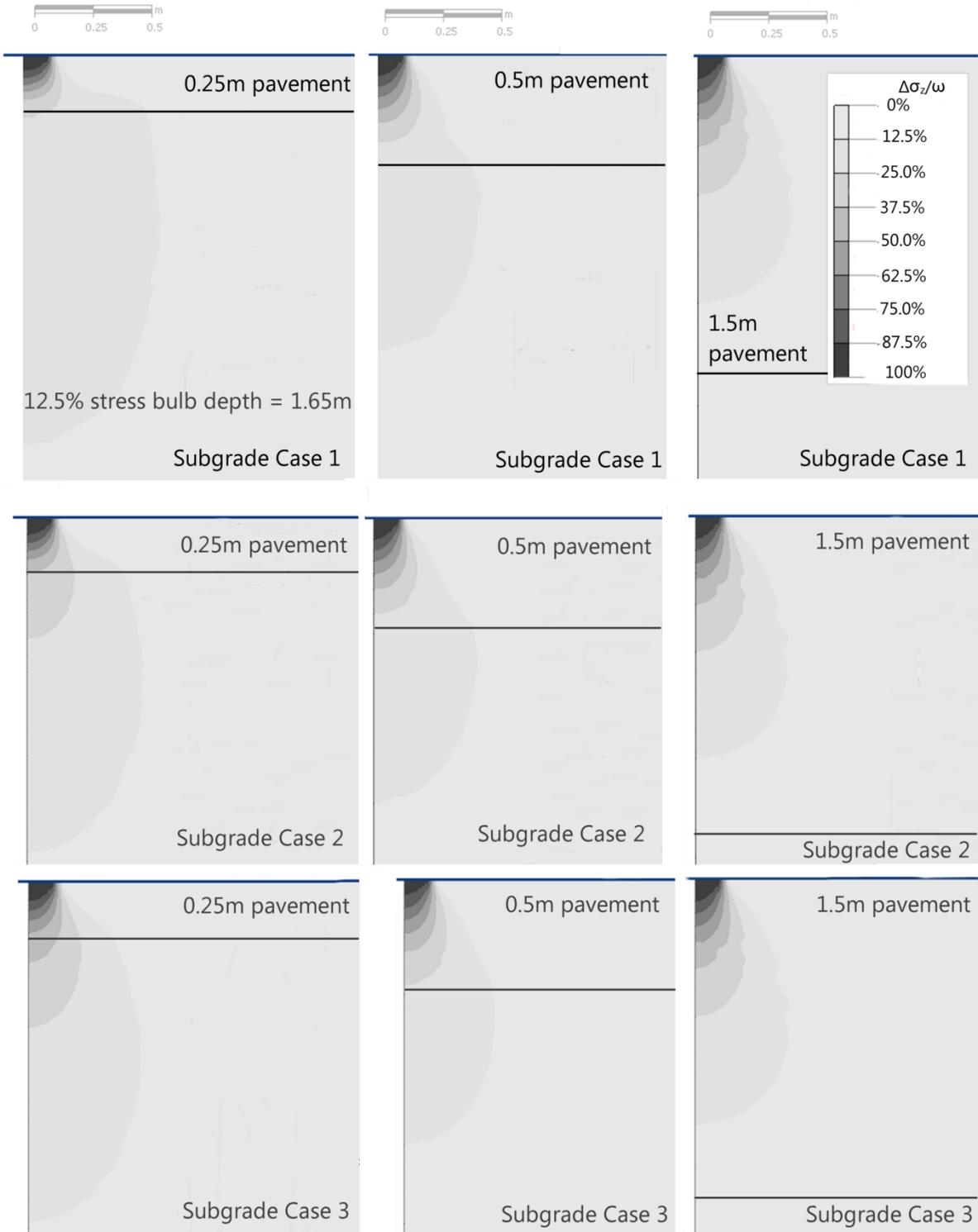


Figure 3: Increments in vertical stress, expressed as a percentage of wheel load pressure ω , from a 0.25m wide surface strip load with Mohr-Coulomb models of varying subgrade stiffness and pavement depth.

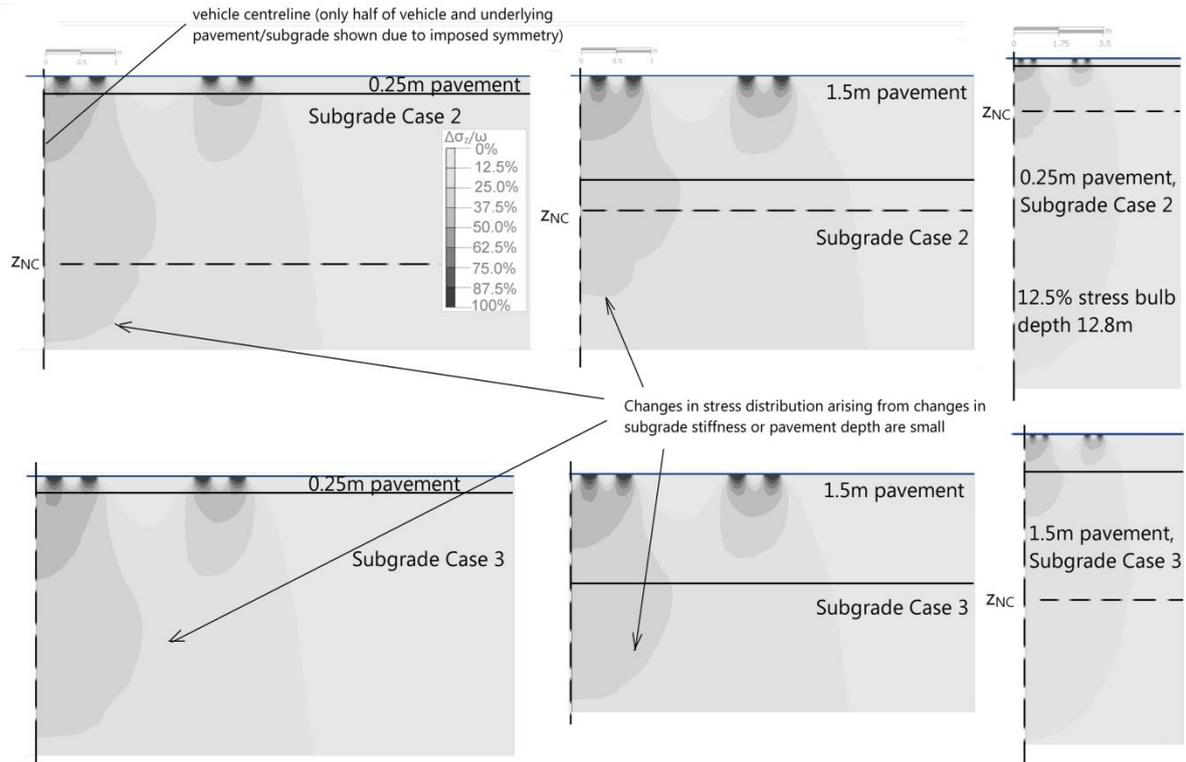


Figure 4: Increments in vertical stress, expressed as a percentage of wheel load pressure ω , from a multi-wheeled vehicle with Mohr-Coulomb models of varying subgrade stiffness and pavement depth. Left and centre – interaction between individual wheels. Right – Large scale stress bulbs over the whole vehicle width, extending to great depth, similar to the 0.25m pavement depth, Subgrade Case 1 stress bulbs in Figure 3.

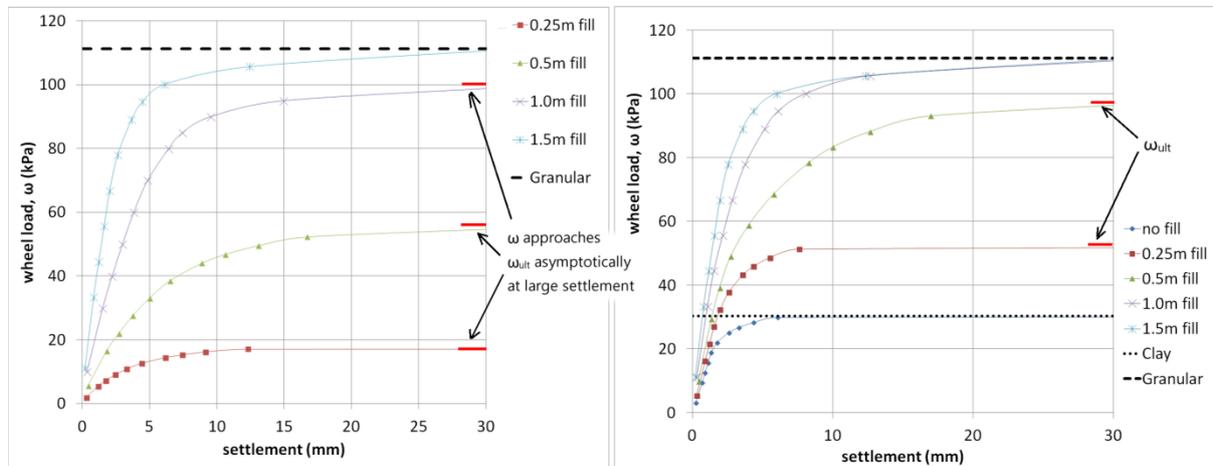


Figure 5: Load-settlement response for single-wheel models with varying pavement depths. Dashed lines indicate the closed-form capacity of the subgrade and pavement fill, calculated using the equations of Brinch Hansen, 1970). Left - Subgrade Case 1. Right - Subgrade Case 2.

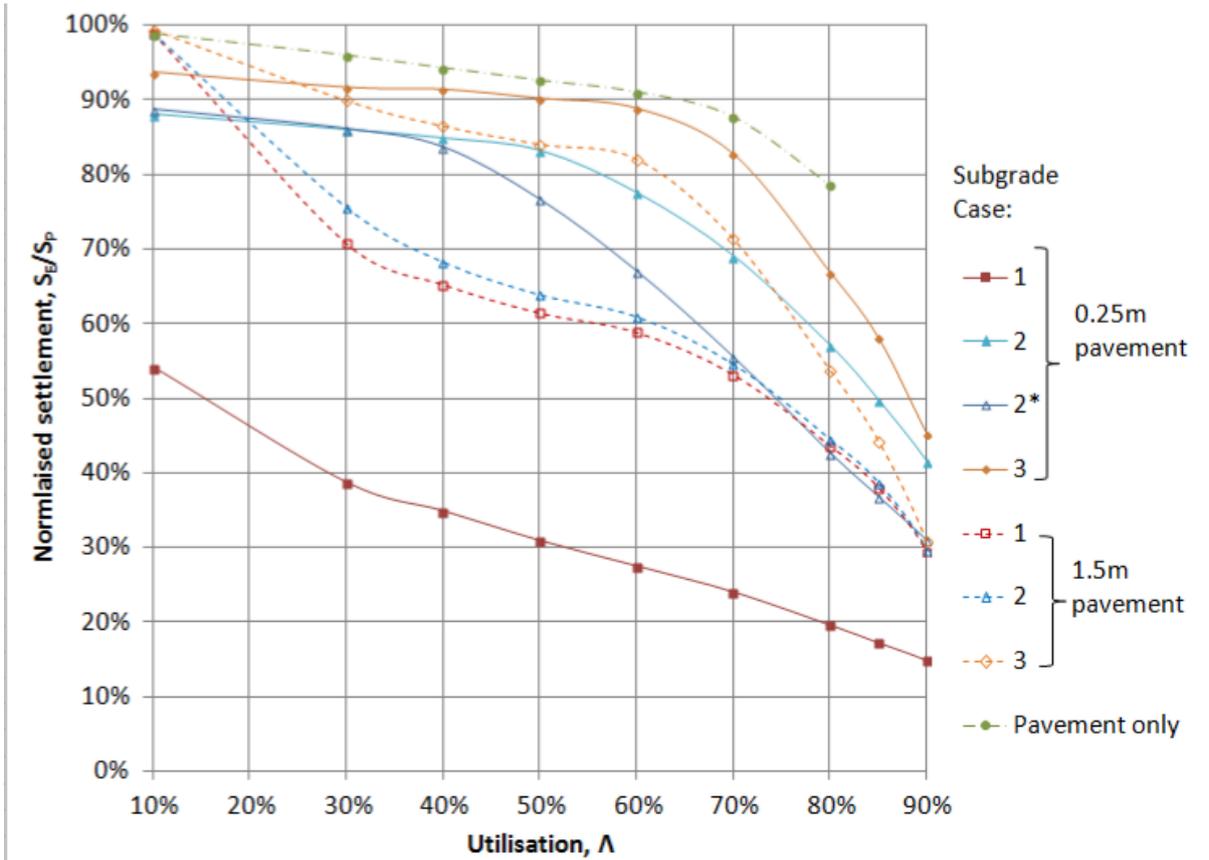


Figure 6: Normalised settlement vs utilisation plots for a range of single wheel Mohr-Coulomb models. A normalised settlement of 100% indicates no yield (fully elastic); decreasing normalised settlement indicates increasing yield. N.b. * = groundwater level at base of pavement.

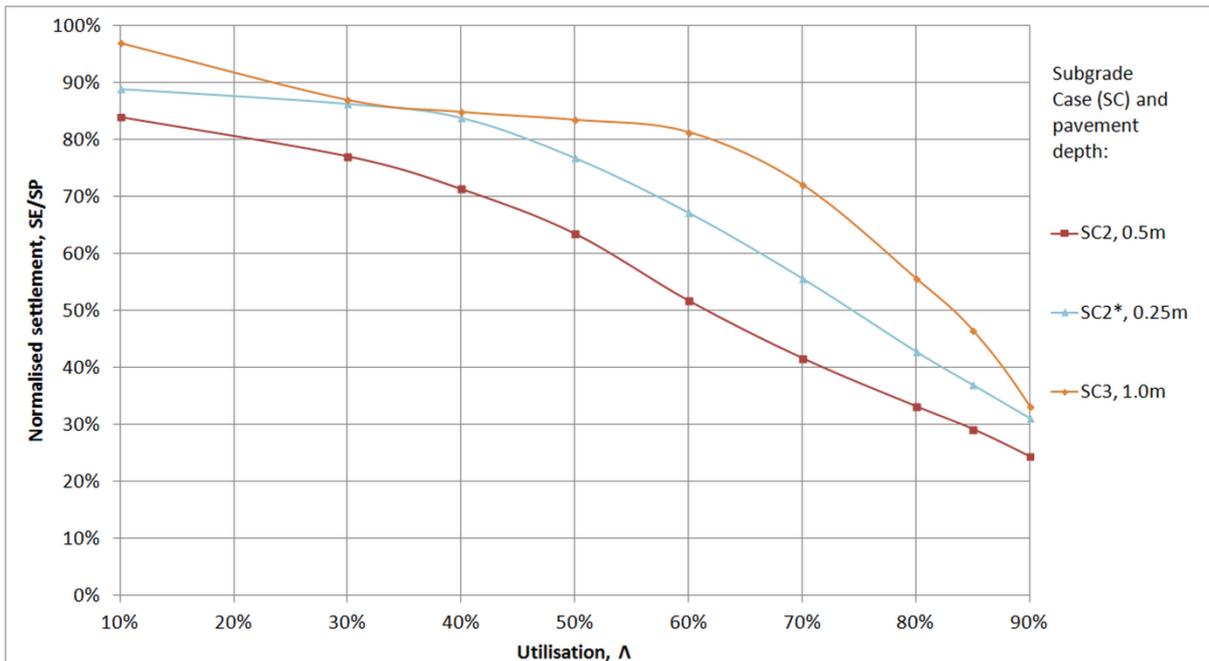


Figure 7: Normalised settlement vs utilisation plots for single-wheel Mohr-Coulomb models with similar N_{Su} ratios (n.b. the N_{Su} values are 1.06, 1.13 and 1.36 respectively). N.b. SC = Subgrade Case, * = groundwater level at base of pavement.

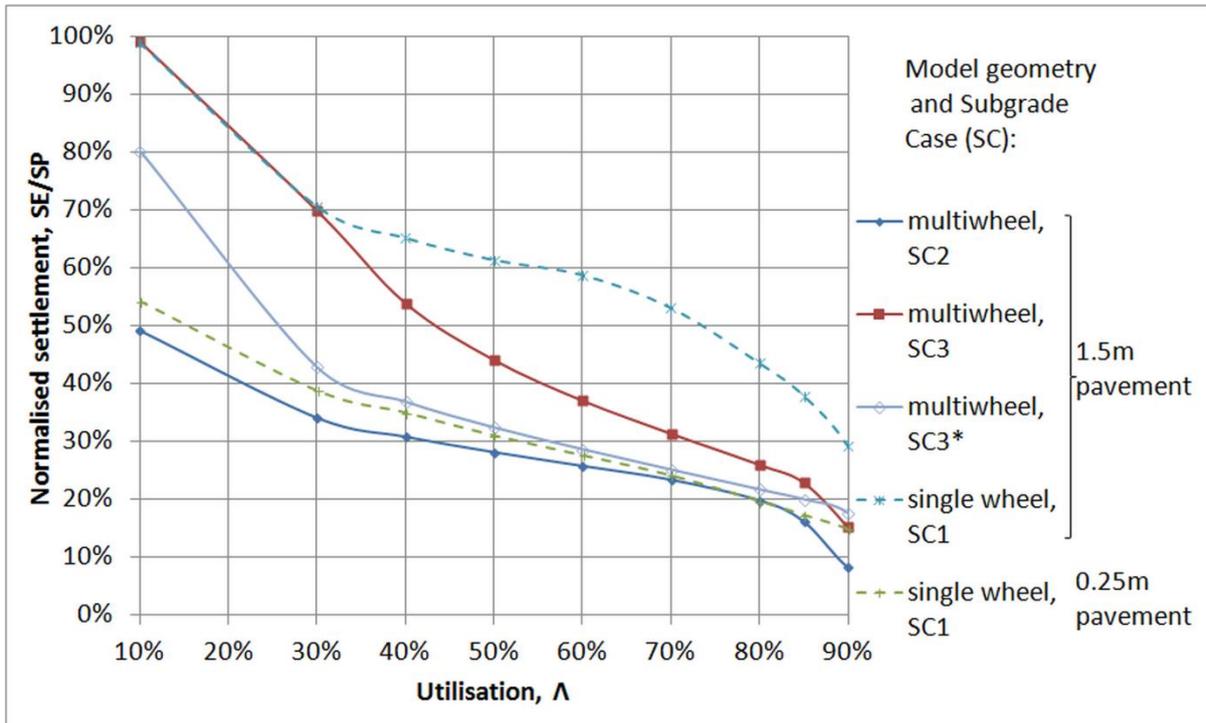


Figure 8: Normalised settlement vs utilisation plots for multiple-wheel Mohr-Coulomb models with varying subgrade models. For comparison, normally consolidated (Case 1) subgrade single-wheel models are also shown. Note that the shear strength and stiffness at the influence depth of 6m is the same for Subgrade Cases 3* and 2. N.b. SC = Subgrade Case, * = groundwater level at base of pavement.

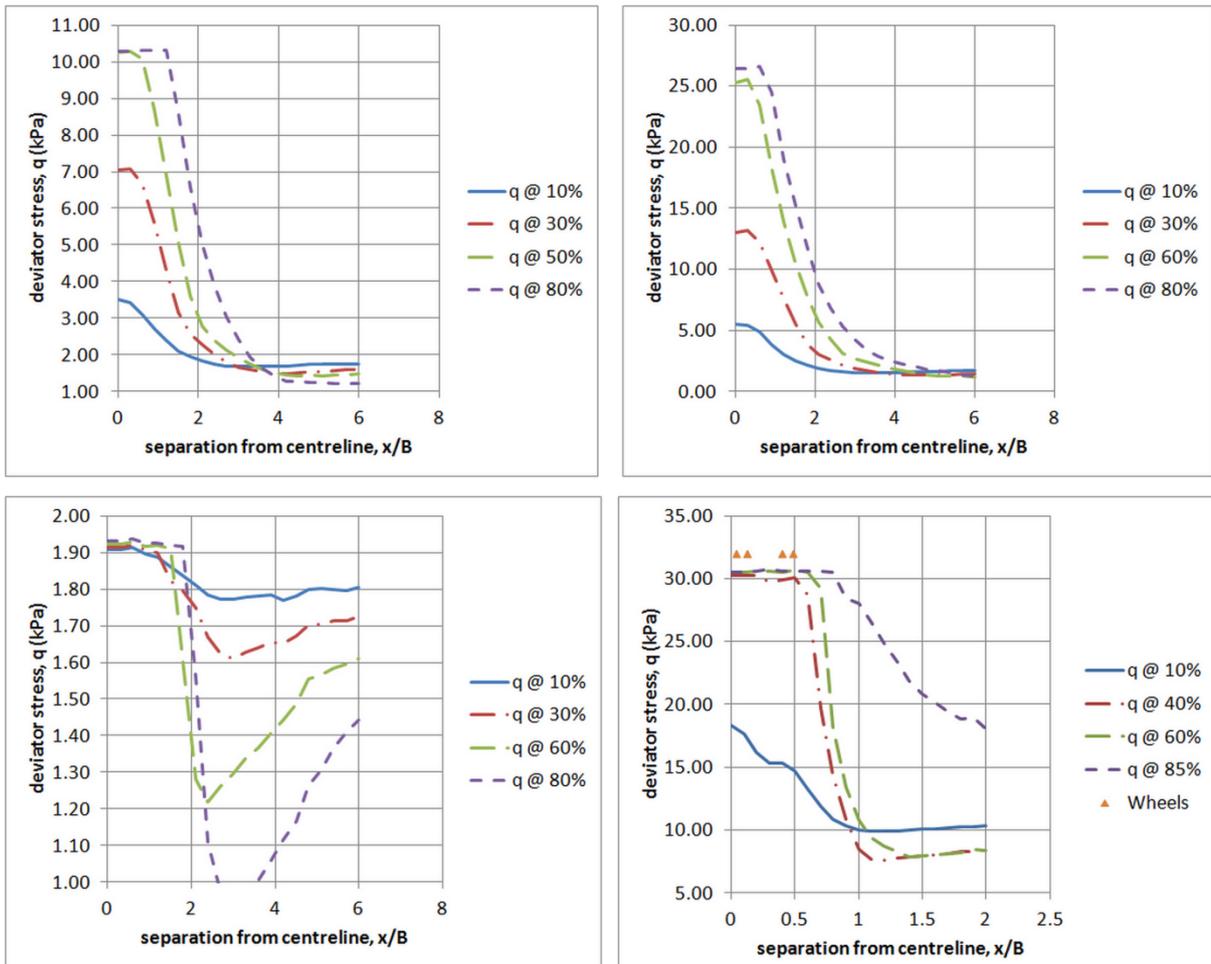


Figure 9: Deviator stress (n.b. equal to shear stress $\times 2$) at base of pavement layer for various degrees of utilisation (Δ). a) single wheel, Subgrade Case 2. b) single wheel, Subgrade Case 3. c) single wheel, Subgrade Case 1. d) multi-wheel, Subgrade Case 3. a) – c): pavement depth = 0.25m, $B = 0.25$ m d): pavement depth = 1.5m, B assumed to be 6m (whole vehicle width).

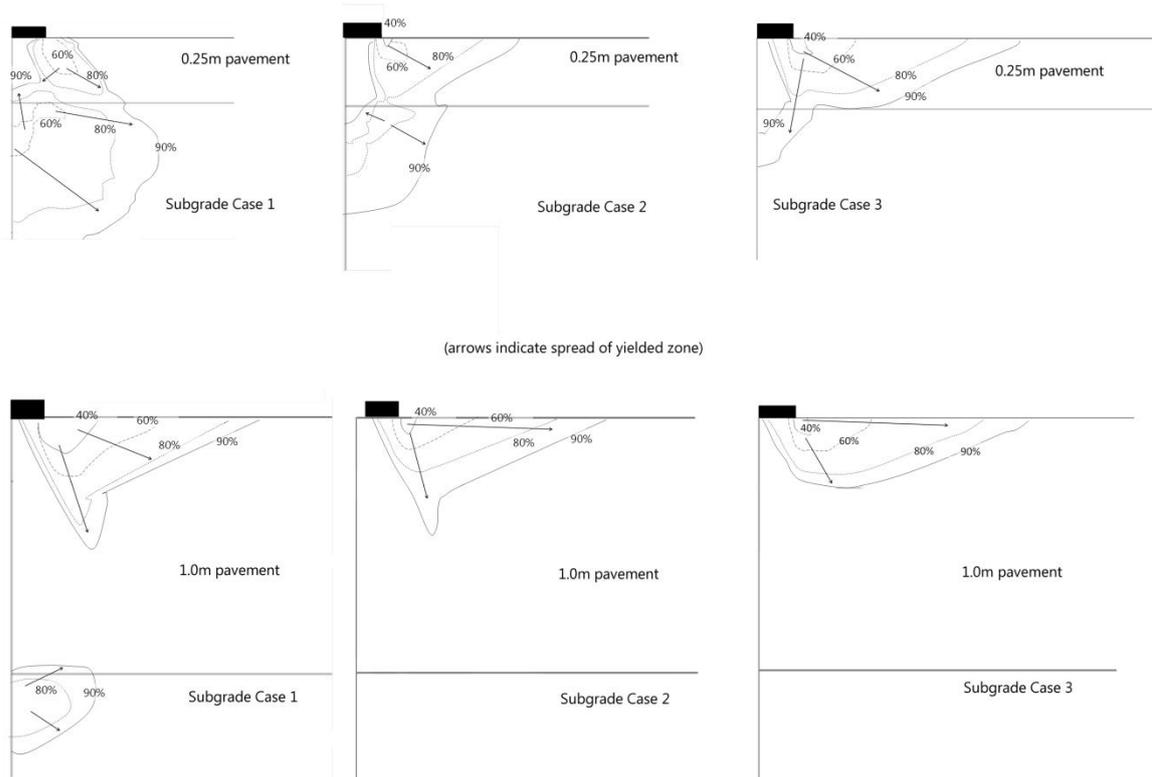


Figure 10: Progression of failure mechanisms indicated by 1% shear strain (i.e. large post-yield strain) contour for various degrees of utilisation (percentage values) for single-wheel Mohr-Coulomb models.

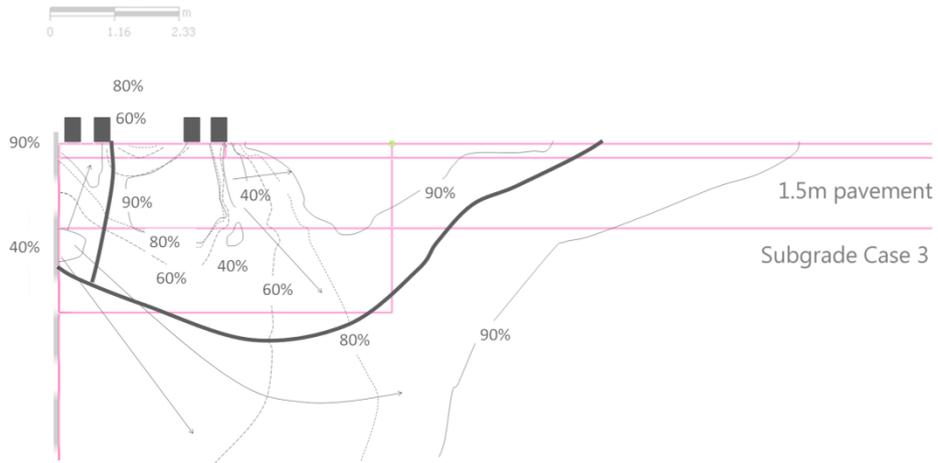
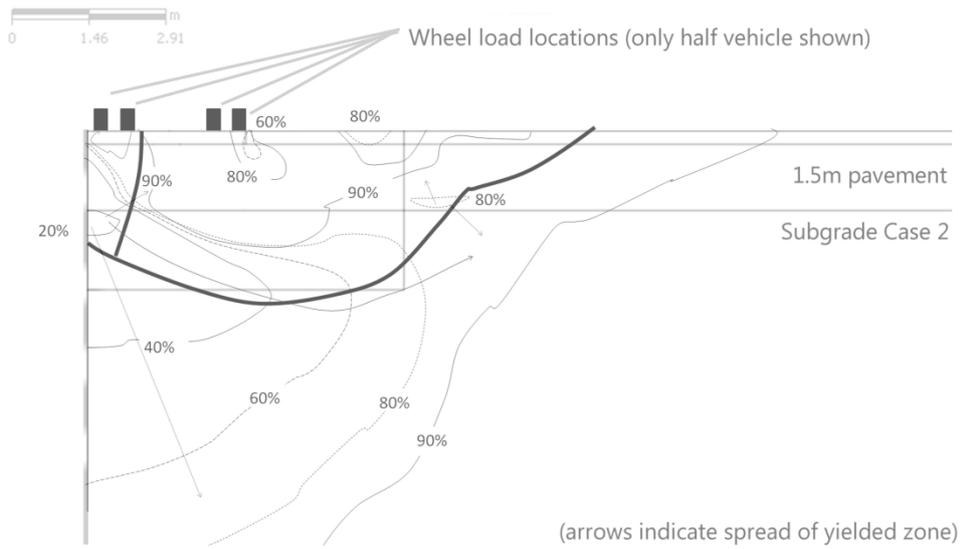


Figure 11: Progression of failure mechanisms indicated by 1% shear strain contour for various degrees of utilisation (percentage values) for multi-wheel Mohr-Coulomb models. The thick line follows the maximum shear strains developed at 95% utilisation, i.e. is approximate to the final failure mechanism.

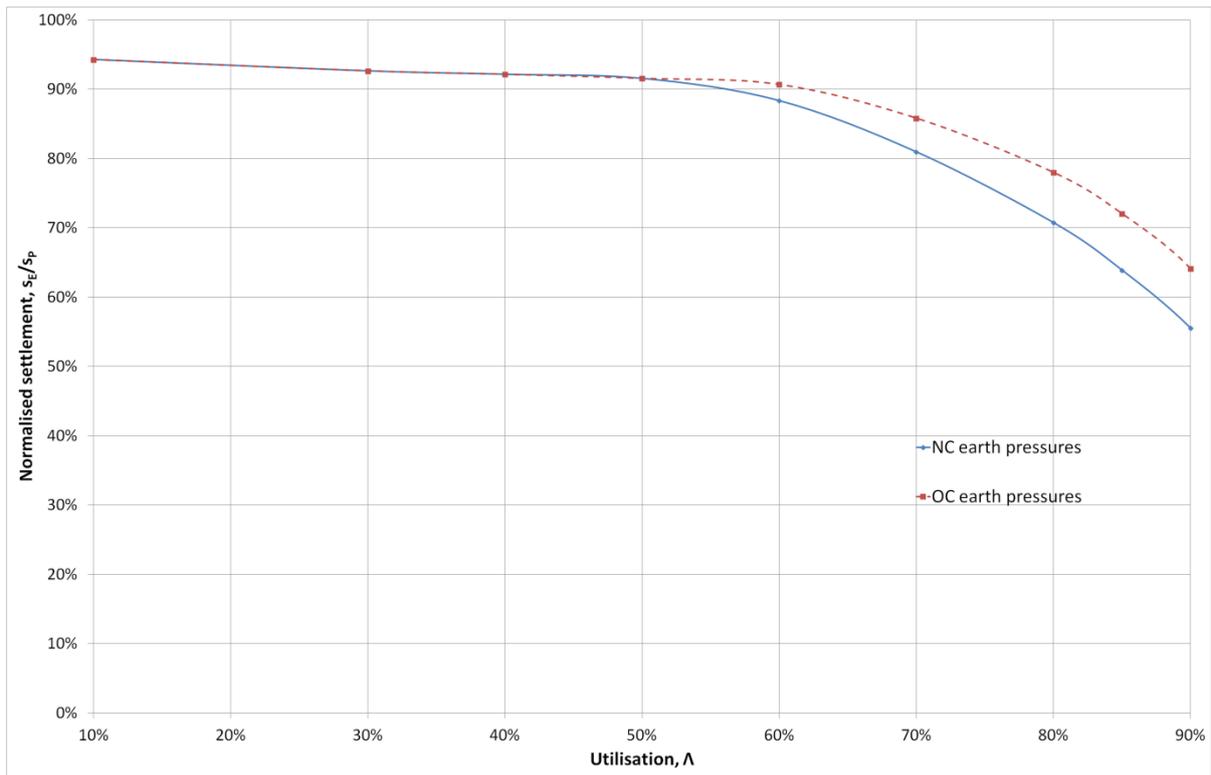


Figure 12: Normalised settlement vs utilisation plots for Mohr-Coulomb single wheel models of Subgrade Case 3* (i.e. groundwater at base of pavement) with pavement depth of 0.25m and varying coefficients of lateral earth pressure in the overconsolidated portion of the subgrade layer.

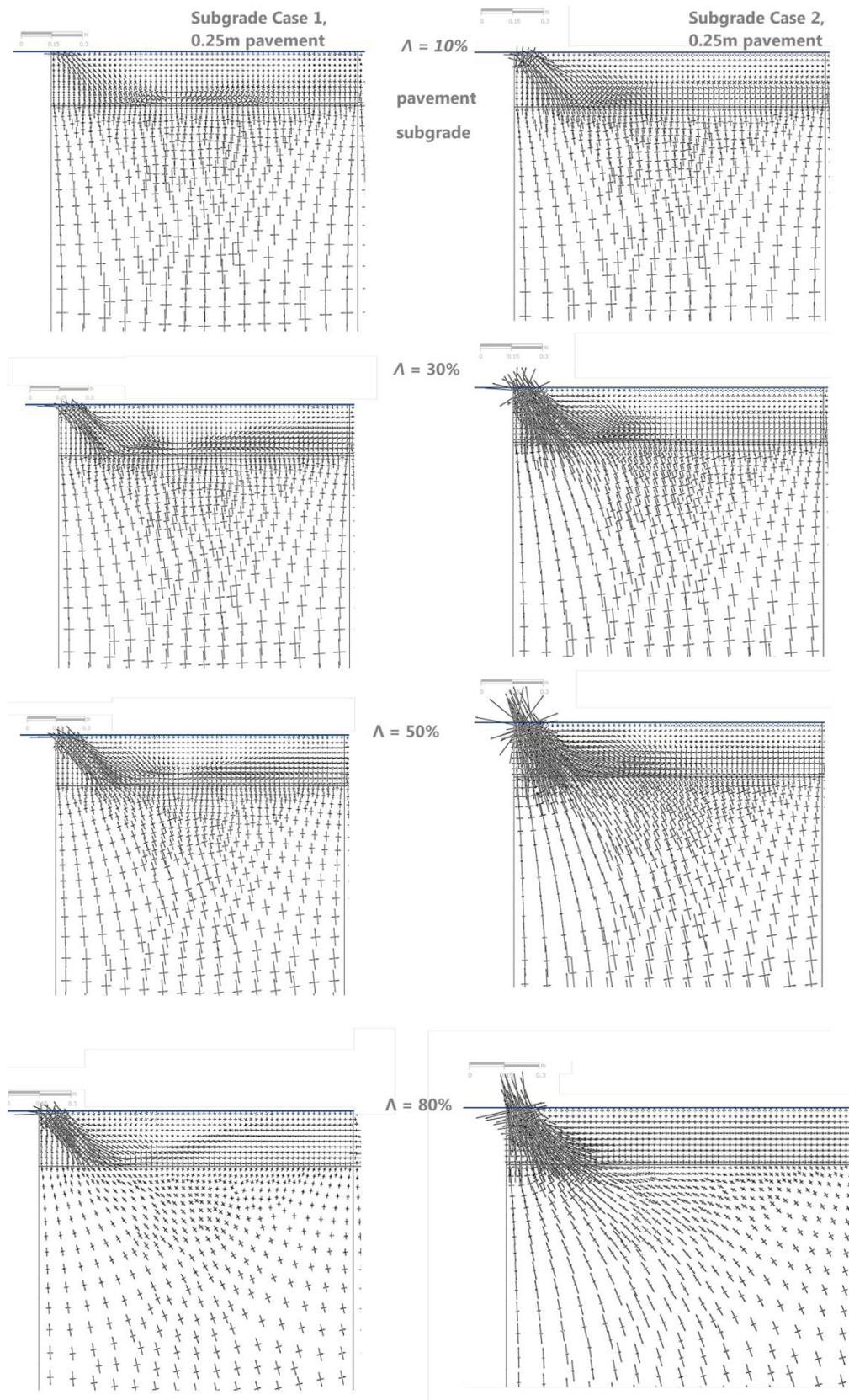


Figure 13: Principal stress vectors (i.e. line length = stress magnitude, rotation = stress direction) for single-wheel Mohr-Coulomb models with varying utilisation (Λ). N.b. where the vector is aligned horizontally and vertically but the horizontal vector is the larger, this indicates a rotation of 90° , i.e. full principal stress reversal.

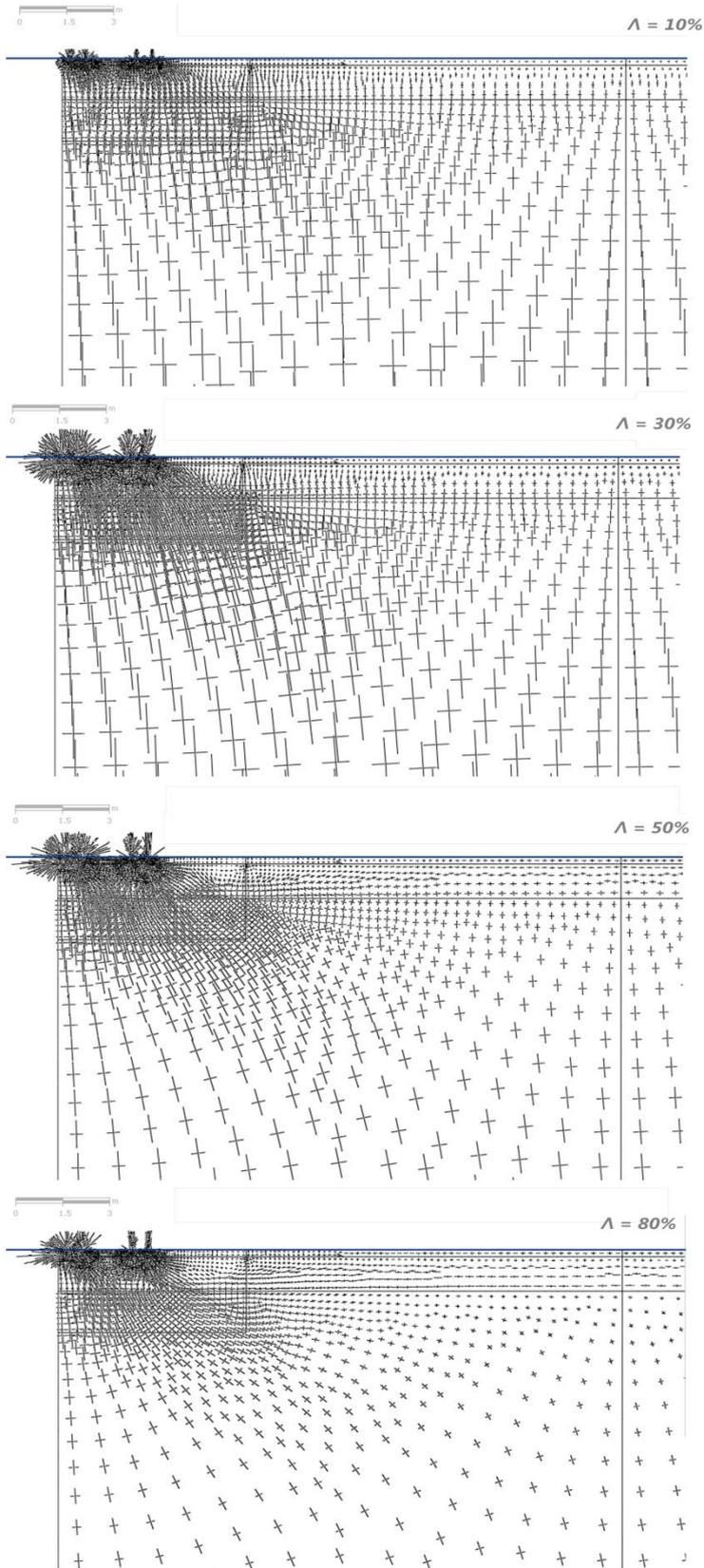


Figure 14: Principal stress vectors (i.e. line length = stress magnitude, rotation = stress direction) for multi-wheel Mohr-Coulomb model with Subgrade Case 3 and a 1.5m pavement depth, with varying utilisation (Λ). N.b. where the vector is aligned horizontally and vertically but the horizontal vector is the larger, this indicates a rotation of 90°, i.e. full principal stress reversal.

List of Tables

Table 1: Parameters assumed for linear-elastic and Mohr-Coulomb models: subgrade models are based on normally consolidated or lightly overconsolidated, soft to very soft alluvial clay of low plasticity (i.e. PI = 10-15%). $m_1 = 457$ MPa/m (i.e. $0.001m_1 = 0.457$ kPa/m).

Material	Unit weight, γ (kN/m ³)	Undrained shear strength at top of layer, S_u (kPa)	Young's Modulus at top of layer, E_0 (MPa)	Poisson's Ratio, ν (-)	Coefficient of lateral earth pressure at rest, K_0 (-)
Unbound pavement	21	-	50	0.263	0.36
Cohesive subgrade, Case 1	17	$=0.001m_1.p'$	$=m_1.p'$	0.495	0.45
Cohesive subgrade, Case 2	17	5.91	5.91	0.495	0.45
Cohesive subgrade, Case 3	17	15.26	15.26	0.495	0.45

Table 2: Model parameters assumed for various pavement designs and corresponding depths to normal consolidation required to satisfy Equation 1. A suffix of '*' in results presented herein indicates groundwater at the base of the pavement.

Pavement thickness (m)	Depth to water table (m)	z_{NC} for Case 2 (m)	z_{NC} for Case 3 (m)
0.25	0	2.700	7.192
0.50	0	2.561	7.053
1.00	0	2.283	6.775
1.50	0	2.005	6.497
0.25*	0.25	2.359	6.851
1.50*	1.50	1.500	4.450

Table 3: Ultimate wheel pressures from finite element modelling for single and multi-wheeled models. N.b. the bearing capacity of the pavement-only single-wheel model is 111kPa.

Pavement depth (m)		Ultimate wheel pressure, ω_{ult} (kPa)		Ratio of ultimate pressures, Case 3:Case 2
		Subgrade Case 2	Subgrade Case 3	
0.00	single wheel	32	80	2.54
0.25		54	105	1.94
0.50		98	111	1.13
1.00		111	111	1.00
1.50		111	111	1.00
1.50: multi-wheel		197	320	1.62

Table 4: Ultimate wheel pressures for single and multi-wheel models with varying pavement strength parameters. N.b. SC = Subgrade Case, * = groundwater level at base of pavement.

Subgrade model and pavement depth	ω_{ult} with pavement $\phi' = 40^\circ$ (kPa)	ω_{ult} with pavement $\phi' = 32^\circ$ (kPa)
Single wheel SC1, 0.25m	18	13
Single wheel SC2, 0.25m	54	29
Single wheel SC2*, 0.25m	63	48
Multi-wheel SC2, 1.5m	197	169
Multi-wheel SC3, 1.5m	320	290
Multi-wheel SC3*, 1.5m	340	320