University of Southern Queensland
Faculty of Engineering and Surveying

Investigates the effect of surface shear on pavements near road intersection

A dissertation submitted by

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In fulfilment of the requirements of

Courses ENG4111 and ENG4112 Research projects

Towards the degree of

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Abstract

The objective of the project was to study the effect of surface shear on the performance of pavements. This was achieved through the collection and analysis of three data sets. Firstly, background information was reviewed providing information on tyre-pavement interaction, pavements design and failure mechanisms. Secondly, field data was collected from four roads and comprised photos of vertical deformation taken regular intervals, assessment of ride quality and movies of vehicles braking and accelerating on the roads. Lastly, a parametric study was carried out using the finite element analysis program SIGMA/W.

The findings confirm that vehicles accelerating and decelerating impose additional stresses within the pavement and consequently result in additional vertical deformation subject on the level of horizontal stress. However, at the investigated intersection the total vertical deformation was within limits specified in Austroads demonstrating that, current design methods are suitable for the design of pavement in areas that experience high surface shear.

To validate these findings further, additional research in this area would be required.

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ENG4111 Research Project Part 1 &
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Date

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Chapter 1

Introduction

This project studies the effects that surface shears, caused by the acceleration and deceleration of vehicles, have on the performance on flexible pavements. The development of these forces are well recognised in current design and pavement documentation however, little information on the magnitude and there consequential effect on pavement performances exists. There is also little written in current design documentation on how to calculate surface shears and specifically how to detail the entire pavement profile in areas that experience high surface shear.

Research in this area has been carried out to understand and model the complexity of the stresses developed at the tyre-pavement interface. Vertical pavement response and layer interfaces responses to surface stresses has also be studies to some depth. However, one area that requires further research is the effect that surface shears have on pavement performance. No field studies could be found identifying deformation patterns in flexible pavement within areas of high surface shears. The following section provides an overview of the project aim, objectives, methodology and implications.

1.1 Overview

The project aim is to study the effect of surface shear on the performance of flexible sealed pavements in the Brisbane area by looking at the appropriateness of current design methods through the collection of pavement data and theoretical analysis. Heavy vehicles accelerating and decelerating are recognised as generating the largest surface shears at the pavement-tyre interface and as such the magnitude of these forces are determined and investigated throughout the study. The study looks at surface shears developed at the tyre-pavement interface in an attempt to expand on the knowledge of the effect that surface shears have on pavement performance. Pavement data is collected from a number of sites by identifying crack and defect patterns within the investigated area.

Background information on pavement and pavement material will be discussed followed by a literature review of tyre-pavement interaction with a focus on vertical and horizontal stress imposed onto the pavement from heavy vehicles changing speed, pavement response to repetitive loading and likely deterioration mechanisms. Review of pavement design looks at the consideration of shear forces, caused by the acceleration and deceleration of vehicles, at intersections and the consequential detailed design procedures to accommodate for these forces ensuring satisfactory performance over the pavement design life. Vertical and horizontal stress state levels at the interface are predicted by reviewing theoretical and experimental research, and the values imputed in a developed FEA model using the program SIGMA/W. Pavement response to fatigue and the shakedown effect from long term loading is discussed as the primary deterioration mechanism.

1.2 Goals

The goal of the project is to collect field data in areas where high surface shears are generated and to investigate the effect of resulting shear stress on pavement performance. Field data include a collection of short video clips of vehicles approaching and going away from each intersection, photographic records along the horizontal alignment within each wheel track and crack and defect assessment along over the pavement surface. General site photos are also taken identifying site features. Traffic counts carried out note the type and frequency of vehicles. Pavement design information from Main Roads and Brisbane City Council (BCC) is also included.

The next goal was to undertake a parametric study using the Finite Element Analysis (FEA) program SIGMA/W analysing the linear response to a range of surface stresses identified above. By inputting different vertical, and ratios of horizontal to vertical tyre pressure into SIGMA/W the stress distribution and consequential effect that the surface shears have on the pavement was examined.

The final goal is to composed and examined both field and theoretical information to identify the effect that surface shears have on pavement performance. One of the strengths of the project is that it draws on both practical and theoretical data when analysing and concluding the findings.

The project is also aimed at establishing if sufficient consideration and design detailing is included in current design documentation and to provide recommendations on pavement design regarding shear forces and the improved detailing of pavement profiles.

The project is constrained by time, meaning that only a limited number of intersections have been examined. Considering the huge number of intersection in Australia of all different ages and designs the confidence level of the findings are debatable, however may provide some insight into the subject area.

1.3 Objectives

There is a need in current design documentation to consider the effect that high surface shears have on the entire profile, not just in the wearing surface. Current design documentation (i.e. Austroads) identifies that areas of high surface shear do exist and explicitly design the wearing surface to resist these forces. However, no design recommendations are provided for the lower pavement layers. This means that current design does not account for the effects of high surface shears on the pavements unbound layers, or the base, sub-base and subgrade materials. Project objectives include recommendations to optimise current pavement design in areas of high surface shear in order for the pavement to reach the design life with tolerable deterioration requiring only standard planned maintenance and restoration at the end of life. Recommendations will be easy to adapt to current design procedures making any implementation simple and attractive.

1.4 Methodology

In order to study the effect that surface shear have on pavements four sites were chosen in areas that experience high surface shears and exhibited some degree of pavement deterioration. Areas where a high percentage of heavy trucks travels on the road (i.e. industrial subdivisions) were selected, and design and performance information collected from a combination of local government resources and site investigations. RoadTek and Main Roads Queensland (MR) were contacted to obtain pavement construction, design and maintenance records. Design procedures were reviewed and comparisons made with current practice, identifying if designers followed current practice and if any consideration were given the high surface shears likely to be imposed onto the pavement.

Literature on pavement design, the interaction at the tyre-pavement interface and pavement performance was reviewed in order to identify stress levels and deterioration mechanisms in areas of high stress. Reviewing pavement design was needed to identify the extent that the documentation considers horizontal stresses in current design practice and also the assumptions it makes regarding the tyre-pavement contact area and stress. It was important to identify likely stress levels at the tyre-pavement interface in order to conduct relevant parametric studies. Finally the literature review identified likely deterioration mechanisms with areas of high surface stresses and helped in analysing field data.

Following the literature review a parametric study looks at different combinations of vertical and horizontal stress on a homogenous soil and the effect that increasing the horizontal component has on the soil. The analysis uses a 2D static, linear elastic model using a roller traveling in the Y direction. An idealised pavement model is also built and analysed.

Lastly, the results of the fieldwork, research and parametric study are used to assess the effects of surface shear on the selected pavements. Cumulating the practical and theoretical data, an attempt is made to identify patterns in areas of high surface shears assessing the effects of high surface shears on pavements. Based on the finding of the above mentioned, recommendations are provided on the suitability of current design practice and construction methods.

1.5 Conclusion

Material and loading simplification adopted in current pavement design provides the designer with a simple recipe to follow for specifying materials and profiles for all types of condition. It is suggested that the simple recipe may be lacking where pavements experience extreme conditions, for example in areas of high surface shear.

The project offers an insight into the extreme surface stress experienced by pavements at signalised intersection and offers solutions, if needed, to the deficiency.

Chapter 2

Background

2.1 Pavements

The following section provides a brief introduction to pavements. A typical flexible pavement structure generally comprise an asphalt surface layer underlain by unbound granular material, typically comprising of base and sub-base gravel layers, which are placed on top of a weaker subgrade strata. A typical pavement profile with stress distribution is shown in Figure 2-1 below. The main purpose of the pavement material is to provide a bridging layer (or protection) over the weaker subgrade strata and to provide a long term trafficable profile satisfying serviceability requirements, structural integrity, ride quality, skid resistance, surface noise and low maintenance.

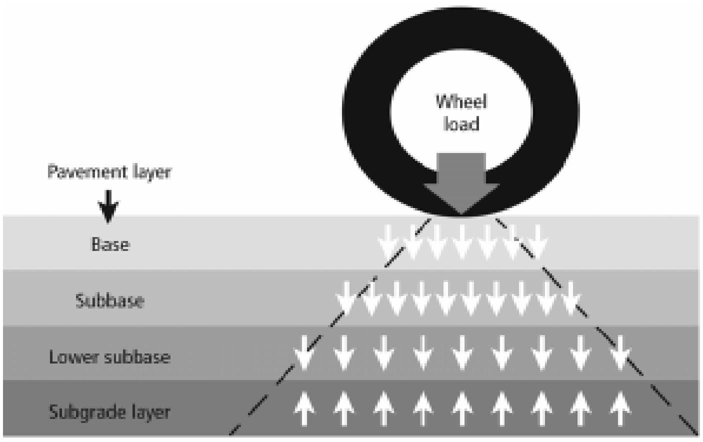

Figure 2-1: Typical Pavement Profile with Stress Distribution

Higher quality material is placed at the upper pavement layers providing protection, or bridging, to the lower weaker layers. The higher quality materials have a higher shear strength and modulus of elasticity helping to dissipate the surface loads as they travels through the pavement providing some protection to the lower weaker layers. Permanent strain can in theory be limited under repetitive traffic loads.

Asphalt is used as the pavement wearing surface with aggregates typically occupying 85% of the total volume with 10% bituminous binder, 5% filler and 5%air voids making up the mix. The mix is designed on likely traffic conditions with differing proportion and particle size distributions of aggregate, binder and filler providing different levels of structural stiffness, deformation resistance, permeability, surface texture and durability. Commonly used gravel mixes, as shown in Figure 2-2 below (including usage), include Dense Graded Asphalt (DGA), Open Graded Asphalt (OGA), Stone Masonic Asphalt (SMA) and Fine Graded Asphalt (FGGA) (*Austroads guide to pavement technology: Ashphalt* 2007, p. 10).

Table A 1: Special mix types to meet particular requirements

Mix type	Use
Open graded	Noise or spray reduction
Stone mastic asphalt	Coarse textured mix with good rutting resistance
Fine gap graded	Durable mix for low traffic situations

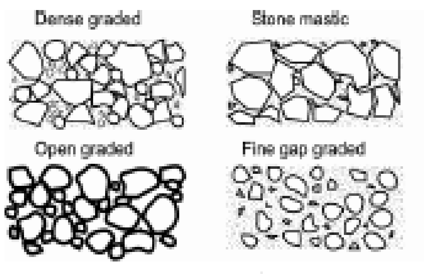

Figure A 1: Comparison of asphalt types

Figure 2-2: Gravel Mix Types and Use (Austroads Asphalt, pg. 10)

Engineering properties of the asphalt used in design are principally stiffness under moving loads (transient), resistance to fatigue and excessive deformation under repeated loading. According to Austroads, elastic response of asphalt under loading is dependent on the stiffness of the binder, inter-particle friction of the aggregate and volumetric composition of the compacted mix. However, other factors that influence performance are temperature, load cycles and frequency of loading, recovery (dependant on temperature), rate of loading and construction technique (Thom 2008, p. 136).

This means that asphalt properties can change on a daily (even hourly) basis, making it difficult if not impossible to model providing a sound reason for simplifying material properties in the design process. Typical asphalt design parameters for elastic analysis taken from Austroads: Guide to Pavement Technology is shown in Table 2-1 below:

Table 2-1: Elastic properties of thin asphalt wearing surfaces

Elastic Properties	Thin asphalt layer
Range of Poisson ratio (v)	0.25 - 0.45
Modulus of elasticity MPa (E)	2500 - 5000[1]
Typical Poisson ratio (v) values used	0.4
Typical Modulus of elasticity (E) values	3500[1]

[1] At 25°C

Gravel is used under the asphalt wearing surface providing additional protection to the subgrade. The main function of the gravel layers is to provide sufficient stiffness to reduce stresses, developed at the tyre/pavement interface, at the subgrade to a level where excessive permanent deformation is avoided. This gravel is manufactured from crushed rock and is often used for both the base and sub-base pavement strata. Higher quality material is placed closer to the surface. Specifications such as particle size distribution (or grading), durability, soundness and texture requirements ensure that the material in-service behaviour and performance can be somewhat predicted.

In the Brisbane region, the base and sub-base materials typically comprise of quarried basaltic gravel material. Sealed base course Californian Bearing Ratio (CBR) minimum values of 80% are used while typically CBR values of between 25% to 45% are used at the sub-base level (*Austroads guide to pavement technology: Structural design* 2007, pp. 57, 64). Layer thickness is dependent on subgrade strength but usually has a maximum box of around 500 mm (Austroads: Pavement Structural Design pg. 161). The engineering properties of gravel used in design are principally stiffness under moving loads (transient) and resistance to deformation under repeated loading. Similar to asphalt gravel properties are dependent on external and internal factors such as rate and magnitude of loading, material properties and moisture content. Simplified parameters make design and modelling of gravel easier. Typical base and sub-base design parameters under thin asphalts surfaces for elastic analysis taken from AusRoads: Guide to Pavement Technology (*Austroads guide to pavement technology: Structural design* 2007, p. 51) is shown in Table 2-2 below:

Table 2-2: Typical base and subbase design parameters

Elastic property	Base quality materials			Subbase quality materials
	High standard crushed rock	Normal standard crushed rock	Base quality gravel	
Range of vertical modulus (MPa)	300–700	200–500	150–400	150–400
Typical vertical modulus (MPa)	500	350	300	250[1]
Degree of anisotropy[2]	2	2	2	2
Range of Poisson's ratio (vertical, horizontal and cross)	0.25–0.4	0.25–0.4	0.25–0.4	0.25–0.4
Typical value of Poisson's ratio	0.35	0.35	0.35	0.35
f	Given by formula $f = \frac{\text{Vertical modulus}}{1+\text{Poisson's ratio}}$			

1. The values are those at typical subbase stress level in unbound granular pavements with thin bituminous surfacings.
2. Degree of anisotropy $= \frac{\text{Vertical modulus}}{\text{Horizontal modulus}}$

Underlying the gravel layers is the weaker (protected) subgrade generally comprising of natural or fill material with CBR values less than 15%. Subgrade performance is mostly a function of vertical loading factors such as frequency, rate of loading and loading magnitude. If the pavement performs as designed, the subgrade is protected from significant stresses both in the vertical and horizontal direction. Surface shears generated at the tyre/pavement interface are typically considered neglectable between 300 mm and 500 mm depth at the sub-base/subgrade interface. This is due to the stiffness of the full depth of pavement materials allowing stress to dissipate throughout the pavement. Consequently it is thought by designers that surface shears have little impact at the subgrade level.

The subgrade play a major role in pavement design and proper evaluation of the material is critical to the performance of the pavement. The subgrade strength in Australian design procedures is the determining factor for determining the thickness, composition and performance of the pavement material (*Austroads guide to pavement technology: Structural design* 2007, p. 46). In most cases this strata is complex and variable. The use of simplified subgrade design parameters, from imperial tests such as the California Bearing Ration test (CBR), allow designers to make decisions on the upper pavement layers using these results in conjunction with estimated traffic loading.

Knowledge of subgrade materials should include stiffness parameters, variations from moisture content (MC), reaction to loading and limiting values (*Austroads guide to pavement technology: Structural design* 2007, p. 85). Understanding subgrade material properties are therefore critical in the pavement design process. It is worth mentioning again that in pavement design high surface shears are not considered to significantly increase the stress levels at this layer and consequently do not contribute to additional surface deformation. As discussed subgrade material properties vary however, some typical pavement design parameters taken from the AusRoads Guide to Pavement Technology are shown in Table 2-3 below:

Table 2-3: Elastic properties of typical subgrade materials

Material	Typical young's modulus values	Typical Poisson ratio (v)
Clay Soils	35 - 100	0.45
Sandy Soils	50 - 1000	0.45

In summary two fundamental areas in which soil behaviour affects pavement performance are stiffness under moving loads (transient) and resistance to deformation under repeated loading. Bitumen-bound materials have the added property required for design of fatigue characteristics (Thom 2008, p. 5). Stresses are dissipated through the stronger surface material (asphalt and base courses) protecting the subgrade from stresses exceeding that which could cause excessive deformation. Flexible pavements are therefore generally constructed over a weaker subgrade material using selected granular material sealed by a high quality wearing asphalt cover.

2.2 Overview of shear force on pavement design

Once subgrade properties have been identified pavement cross sections are designed based on the anticipated cumulative traffic loading over the design life of the pavement. Design is simplified by using a single design load and the number of passes of an equivalent standard axil. The design process considers all types of vehicles and speeds when determining the standard axil and number of load repetitions.

Pavements must confirm to strict geometry and serviceability requirements. Pavements must be strong enough to cater for the heaviest vehicles and the cumulative effect of all vehicles over the design life. Austroads *'guide to pavement technology part 2: pavement structural design'* details clearly how to design moderate to heavy trafficked pavements considering standard axils, heavy vehicles and failure mechanisms (including fatigue). While the development of shear force between the tyre and pavement interface is recognised, design procedures provides no explicit details and it is consequently left up to the designer to design for. The effects of shear forces are again mentioned in section 8 however, states that these forces are only concentrated in the upper layers and should only be considered when designing the wearing course.

This assertion that shear forces developed at the tyre-pavement interface is only concentrated in the upper layers and has no significant effect at lower levels is highlighted in all Austroads publications and most other design (book standards) publications. Mention of the development of shear forces can be found through the Austroads publications and comments that they should be considered in the design process but gives no specific guidance on how to estimate these forces or provides specific design guidance. However, *'Ausroads: Guide to Pavement Technology Part B: Asphalt'* document provides a general guide for the selection of the type of asphalt mix, nominal size and type of aggregate mix, layer thickness and type of binder based on traffic loading and conditions. Traffic factors include estimating commercial vehicle volume, loading, speed and amount of acceleration and braking.

In areas where higher loading and adverse conditions are likely to be encountered, such as intersection, stiffer mixes with higher air voids are recommended to resist traffic stress. Stiffer binders, such as types of polymer-modified binders, are recommended to improved deformation and fatigue resistance (*Austroads guide to pavement technology: Ashphalt* 2007, p. 53). The designer of the mix may also specify low binder contents and higher quality aggregates to resist the surface shears.

In summary, design of the wearing surface to resist surface shears is covered in most design procedures. On the other hand design, the effect of high surface shears is neglected when designing the full depth of gravel and subgrade evaluation.

2.3 Identifying Areas of High surface Shear

In order to identify where the maximum damage is likely to occur as a result of high surface shears it was important to know where the point of maximum acceleration and deceleration of the vehicle occurs. One of the difficulties is differentiating between deterioration caused by slow rates of loading (vehicles traveling at slow speeds or stopped) and that caused by the acceleration and deceleration of the vehicle. While the cause of deterioration is different, it is unknown if the type damage would appear similar. This is why identification of areas of maximum changes of speed is important when analysing the effects of surface shears on pavements.

Akcelik and Besley (2001) have look at different methods of predicting acceleration and deceleration patterns including sinusoidal, polynomial and straight-line methods and conclude that the real life pattern follows an S-shape and best predicted by the polynomial model. Akçelik and Besley use a polynomial model calibrated to real life driving to describe the acceleration and deceleration patterns (Akcelik & Besley 2001) over a 100 m length with beginning and end speeds of 0 m/s, 30 m/s and 60 m/s. Their research identifies that deceleration rates are generally greater than acceleration and this gap is even more amplified for heavier vehicles. Observation made, particularly at the Darra site, confirmed the findings that deceleration of heavy vehicles is larger (significant larger at the Darra site) than vehicles accelerating.

Hammoum et al (2010) on the other hand argues that the maximum horizontal force is attained during the first 10 m of acceleration with the rest of the stop start patterns appearing to fit well to the polynomial model. It was observed that vehicles acceleration from a stop occasional jolting forward before acceleration smooths out, confirming this finding. With the exception of the first 10 m both authors point out that there can be large variations in real life driving scenarios and that further research in this area is needed to more accurately predict real life acceleration and deceleration patterns. However, both authors indicate that the largest change of speed is considered to be in the approach lane (left), decelerating to a stop and the first 10 m when a vehicle accelerates from a stop.

It was observations that the distance it takes vehicles to accelerate to the design speeds or decelerates to a stop increase with increasing vehicle size. Lighter vehicles ,cars and light trucks (type 1 to 4), generally achieved the design speed between within 100 m of accelerating while class 9 and 10 vehicles may take up to 150 m or more to reach the design speed. Similarly, light vehicles may take only 60 m to come to a stop whereas larger vehicles may take a distance of 150 m or more to come to a complete stop. If we calculate a heavy vehicle accelerating (or decelerate) linearly at 1.5 m/s^2 then it would take 167 m to reach the design speed.

It seems reasonable to conclude the greatest amount of damage resulting from the shear forces developed from accelerating, decelerating would occur within areas of the great shear. Comparing pavement damage at different distances in the braking, and accelerating lane should provide some insight into the likely damage caused by high surface shears. Using the polynomial model it is expected that the area of highest shear should be between (say) 30 m and 70 m from the intersection.

2.4 Vehicle Stresses Imposed on the Pavement

Design in Australia (and overseas) use simplified circular tyre contact shape and uniform vertical pressure for pavement structural analysis. The main reasons for the simplification of the standard axil, uniform contact shape and pressure is design and modelling is made significantly easier and allows designers to design a pavement considering the loading of a single vehicle type. Accepted confidence levels for pavement design are generally between 95% and 98%. Meaning that the estimated performance criteria will be met 95% to 98% of the time.

Austroads generally uses a simplified uniformly loaded circular tyre-pavement contact area of 750 kPa to design pavements, with the option to vary pressure in extreme conditions (*Austroads guide to pavement technology: Structural design* 2007, p. 99). For the purposes of general design, this method has proved to be a reasonable assumption with roads in Australia generally performing well. However, while most roads perform satisfactorily high rates of damage has been observed at intersection within areas of high surface shear. Indicating that the current simplified assumption used for most roads may not be appropriate for areas of high surface shear and design inputs may require revising when designing pavements within these areas of high surface shear.

From field observation Type 10 vehicles generally have the highest inflation pressure of around 850 kPa with tyre pressures generally decreasing with decreasing vehicles size. An exception is in the case of a super single tyre which may have inflation pressures of up to 1350 kPa. This project will study the effect of conventional sized vehicles (maximum type 10 vehicles) on pavements.

To propose revisions on current design (if required) understanding the actual stresses at the tyre-pavement interface is required. Data supports that the contact shape, stress levels and pressure distribution for heavy vehicles do not match closely with the simplified assumption used for the standard axil. In fact the variability of stresses developed for heavy vehicles at the tyre-pavement interface is significant and the addition of breaking and accelerating only adds to the dynamics. Heavy vehicles in fact have a contact shape closer to rectangular and an uneven and dynamic pressure distribution. The following sections discuss the dynamic stresses developed by heavy vehicles at the tyre pavement interface.

2.5 Vertical Tyre Contact Stress

Actual tyre loading is typically transmitted to the pavement through the tyre ribs and has a maximum value at the centre of the contact area (Yoo et al. 2006, p. 80). Research suggests that the maximum stress value for heavy vehicles is at the centre of the contact area and can be between 20% and 60% higher that the tyre inflation pressure (Kim 2008, p. 850). If a truck tyre with an inflation pressure of 850kPa is loaded normally then local stress at the centre of the tyre can theoretically be as high as 1360kPa. However, there are some exceptions like in the case of an overloaded/underinflated tyre that can have a peak value at the tyre edge of 2 to 3 times the inflation pressure (M de beer et al. 1999, p. 4). Stress levels are also dependant on tyre type, age, tread pattern and tyre pressure. Figure 3 and 4 below shows two examples of measured tyre contact pressure distribution from de Beer et al (1996). Figure 2-3 is an example of a typical vertical pressure shape. Figure 2-4 is a vertical tyre pressure with another tyre type (de Beer et al. 1996). It should be noted that peak values are evident in both tyre types.

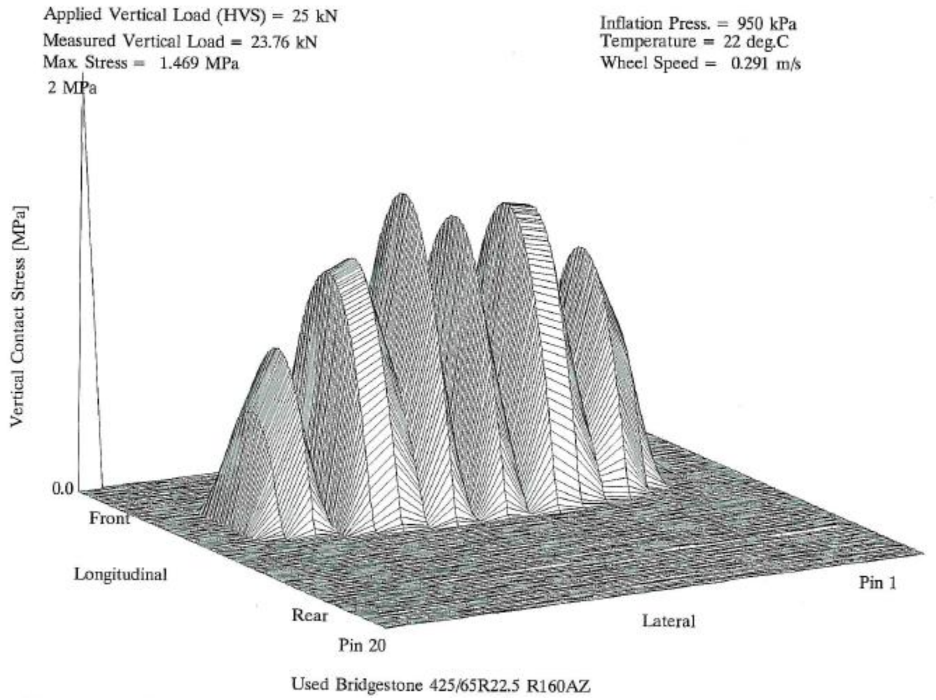

Figure 2-3: Typical Vertical Contact Stress

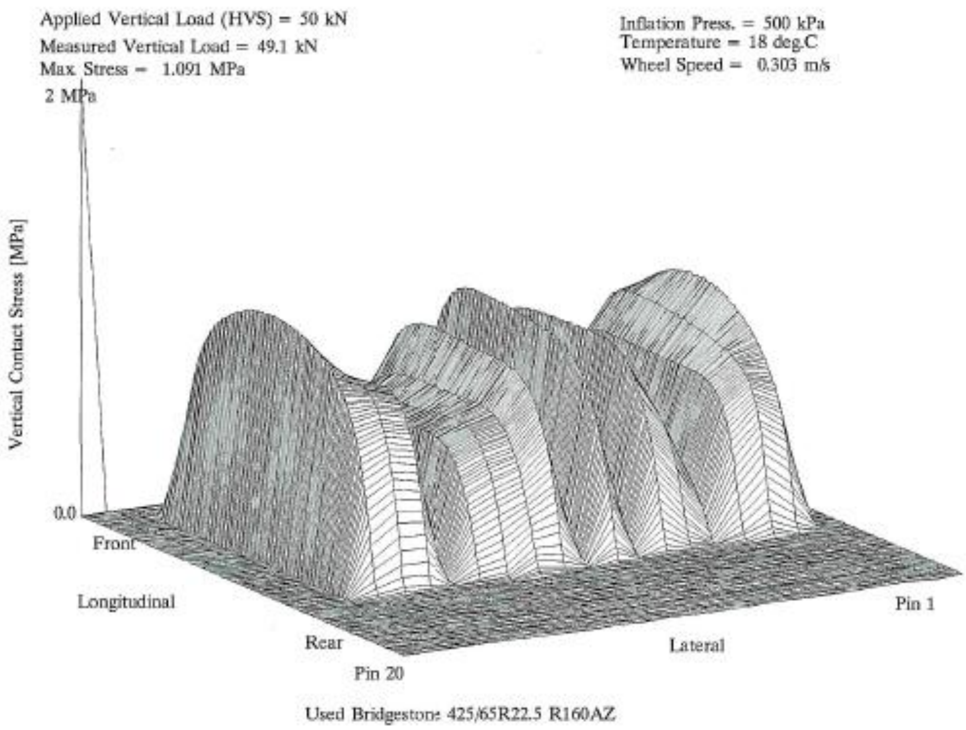

Figure 2-4: Vertical contact stress

Yue and Svec (1995) suggests that provided the surface material is sufficiently stiff peak stress values will dissipate relatively quickly through the wearing surface and the response to stress in the lower layer depends mainly on the overall load. The peak vertical stresses is considered to predominantly affect the performance of the wearing course provided it is not less than about 40 mm thick. If the wearing course is greater than about 40 mm, peak values nearly dissipated by the time they reach the asphalt-base interface and the stress distribution become similar to a normal stress distribution shape (Yue & Svec 1995, p. 858).

An interesting finding found, was that stress levels at the asphalt-base interface were in fact lower when using the actual tyre stress distribution patterns then when compared with using a uniform contact pressure (Siddharthan et al. 2002, p. 140). Indicating that the use of the conventional even stress distribution pattern is slightly conservative. Simplifying pressure distribution is therefor considered to be a suitable assumption and no coefficient increasing the model stress would be needed to compensate this simplification.

Another factor affecting vertical contact force is changing load distribution between axils as a result of changing speed. This dynamic force increase with increasing changes in velocity, and with low rates of changes in speed the peak values are more pronounced and maintain the peak value for longer (Zuo et al. 2006, p. 8). When a truck changes speed the load redistributes between the axils creating a peak value (this is actually quite dynamic) at the rear wheel in the case of accelerating and at the front wheels when decelerating. If we assume a maximum acceleration and braking of between 1.5m/s^2 and 3 m/s^2, then interpolating the graph of force vs. deceleration (Zuo et al. 2006, p. 11) and using measured changes in vertical force from Hammoum et al (Hammoum et al. 2010, p. 1261) peak vertical forces can be 5% to 10% of the vertical force compared to when the vehicle is static. For example, a static tyre contact force of 35,000 kN may peak at between 36750 kN and 38750kN when the vehicle changes speed.

It’s assumed that this increase in axil load would cause a corresponding increase in the tyre contact area resulting in the contact pressure staying relatively similar with possibly only minor increase in the peak value.

Travelling at a constant speed also imposes a degree of dynamics in the loading. Papagiannakis and Masad (2008) use a coefficient of (vertical) variation (CV) depending on suspension type to account for the dynamic variation at a constant speed. With a vehicle traveling at 60 km/hr CV is in the range of 5.7% for (air-spring suspension) and 8.7 (rubber spring suspension) while variation of tyre loads from changing speed vary between 0.5% and 6.5% (Papagiannakis & Masad 2008). Vertical pressures are dynamic and may range between 0.5% and 10% of the static load when the vehicle changes speed.

The relatively high and dynamic truck loads may in fact give reason to modify the design vehicle in areas where these vehicles are likely to traffic the pavement. Perhaps an extreme design (truck) vehicle load could be developed with consideration to increase the force by 5% to 10%.

2.6 Longitudinal tyre pressure

The longitudinal (along the vehicle travel direction) tyre contact pressure is dependent on factors similar to the vertical contact pressure (Hu & Walubita 2011, p. 253). Traditionally the longitudinal tyre pressure is considered small in comparison to vertical tyre pressure and consequently it is assumed to not have a significant influence on pavement performance (Hu & Walubita 2011, p. 251). This assumption is clearly evident in the Austroads Guide to Pavement Technology and other design documents reviewed.

A vehicle travelling approximately 60 km/h exhibits a maximum longitudinal force of about 12 % of the maximum vertical force (Siddharthan et al. 2002, p. 5). While this value may not have a significant impact on pavement performance, it has been shown that shear forces resulting from vehicles accelerating and decelerating with a tyre pressure of 850kPa can in fact reach up to 39 % of maximum vertical force at the braking wheels (Hammoum et al. 2010, p. 1261). A typically loaded truck, in theory, could therefore apply a longitudinal force of 332 kPa or more (0.39 x 850 kPa) to the pavement surface. Figure 2-5 and 2-6 below shows two examples of measured longitudinal contact pressure from vehicles travelling at a constant speed (de Beer et al 1996). Note the positive and negative values on the contact area.

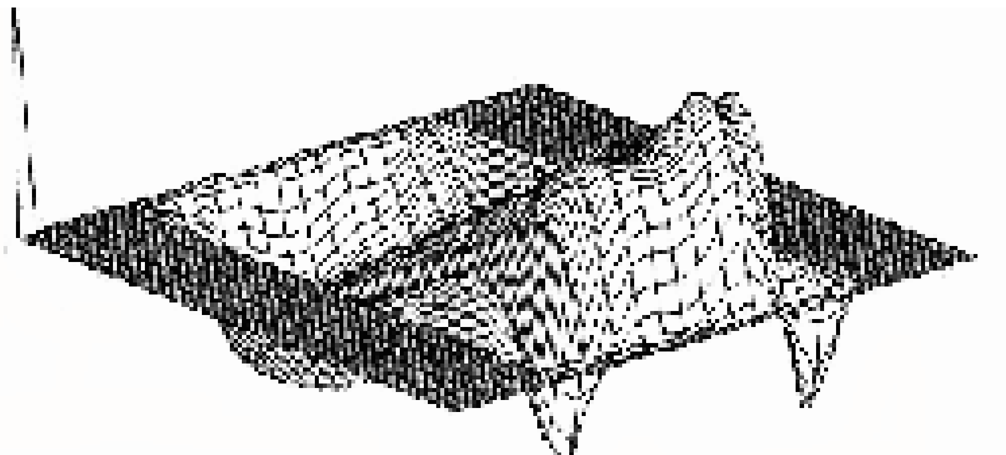

Figure 2-5: Longitudinal Stress with 650kPa pressure

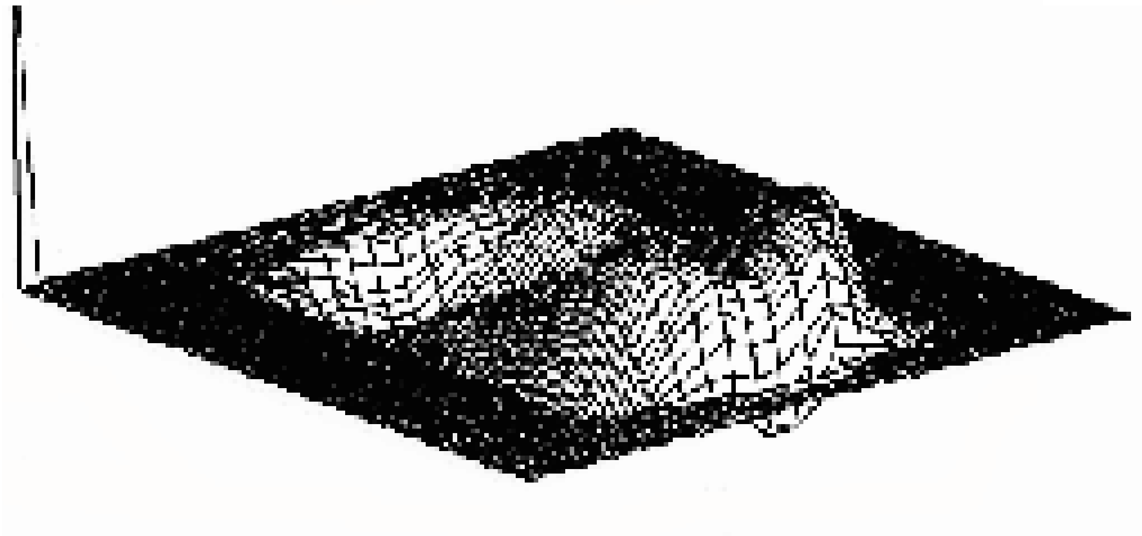

Figure 2-6: Longitudinal Stress with 650kPa pressure

Considering the relatively large vertical tyre pressure of trucks and the shear stresses of up to 39% (figures above show 10% -12%) of the vertical force it seems reasonable to assume that this increase in surface stress could affect pavement response below the asphalt. The purpose of this project is to study the effect that different rations of surface shear have on the performance of pavements.

2.7 Tyre Contact Shape

Tyre-pavement contact shape for truck tyres has been shown to be more rectangular that circular (Kim 2008). Once again, the contact area is dependent upon axil load, tyre inflation pressure, tyre type, age of tyre etc. The consequence in the structural analysis of the pavement between a rectangular and circular contact shape is minimal. Figure 2-7 and 2-8 below show two examples of the tyre contact shape measured by de Beer et al (1996):

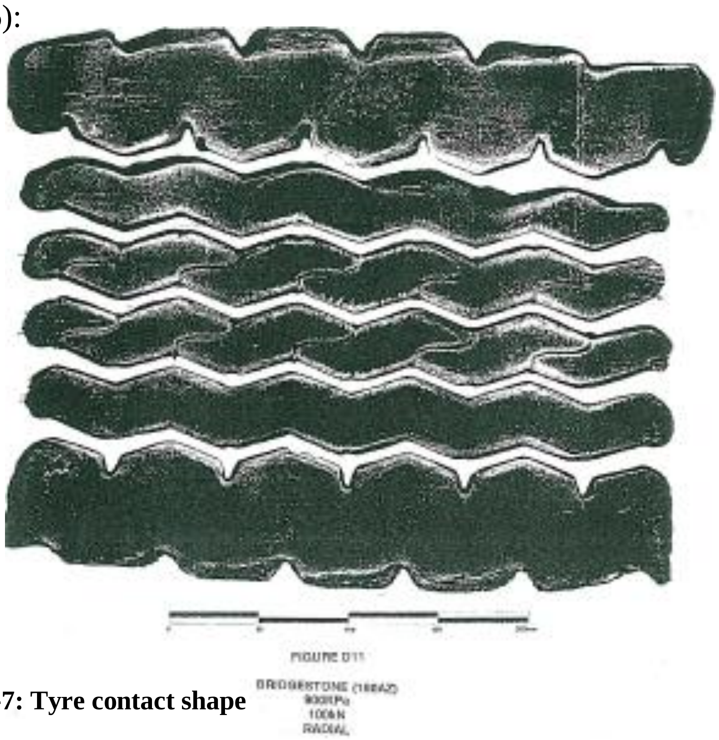

Figure 2-7: Tyre contact shape

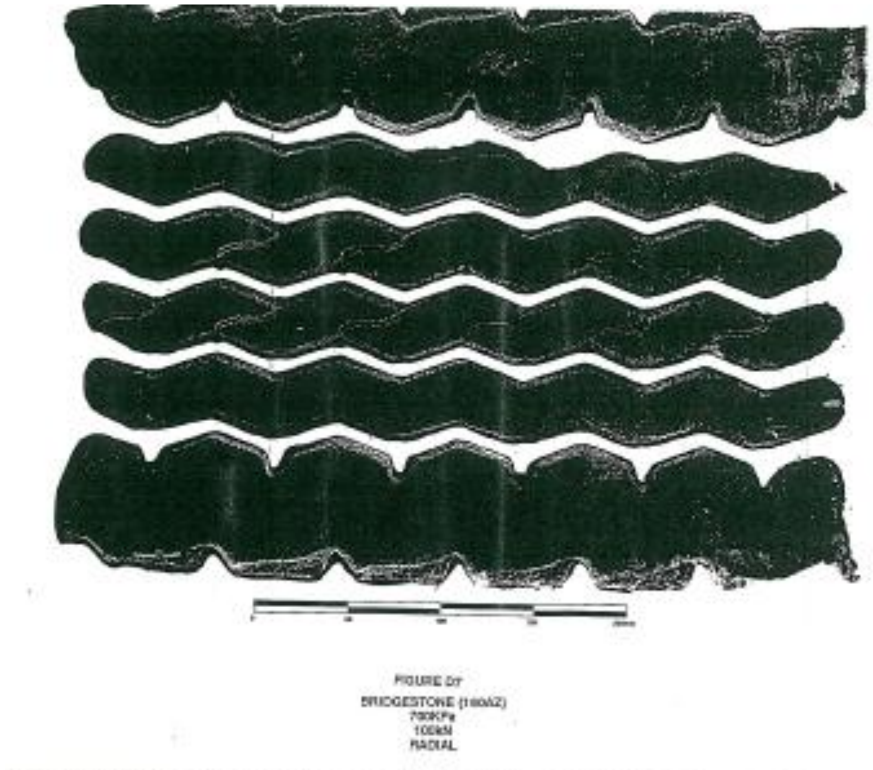

Figure 2-8: Tyre Contact Shape

2.8 Tyre Load Conclusion

The tyre stresses applied to the pavements surface are complex and vary depending on factors such as type of tyre, age of tyre, tread pattern etc. Pavement response to the stresses is also complex and dependant on many factors such as construction techniques and material properties. Theoretical tyre stresses imposed onto the pavement surface by typical heavy vehicles are summarised in Table 2-4 below:

Table 2-4: Typical heavy truck tyre contact stresses

Typical Vertical Truck Tyre Contact Pressure (kPa)	Maximum Longitudinal (shear) stress (kPa)
450 - 850	175 - 332

Super single tyre loads can be up to 1350kPa but infrequently travel on roads and consequently are not considered in this project.

2.9 Pavement Damage

Types of damage that can occur in pavement material include cracking of asphalt and vertical and horizontal displacement of the pavement layers. As discussed above the wearing surface should be designed with surface shear in mind and hence be somewhat resilient to shear damage. Any premature damage to occur in the asphalt will likely be from poor design, mixing or construction. Furthermore, any displacement in the asphalt is likely to be relative to the layer thickness. For example, a 40 mm compacted asphalt layer if put under extreme condition may settle 0.5 % to 1%, or 2 mm to 4 mm. Lower gravel materials on the other hand are designed for repetitions loading of the standard axial.

Displacement under repetitive loading is helped explained by ‘shakedown theory’. This considers the initial stress state of the material and repetitive loading. Under small repetitive loads, the pavement is likely to have an elastic response and have no permanent horizontal displacement. If loads are increased the pavement material develops small levels of permanent strain, but movement soon settles down and no further deformation occurs. This is called plactic shakedown, when particles densify by limited slipping. If loads increase further the shakedown limit may occur and usually results in continual movement followed by racketing.

Plastic shake down is predicted for most roads and is considered to be the likely failure mechanism in the investigated pavements. An example of strain from under repetitive loading can be seen in Figure 2-9 below (Austroads A Guide to Pavement Technology: Structural Design pg. 53).

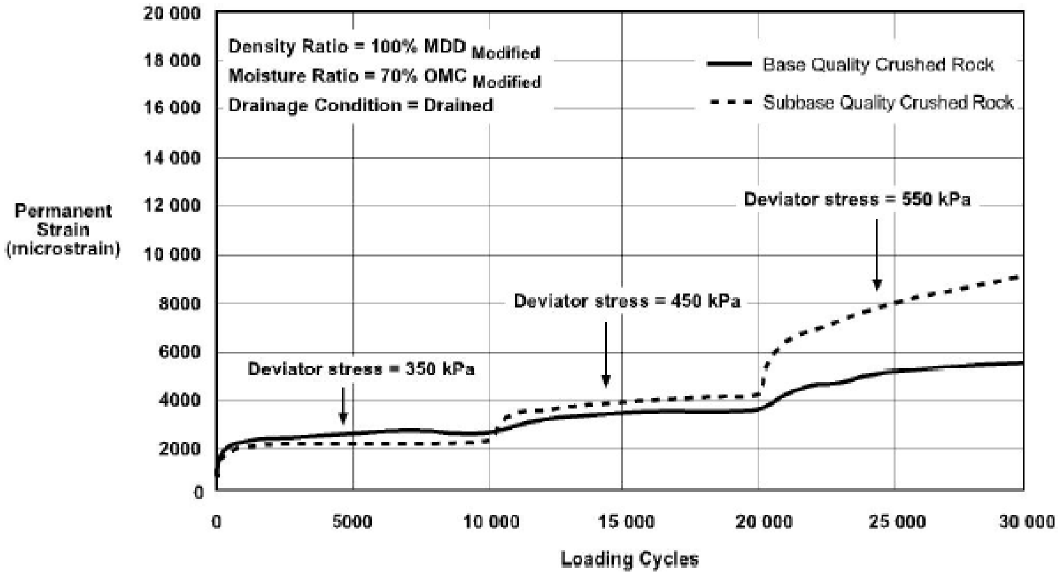

Figure 2-9: Example Relationships Between Permanent Strain and Loading Cycles

Strain in the first two stages of testing would be considered in the plastic shakedown range. In the third stage, 550kPa the shakedown limit is reached and the strain increases under the same repetitive loading conditions.

Chapter 3

Research Design and Methodology

The design and methodology component of the research project was shaped so that the information collect would come from different sources and could be compiled to provide a moderate degree of certainty in the results. Chapter three describes the three sets of data that were collected for the project. The data sets include a literature review, field measurements and observations, and a parametric study. The chapter begins by describing the rationale and limitations behind the literature review and discussing the need for the background study, the need to obtain realistic parameters subsequently used in the parametric study and likely failure mechanisms for increases in pavement traffic and loads. The next section describes the rational, limitation and methodology used for the collection of field data and observations. Locations and details of site specific features of the roads are included in this subsection. Following is a description of the parametric study using the finite element analysis program SIGMA/W. This includes verification of the model, model inputs and parameters used. Finally, the research design and methodology chapter is concluded.

Results are in the following chapter.

3.1 Literature Review

3.1.1 Rational

The purpose of the literature review was to study the complex interaction between the tyre and pavement, to identify likely pavement failure mechanisms and to obtain real life pavement parameters consequently used in the SIGMA/W model. Pavements are designed based upon certain simplified assumptions, mostly conservative, that are used to model the tyre pavement contact stress distribution and shape. Considering, that the actual stress behaviour and pavement characteristics are considerably complex and dynamic, the adoption of a simple model to base pavement design on seems reasonable and in the most part has performed well.

However, there are some exceptions where these assumptions have proven to be lacking. One example is in the approaches to bridges where roads are usually moderately sloped, have slower loading rates and trafficked by all types of vehicles (Class 1 to Class 10 typically). Under these conditions the likelihood of pavement failure has been observed to increase and consequently different design assumptions and considerations are now adopted for the design of bridge approaches.

It could be argued that pavements at signalised intersections undergo the same set of external conditions. Namely relatively large surface shear and lower rates of loading then conventional roads. Austroads do in fact recognise that relatively higher surface shears are developed when vehicles accelerate/decelerate at signalised intersection and within areas that are trafficked by heavy vehicles. However, Ausroads also assume that these forces are quickly dissipated in the surface asphalt material and consequently do not significantly increases stresses within the pavement gravel or subgrade and consequently are not taken into account in the design model.

In cases where the design engineer considers that large surface shears are likely to occur higher quality asphalt (or asphalt constitutes and thickness), with no improvement to the gravel base material or increase in box depth, is recommended. One of the focuses of the project is to investigate the assumption that surface shears are dissipated within the asphalt and high surface shears do not affect that pavement performance.

Another objective of the literature review was to collect information regarding the tyre-pavement interaction and to obtain pavement material parameters subsequently use in the parametric study. In order to develop a relevant FE model the characteristic of the tire-pavement interaction needed to be known. Actual surface stress developed between the tyre, pavements were determined from previous research carried out by DeBeer (1996), and included direct stress measurements imposed by the tyre onto the pavement and pavement reaction stresses. By identifying the pavement response to increase in both horizontal and vertical stresses it was hoped that the effect (from a theoretical perspective) of high surface shears could be revealed and the values obtained would consequently be used in the parametric study.

3.1.2 Limitation

The main objective of the literature review was to obtain information relevant to the project and in turn add confidence to the findings. Limitation of the literature review comprised time constraints and restrictions to relevant research.

Regarding time constraints, the literature review and fieldwork needed to be almost completed within one semester. Only a limited amount of research could be completed within this time frame. While the allocated 90 hours may seem like a long period of time, it is considered to be inadequate to comprehensively research the topic. As any researcher would know, one can spend countless hours carefully combing through journals and papers trying to find nuggets of information to support an idea or theory. Once one finds the answer they are looking for, this commonly leads to another question equally important and opens up ideas to other important research avenues.

To help offset the limitations imposed by the time constraints, the literature review scope was narrowed down to a few defining topics that would provide the most worthwhile information relevant to the project. By concentrating on a select number of topics directly related to the project topic it is considered that the findings in the literature review provides some level of confidence.

Another limitation was the restricted access to relevant journals. Some research pertaining to the effects of surface shear on pavements could only be accessed through purchasing the articles at full price, and as this project has no direct funding the option to not purchase the articles was made. While other research very closely related to the topic were carried out in China, and in turn were written in Chinese. While access to these articles and the ability to read Chinese may have slightly to moderately improved the quality of the literature review, access to many other peer reviewed articles equally as important were found to lessen the effect of the restrictive access to a limited number of articles.

3.1.3 Literature Review Summary

The background information collected is considered to have a moderate level of significance to the project resulting in a moderate level of confidence in the findings. While certain limitations such as time constraints and restrictions to certain articles may have reduced the value of the information, each limitation was partially offset by targeting the research and by obtaining other relevant articles.

3.2 Field Data and Observation

The object of collecting field data was to gather uncontrolled data in areas that experience high surface shear and to inturn identify patterns, if any, of pavement deformation and / or failures within these areas. Five types of field data were collected; these were 1) photographs of vertical deformation, 2) short movies that record vehicles travelling along the length of road, 3) defect assessment, 4) vehicle counts and 5) the collection of pavement design data.

3.2.1 Rational

The objective of the project is to study the effects of surface shear on pavements. To achieve a higher level of confidence in the conclusion three data sets were collected. One data set, and probably the most relevant, was the field data. Much has been written about the interaction between the tire and pavement under a moving load and equally, the amount of theoretical research papers and controlled experiments found during research was large. However, no real life evaluation of pavements could be found directly relating to the project topic. This is not to say that the studies do not exist, but rather that the number is limited.

For example De Beer et al has done extensive research on tyre pavement contact stress and has developed measuring devices within the pavement and on tyre suspension, to achieve the highest level of confidence in his results. Evidence of his research can be found in many scholarly journals. Hammoum's et al work was an excellent field study measuring vertical and horizontal stresses of accelerating and decelerating tramways (rubber tyres and asphalt roads). A final example of relevant research was that found in the Ausroads publication between permanent strain and loading cycles. While each of these practical and theoretical research topics have direct relevance to the project no real life data was found on the performance of pavements under non-controlled conditions.

Clearly one of the strengths of the research project is the comparison of non-controlled field data to both parametric and theoretical data. The project was able to compare the results of what is supposed to happen (theoretical) to what actually happens in the field (practical). The parametric study confirms that an increase in surface shear results in an increase in shear stress developed within the pavement. The literature review reveal that stress at a certain limit may cause elastic shakedown and the result of and the field study shows that areas of high surface shear have greater vertical deformation when compared to areas that experience normal loading conditions.

The below section describes the fieldwork methodology used to collect the field data.

3.2.2 Methodology

Field data comprised a collection of photographs along a length of pavement, a series of short movies of vehicles trafficking the road, pavement defect mapping and the collection of pavement design data from the relevant council. As identified in the background study, the area's most likely to be effected by high surface shears are the areas where acceleration/deceleration is highest. In the case of a vehicle traveling at a speed of 60km/hr this area is likely to occur between 30 m and 70 m from a signalised intersection. To focus on areas that experience the highest surface shear the field data was collected from approximately 0 m to 120 m from each chosen intersection.

Photographic data was collected at 4 m to 5 m intervals along this length of road with distances measured by stepping. Prior to taking the photographs the most frequent wheel paths were identified by observing vehicles driving on the length of road and also by inspecting Near Map aerial photography to identify possible wheel markings.

The photographs were taken of a 1 m long straight edge paced horizontally at each interval along the wheel tracks. Once taken the photographs were examined for deterioration patterns and the vertical deformation measured.

Vertical deformation was, to some degree, estimated from the markings on the straight edge. Different lighting conditions and angles (that the photos were taken at) made it difficult to accurately determine the vertical deformation between the straight edge and the pavement surface. However, this inadequacy was offset through consistency in the readings by using the same analyser to interpret all results. To help partially offset this limitation, the zoom function was used to measure deformation at each location. Although measurement would have been more accurate if recorded in the field, this would have meant that the time spent on the road recording data needed to be increased considerable. This in turn would have increased the risk to an unacceptable level. Figure 3-1 below shows a typical photograph taken from the field investigation. All other photos can be seen in Appendix A to R.

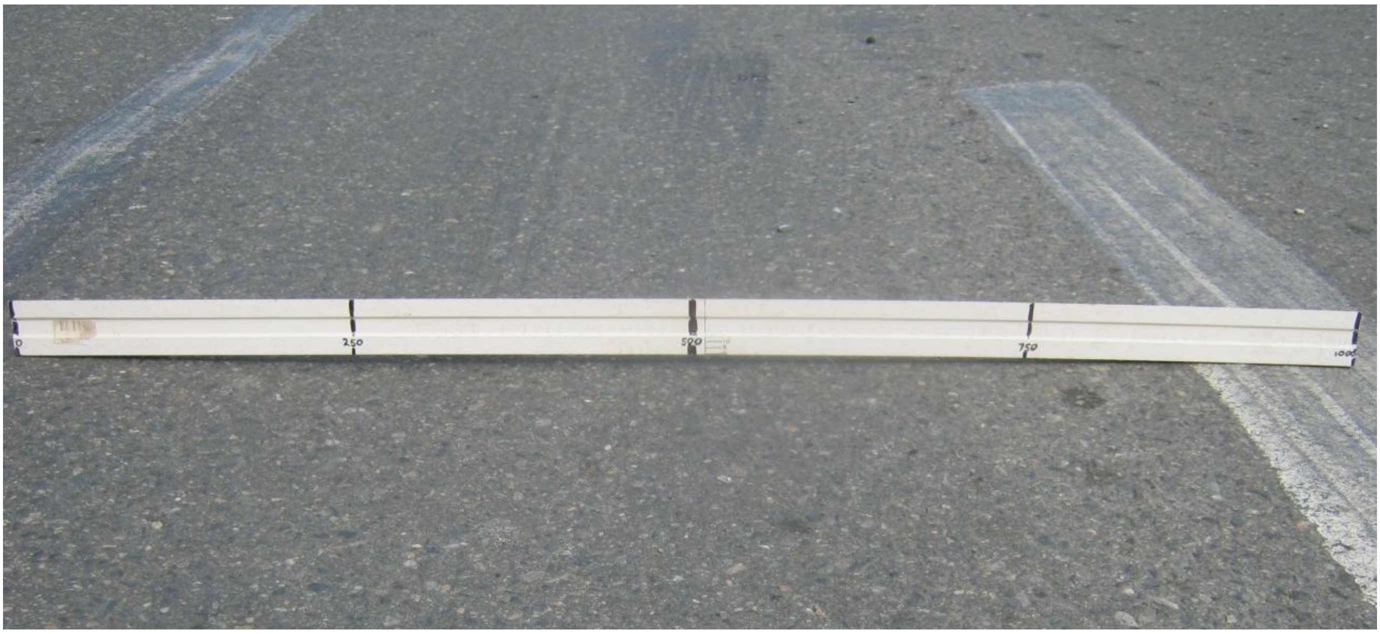

Figure 3-1: Straight edge showing vertical deformation

Next, movies of vehicles trafficking the site were recorded. In the approach direction the camera was set up near the intersection. Recordings commence prior to the vehicles initial deceleration (usually when the vehicle come in the line of sight) and continue until passing the camera or in some occasions the camera follows the vehicle around the corner. In the away direction, the camera was set up approximately 150 m from the intersection to capture the vehicle accelerating up to the design speed of 60 km/h. Most recordings are less than one minute in length while longer recording show consecutive vehicles travel the road. The movies were then examined to determine braking and acceleration patterns and in conjunction with the photographs and pavement analysis used to determine if a link exists between areas that experience relatively high surface shears and additional deformation.

One of the interesting points to note is that when changing up a gear the vehicle is in angel gear between changes, and when the next gear is engaged there is a brief point where acceleration is relatively dynamic. Also, when the vehicles first disengage gears there tends to be a brief redistribute of load to the front axil, as identified in the literature review. A good example of this pattern of acceleration can be seen in Movie M2O00461. This response to gear changes was the most dynamic action the vehicles experienced.

Observed traffic braking and accelerating patterns were used to determine if the patterns identified in the background study hold true. The level of accuracy in determining braking patterns is wholly dependent on the competency of the observer and as such would have a varying degree of confidence based on the experience of the observer. It is considered however, that while speeds may be somewhat difficult to estimate with any degree of certainty, patterns of heavy vehicles accelerating and decelerating can be easily recognised. Visual observation included the estimation of the braking including noticing when braking starts and finishes by observing the brake lights.

Determining the braking patterns of patterns of heavy vehicles was also made easier as 'engine brakes' are used during moderate to heavy braking. For example, when the engine brake is engaged, the 'roar' of the engine indicates the level to which the brake is being applied. The louder the noise the heavier the brake is being applied. This allowed not just visual assessment of braking but also hearing assessment, and in turn made the assessment of braking patterns considerably easier and more accurate.

Vehicle counts and classification were then carried out in accordance with 'AusRoads: Pavement Structural Design'. This data was then used to calculate the annual average daily traffic for each road. It should be noted that when estimating the AADT, counts were only taken over a time period of approximately 1 hour and correction for daily and seasonal variations have been estimated and consequently, should be considered as a very rough estimate only. The vehicle classification system adopted was taken from Ausroads 'Guide to Pavement Structural Design 2010' and is shown in Figure 3-2 below.

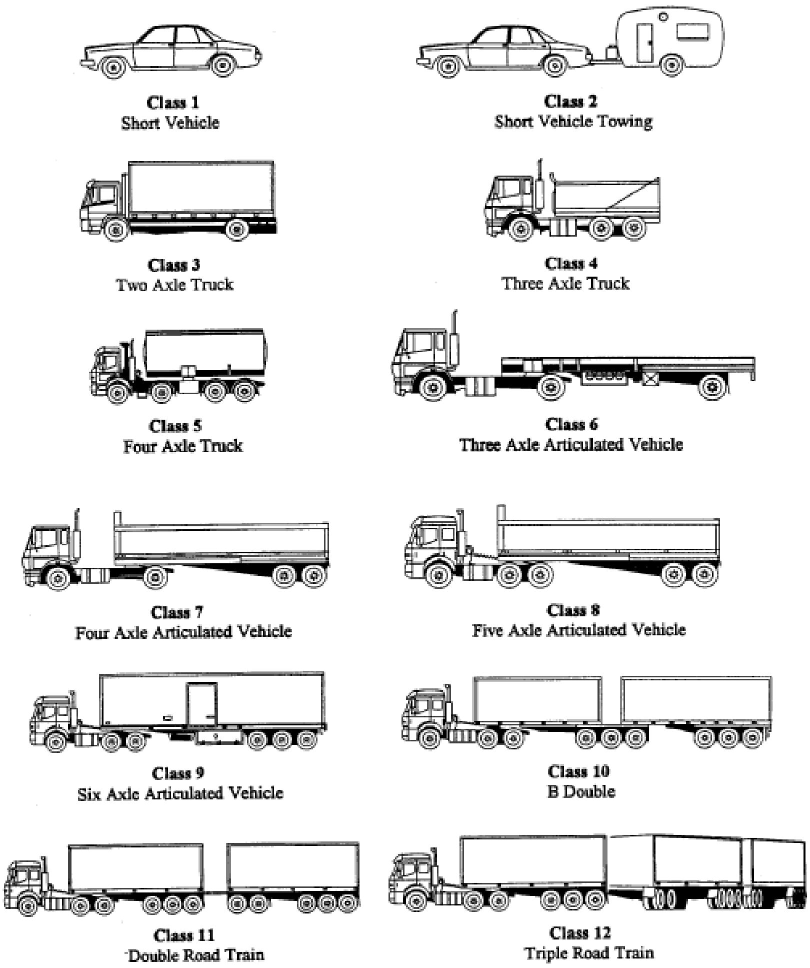

Figure 7.1: Dominant vehicles in each Austroads class

Figure 3-2 Ausroads vehicle class (pg. 85)

An additional assessment of the roads included defect assessment and a drive-through to assess drivability. Visual assessment of the pavement defects helps to explain irregularities in the field results and to provide justification for the manipulation of data. The visual assessment involved visually identifying defects, taking measurements and photographing the defect and was carried out in accordance with Ausroads *'guide to pavement technology part 5: pavement evaluation and treatment design*'. The drive-through assessment is subjective and rates the ride quality at the intersection on both incoming and outgoing lanes. A simple rating assessing the ride quality was given to each road. The rating was between 1 and 5 with 1 being the worst indicating a bumpy ride and 5 being a relatively smooth ride. It was also based on the consideration that all industrial roads should perform adequately, and so a rating of 1 does not indicate that vehicles bounce uncontrollable like when four-wheel driving. But that the ride is relatively bouncy.

3.4 Selected Roads

Four signalised intersections were chosen for the research project. Each road had a 60 km/hr speed limit. The main criteria for selecting the intersections were that they needed to have a high percentage of heavy vehicles. Consequently, all intersections were selected within industrial subdivision within and around the Brisbane area. Each site had different features (i.e. slopes, approaches and signalisation) resulting in different braking patterns and braking intensities subsequently providing different ranges of horizontal to vertical tyre pressures (Mu).

While the ranges of Mu cannot be estimated with any degree of certainty each road has been comparatively ranked (with each other) to indicate the likely level of Mu acting as an aid when analysing the results. Further, it is considered that increasing Mu values indicate increases in the peaks of stresses during dynamic loading. Thus the selection of roads provides a range of site conditions that are able to be compared to the different scenarios considered in the parametric study.

Figure 3-3 below shows where the sites are located in relation to each other and within the Brisbane area.

Figure 3-3: Road location plan

The below subsection describes the main features of each road with an aerial picture of the investigated shown.

3.4.1 Mica Street, Wacol

The corner Mica and Cobalt Streets is located within the Wacol industrial subdivision approximately 18.5 km southwest of Brisbane. The intersection is controlled by untimed traffic lights with Mica Street being relatively level in both the investigated area and the approach. As shown in Figure 3-4 below the approach to the intersection is one lane but divides into two lanes approximately 110 m back from the intersection while the outgoing lane is a single lane. Most vehicles trafficking the road are travelling to or from industrial building located within the subdivision and it appeared that most vehicles approaching were travelling at the design speed (60kn/hr).

One of the features of Mica Street was most vehicles started braking further away from the intersection than compared to the other sites and the braking patterns appeared to be smother and more controlled (i.e. less dynamic). This may be due to the fact that traffic lights controlled the flow of traffic and if timed properly vehicles could continue through the lights without stopping. Mu values for Acanthus Street could be in the mid-range (say 0.15) of likely values. Figure 3-4 below shows an aerial picture of Mica Street, Wacol.

Figure 3-4: Mica Street and Cobalt Street, Wacol

3.4.2 Acanthus Street, Darra

The corner of Acanthus Street and Boundary Road is located within the Wacol industrial subdivision approximately 13 km south west of Brisbane. The intersection is controlled by stop signs. The approach to the intersection has a moderate (say 5% to 7%) downhill slope and grades into a slight slope (say 1% to 3%) around 130 m from the intersection. Approaching the intersection at Acanthus Street it was observed that most vehicles were traveling close to the maximum speed limit (and possibly over) within 150 m of the intersection. As a result, vehicles decelerating at Acanthus Street seemed to break a lot harder at this site when compared to the other sites. Conversely, deceleration of vehicles was less in the outgoing lane. The combination of a sloping road and higher speeds suggest that this site experiences relatively larger surface shear in the deceleration lane when compared to the other investigated roads. It is possible that Mu values for Acanthus Street could be in the higher end (say 0.25) of likely values. Compared to Acanthus Street vehicles on the other assessed roads all appeared to approach the intersections at seeds lower. An aerial picture of Acanthus Street, Darra shown below:

Figure 3-5: Acanthus Street and Boundary Road, Darra

3.4.3 Beaufighter Avenue, Archerfield

The corner of Beaufighter Avenue and Boundary Road is located within the Archerfield industrial subdivision approximately 10.7 km south west of Brisbane. The intersection is controlled by a stop sign with Beaufighter Avenue being relatively level at both the investigated area and the approach. Both the approach and outgoing lanes are single. The majority of vehicles observed during the investigation comprised heavy vehicles exiting the service station approximately 160 m south of the intersection. Most vehicles were therefore traveling less that the speed limit, within the investigated area, and when compared to the other sites it was observed that deceleration was the lowest and possible the least dynamic. Indicating that Mu values for Beaufighter Street could be in the lower end (say 0.10) of likely values.

In regard to vehicles exiting the service station the deceleration (approach to the intersection) also appeared to be relatively constant. Since vehicles were traveling at a speed lower than the speed limit and were expecting to stop at the intersection, the accelerations appeared to be more controlled and consequently more constant.

An aerial picture of Beaufighter Street, Archerfield is shown below:

Figure 3-6: the corner of Beaufighter Avenue and Boundary Road, Archerfield

3.4.4 Lavarack Avenue, Eagle Farm

The corner of Lavarack Avenue and Holt Street is located within the Eagle Farm industrial subdivision approximately 8.7 km northeast of Brisbane. The intersection is controlled by a giveaway sign and roundabout. The approach to the intersection is level and has a bend approximately 65 m from the intersection. Approaching the intersection at Lavarack Avenue it was observed that most vehicles approached faster and braked hardest relatively close to the intersection. It appeared that most vehicles anticipated that stopping was not required, but accelerated straight through if clear to do so. However, when required to stop vehicles braked relatively hard. Vehicles appeared to approach the intersections at speeds close to 60km/h and braked heaviest within approximately 20 m to 50 m from the intersection. Mu values for Lavarack Street are estimated to be in the middle range (say 0.20) of likely values.

An aerial picture of Acanthus Street, Eagle Farm shown below:

Figure 3-7: the corner of Beaufighter Avenue and Boundary Road, Archerfield

3.2.3 Limitation

The method for the collection of field data was originally determined by considering time constrains, available resources and the scope of the project. This led to a number of limitations and includes the following:

- a longer length of road could not be examined;
- actual wheel tracks may be wider than 1 m. Ausroads also recommends using a 1.5 meter straight edge;
- results would be more accurate if vertical deformation measurements could be recorded on site and not from photographs;

- determining vertical deformation from photos was difficult and some judgement was needed, possible resulting in some degree of error; and
- Some measurements were particularly difficult to determine as shadows and sunlight increased the difficulty in measuring.
- Cannot estimate Mu with any degree of certainty

Other limitation (or possible errors) includes those from the interpretation of the photos. Photos were not taken from ground level and consequently some judgment was needed when determining vertical displacement. It is also considered that if another person was to interpret the results vertical displacement values presented may be different. However, values were checked for interpretive consistency and similar results/patterns should not depend on the persons determining the values.

Considering the above limitation, field results are considered fit for the project and do provide valuable information for the comparative analysis with the FEA program SIGMA/W and literature review.

3.1 Pavement Design

Another objective of the project was to determine if current design methods have been adopted at the investigated intersections. This requires determination of the AADT, design traffic and the comparison between the constructed pavement design and the recommended pavement design in accordance with the *'Ausroads guide to pavement technology'*. The method for determining AADT was developed specifically for the project and provides a rough approximation. The method for determining design traffic and recommended pavement design was adopted from *'Ausroads guide to pavement technology – Part 2 – Structural Design'*.

3.1.1 Rational

Determining whether or not current design standards have been adopted allows a review of the appropriateness of the design procedure in areas that experience relatively high surface shears. If the adoption of current design standards leads to unacceptable pavement failure then improvement recommendation could be made. This may include suggestions to further conservatize design simplification or the addition of design factors that could be used in these areas. However, if the pavement performs satisfactorily then this would confirm that the procedures and adopted simplification are sound. No improvement in pavement design will be offered if the areas perform well. In the mid stages of the project it was realised that any recommendation to change pavement design standards would require superior information on the design, construction and performance of roads and could ultimately lead to a reduction in pavement performance. Therefore, if pavement design was to be improved, further detailed study would be required and should be carried out in conjunction with the Austroads board (the experts).

3.1.1 Methodology

The methodology used to estimate the AADT was a based on the traffic count and some creative reasoning. For example, traffic counts were taken over a period of approximately 2 hours (1 hour in each direction) during a normal working day. Clearly, multiplying the number of vehicles by 24 to estimate the AADT would not account for daily, weekly or seasonal variation in traffic flow. So for the purposes of this project the following method was developed to calculate AADT:

$$AADT = ((C_{av} x\ 8) + (C_{av}\ x\ 0.05\ x\ 16))^{0.6}$$

Cav = Average hourly traffic based on the most heavily-trafficked lane

This method of calculation implies the following:

- Traffic over a normal 8 hour working period remains relatively constant
- Traffic for the remaining 16 hours of the day is 5% of the working period. This value considers night times (maybe less), weekend (would be variable) and other work periods (may be significantly more) not included in the 8 hour working day.
- The AADT traffic flow was further reduced as the results without the reduction were overly large.

The AADT value was then used to calculate the design traffic. The formula is shown below and is taken from Austroads ‘guide to pavement technology part 2: pavement structural design’:

$$N_{DT} = 365 \times AADT \times DF \times \frac{\%HV}{100} \times LDF \times GF \times N_{HVAG}$$

Where:

- AADT = Annual Average Daily Traffic in vehicles per day in the first year (as calculated above)
- DF = Direction Factor is the proportion of the two-way AADT travelling in the direction of the design lane (taken as 1)
- %HV = average percentage of heavy vehicles as determined by traffic count (class 3 to 12 as per Austroads)
- LDF = Lane Distribution taken as 1 (Section 7.4.3)
- GF = Cumulative Growth Factor (Section 7.4.5, based on a 20 year design life and annual growth rate of, say, 3%)
- NHVAG = average number of axle groups per heavy vehicle taken as 2.8 (Section 7.4.6, Table 7.5)

When the above formulas were used to determine the design traffic, the equivalent number of standard vehicles was significantly large when compared to standard design practises. As a result an important adjustment to the above calculation was made. This was the alteration of the percentage of heavy vehicles. If the percentage of heavy vehicles counted during the site investigation was used to calculate N_{dt}, then the DESA was excessively high. For example, the number of vehicles counted in Lane-2 Beaufighter Avenue was 75 with 69 % of heavy vehicles which equates to an ESA of over 1.2 x 10^7. In conventional design the percentage of heavy vehicles for lightly trafficked areas, shown in Table 3-1 below, is normally estimated to between 3% and 8%. In heavily trafficked the percentage of heavy vehicles areas may be estimated to between 10% and 25% (and possibly higher in extreme situation).

Consequently, the percentage of heavy vehicles was given the range of between 15% and 20% for the DEAS calculation. The design estimated standard axil was then calculated from the following:

DESA = ESA/HVAG · NDT

Where;

ESA/HCAG is taken from Table 12.2; Austroads guide to pavement technology-Part 2-Structural design.

Table 12.2 (Austroads), shown below, provides an estimation of the ESA for lightly trafficked roads on different classes of roads. While the investigated areas are considered to be within industrial and heavy commercial areas the table allows for a design simple check.

Table 3-1: Indicative heavy vehicle group volumes for lightly-trafficked urban streets

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Table 12.2: Indicative heavy vehicle axle group volumes for lightly-trafficked urban streets

Street type	AADT two-way	Heavy vehicles (%)	Design AADHV (single lane)	Design period (years)	Annual growth rate (%)	Cumulative growth factor (Table 7.4)	Axle groups per heavy vehicle	Cumulative HVAG over design period	ESA/HVAG	Indicative design traffic (ESA)
Minor with single lane traffic	30	3	0.9	20	0	20	2.0	13,140	0.2	3×10^3
				40	0	40	2.0	26,280	0.2	5×10^3
Minor with two lane traffic	90	3	1.35	20	0	20	2.0	19,710	0.2	4×10^3
				40	0	40	2.0	39,420	0.2	8×10^3
Local access with no buses	400	4	8	20	1	22.0	2.1	128,480	0.3	4×10^4
				40	1	48.9	2.1	285,576	0.3	9×10^4
Local access with buses	500	6	15	20	1	22.0	2.1	240,900	0.3	8×10^4
				40	1	48.9	2.1	535,455	0.3	1.5×10^5
Local access in industrial area	400	8	16	20	1	22.0	2.3	256,960	0.4	1.5×10^5
				40	1	48.9	2.3	571,152	0.4	3×10^5
Collector with no buses	1200	6	36	20	1.5	23.1	2.2	607,068	0.6	4×10^5
				40	1.5	54.3	2.2	1,427,004	0.6	10^6
Collector with buses	2000	7	70	20	1.5	23.1	2.2	1,180,410	0.6	8×10^5
				40	1.5	54.3	2.2	2,774,730	0.6	2×10^6

Note : Direction factor is 0.5, except for Minor Street with single lane traffic where DF= 1.0

3.1.1 Limitation

Limitation to the estimation of the design standard axil include recording the traffic count over a period of only 2 hours and the estimation of the AADT based on a developed formula. Ideally, determining traffic flow should be based on detailed counts or other available traffic flow data. As this type of additional data was not obtained during the project it was considered that a comparison, between lightly loaded roads and the ESA results obtained from the investigated industrial and heavily loaded areas, could be made to provide some form of reference check. As expected, when comparing the design ESA used in the pavement design (for the investigated roads) with Table 3-2, the ESA for industrial and heavily loaded would be expected to be increased by a factor of between 10 and 15 for similar road classifications.

The estimation of AADT could also have a high degree of error. A formula was developed to determine AADT and this consequently influences heavily the ESA. The method of formulation is considered to be backwards as the formula was adjusted using the design data for lightly trafficked roads and Table 3-2. This was done as traffic flows being estimated were relatively large and would have resulted in an overdesigned pavement. The large estimated traffic flow is considered to be caused by the relatively large percentage of heavy. Without an accurate approximation of both AADT and the ESA a proper detailed design and comparison cannot be accurately carried out. For the purposes of the project, particularly for determining if current design methods have been used at the intersection, this limitation has the highest degree of uncertainty and consequently the ESA calculated cannot be relied upon with any degree of certainty.

Another limitation regarding the examination of the appropriateness of current design practices was the comparison of the design data with the Austroads publication which is primarily based on the design of lightly trafficked urban streets. The study assesses the impact on heavily trafficked and loaded roads within industrial subdivision, and while some comparison is made with Austroads, widely adopted industry design practices have not been reviewed or compared with.

Findings in this area of the project have the potential to have major implications in terms of pavement design. The main limitations include inadequate traffic flow data and estimation, and that design data was used to partly calibrate the formulae and offset errors in ESA approximations. Therefore, based on the above limitations it is considered that in order to have any confidence in the findings on the appropriateness of current design practices, further research would be required.

3.3 Parametric Study

A FEA was carried out to study the change in shear stress for different ratios of horizontal to vertical tyre pressure (Mu). Two models were developed for the study. First was a homogeneous material model was developed primarily used to study the changes in shear stress with changes in Mu. Similarly an idealistic pavement was modelled was developed to study the changes in shear stress with changes in Mu. The location and maximum shear stresses (xy plain) were examined in each model for each value of Mu.

3.3.1 Rational

By studying the change of shear stress as Mu increases (for different vertical stresses) demonstrates the influence the braking force has on the pavement material. One of the underlying design assumptions (design simplifications) is that shear forces developed at the tyre-pavement interface dissipate within the wearing surface and consequently have minimal effect on the underlying pavement material. One of the main objectives of the parametric study was to test this assumption and determine if this and other design simplification are reasonable for the design of pavements by focusing on areas that experience high surface shear as result of acceleration and deceleration of vehicles.

3.3.1 Methodology

The purpose of the parametric study was to investigate the effect that changing levels of surface shear have on the shear stresses (x-y plain) within both the homogeneous and idealised model. The SIGMA/W parametric study comprised of five components namely; 1) develop an homogenous material model, 2) validation the model, 3) solving the model for different combination of Mu, 4) and develop and solve an idealist pavement for different combinations of Mu 5). The models developed were 2D elastic stress models. This means that rather than having a rectangular or circular contact shape the 2D model act like a roller traveling in the east direction. The SIGMA/W shear stress results would therefore be slightly higher when compared to a model having a 3D (rectangular of circular) contact shape.

Other simplification includes using a single tyre width, static analysis, uniform contact pressure and flat contact area. While the simplification may increase or decrease the value of Txy, the analysis should still provide valuable information on the soil response to changes in surface shear.

Firstly, the homogenous material model was developed in SIGMA/W using the below inputs. This model was then used to analyse the response of the soil when different vertical stresses and Mu values were applied to the surface.

Table 3-2: Parameters used in the homogenous material model

Input	Value
E	300 kPa
Poison ration	0.35
Analysis type	Stress-strain
Grid size	0.05
Model size	2 m x 5 m

However, before proceeding with the parametric study the model required validation. Validation of the model was carried out by comparing with the pressure bulb (shown in Figure 3-8 below) taken from the textbook, *Foundation Analysis and Design*, by Joseph E.Bowles (McGraw-Hill Book Company, 1975, p. 152). The pressure bulb on the right side shows the pressure isobars based on the Boussinesq equation for a strip footing which are comparable to the roller scenario used for the model analysis.

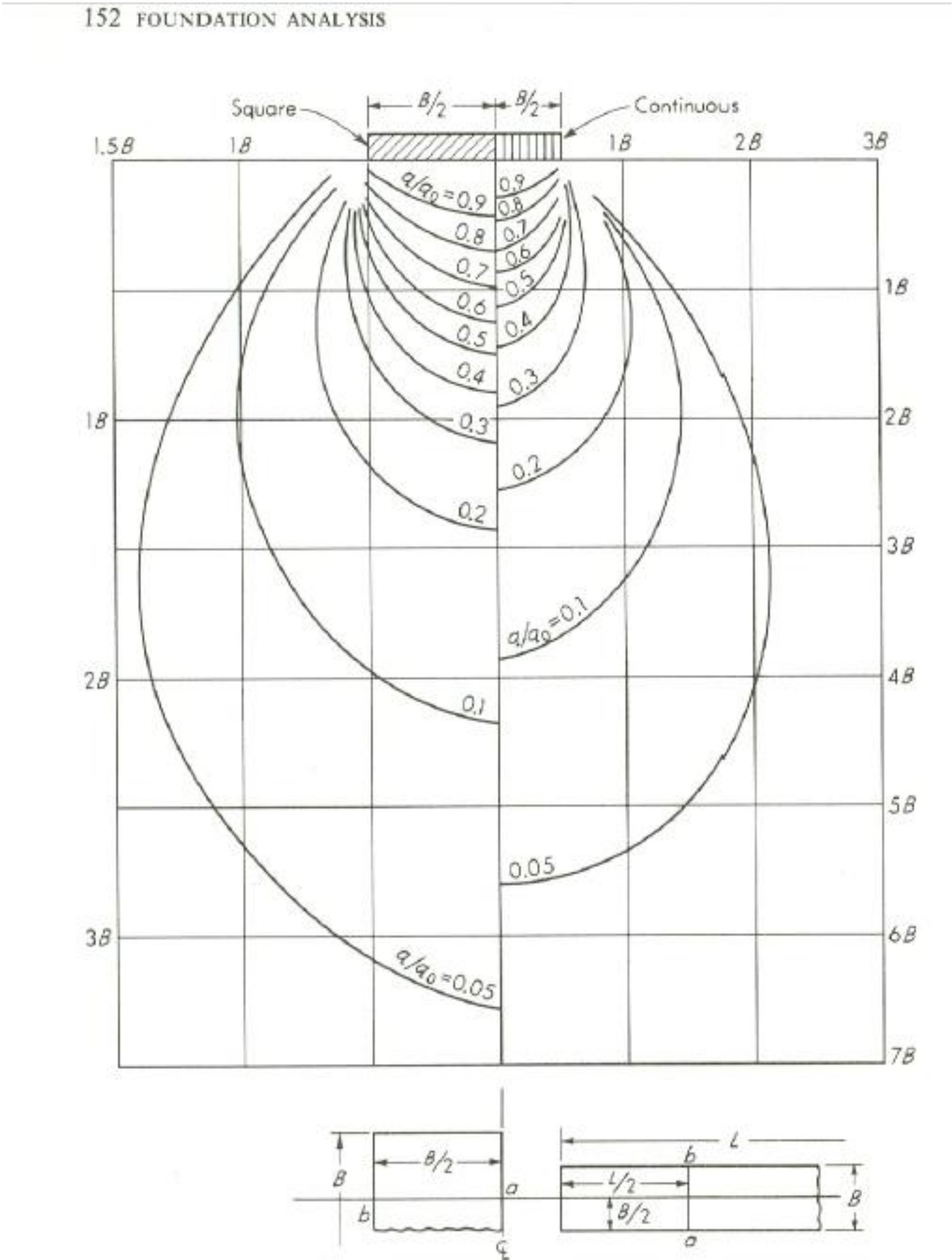

Figure 3-8: Bowlers Pressure Bulb

The model uses a 100 kPa load over a 1m wide roller for the validation. Figure 3-9 below shows the SIGMA/W computed vertical stress contours. This represents the typical bulb of vertical pressure for the roller scenario used in the pavement analysis.

The pressure (100kPa) and width of the applied surface stress are chosen so that the stress contours are comparable with Figure 3-9.

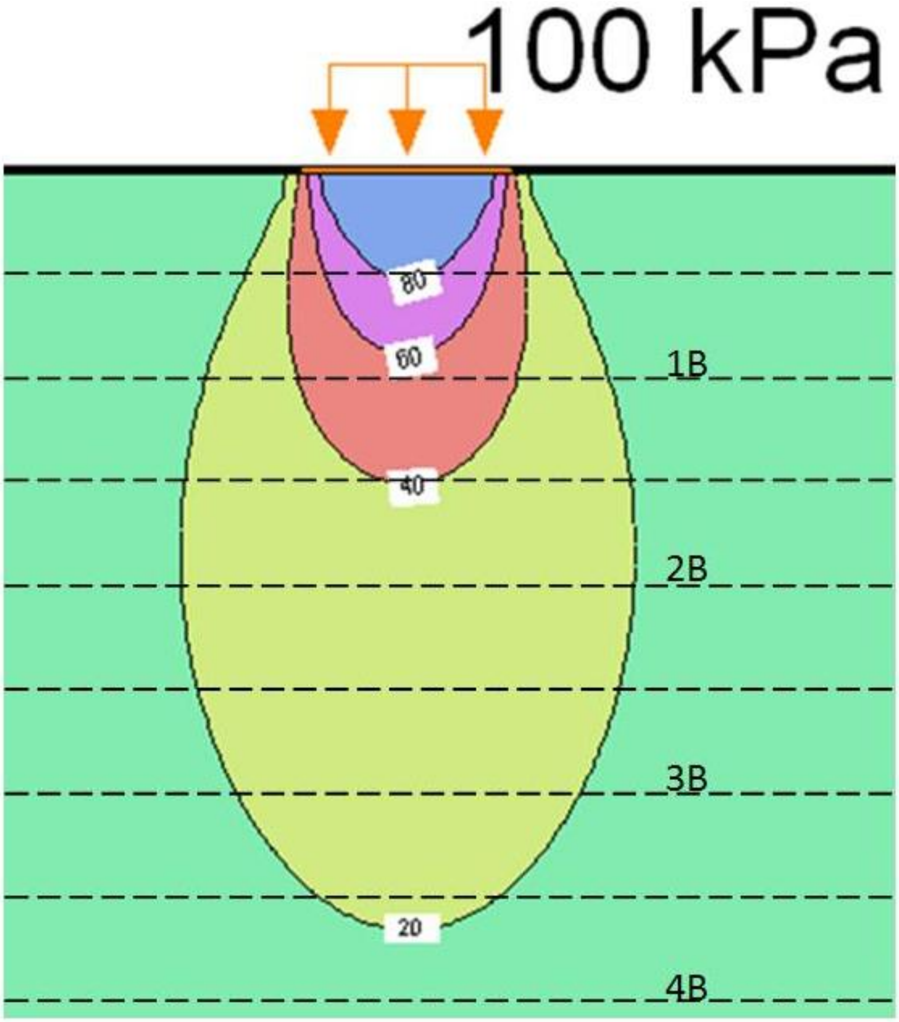

Figure 3-9: SIGMA/W pressure vertical stress contours

As can be seen the comparison between the Bowles pressure bulb and the SIGMA/W vertical pressure contours match reasonable well up to the 40% stress contour line. However, the lower-stress range (20%) is much deeper in the in the SIGMA/W model. The SIGMA/W 20% line is at approximately 3.7B while the chart value is at about

2.7B. Validation of the SIGMA/W program (in the program documentation) describes the differences between the charts (like Bowlers based on the Boussinesq equation) and the results obtained from SIGMA/W by the fact that simplifying assumption are necessary to apply the Boussinesq equation. The FEA used by SIGMA/W is a more mathematical rigorous analysis and is likely to be a better representation of the actual stress distribution.

While some differences exist between the Bowles pressure bulb and the SIGMA/W model, particularly with increasing depth, the comparison indicated that the SIGMA/W model outputs correct results. Therefore the comparison between the SIGMA/W model with Bowlers pressure bulb (1975) and DeBeers longitudinal stress (1996) indicate that the model outputs correct results.

While comparing longitudinal stresses from DeBeer (1996) was not initially part of the verification some differences were spotted between his findings and the SIGMA/W model output. When vehicles exert a surface shear (also generated when moving at a constant velocity), shear stress at the rear portion of the tire generate negative longitudinal surface stresses and positive surface shear at the front of the tire. This is shown in the literature review page 20. When horizontal stresses are applied to the surface of the model shows negative shear stresses on the right hand side.

The negative force identified by BeBeer represents a pulling (or lifting) force from the back part of the tyre applied to the pavement surface. The negative shear stress in the model outputs represents the shear stress resulting from the applied surface stresses (not a lifting force). This indicated that applying a horizontal surface stress to the model does not model correctly the interaction between the tyre and pavement.

To demonstrate the influence of the braking force, the next stage was to apply different vertical loading combination to the model while gradually increasing Mu (Mu=H/V). Vertical stresses applied to the model were typical of the loading applied from small to heavy (Class 3 to Class 10) vehicles. Horizontal (or shear) stresses applied to the model were representative of the shear stresses likely to be encountered in the field, as identified in the literature review. Table 3-3 below displays the different scenarios used to analyze the effect of surface shear in the homogenous SIGMA/W model.

Table 3-3: Vertical stress and Mu scenarios analysed in SIGMA/W

Vertical Stress		Mu (H/V)							
(Psi)	**(kPa)**	**0.0**	**0.5**	**0.1**	**0.15**	**0.2**	**0.25**	**0.3**	**0.35**
40.0	**275.8**	0.0	13.8	27.6	41.4	55.2	68.9	82.7	96.5
70.0	**482.6**	0.0	24.1	48.3	72.4	96.5	120.7	144.8	168.9
100.0	**689.5**	0.0	34.5	68.9	103.4	137.9	172.4	206.8	241.3
130.0	**896.3**	0.0	44.8	89.6	134.4	179.3	224.1	268.9	313.7

Results for each scenario were then reviewed. Of particular interest was the changing level and position of shear stress as a result of changes in Mu. Results can be seen in the following chapter.

After incrementally increasing Mu for a range of vertical stresses an idealized pavement model was developed using the parameters in Table 3-4 below. All assumption and simplification in the homogeneous material model were applied to the idealized pavement model.

Table 3-4: Parameters used in idealised pavement model

Parameters	**Material**			
	Asphalt	Base course	Sub-base	Subgrade
E (kPa)	3500	500	350	50
Poison ration	0.4	0.35	0.35	0.35
Thickness (mm)	40	150	150	1660

The idealized pavement model was developed to analyses the stress distribution in real pavements and to compare with the homogeneous model. Table 3-5 below displays the different scenarios used to analyze the effect of surface shear in the idealized SIGMA/W model.

Table 3-5: Vertical stress and Mu scenarios analyzed in idealized SIGMA/W model

Vertical Stress		**Mu (H/V)**							
(Psi)	(kPa)	**0.0**	**0.5**	**0.1**	**0.15**	**0.2**	**0.25**	**0.3**	**0.35**
40.0	275.8	116.0	136.1	140.6	145.4	150.3	155.1	159.9	164.5
130.0	896.3	425.9	441.1	457.1	472.3	487.9	503.5	519.1	534.6

Results of each model were then analyzed. Results can be seen in the following chapter.

3.2.4 Limitation

The program SIGMA/W used to model the interaction between the tyre-pavement interfaces uses several simplifications. As vehicles travel across a road the loading conditions (i.e. the stress distribution) at the wheel can change dramatically. Changes in vehicle speed, load distribution, roughness of the surface, road alignments all contribute to a dynamic tire-pavement interaction. A comparison between the SIGMA/W model simplification and actual tire-pavement features are shown in Table 3-6 below. Other simplification along the length of (the selected) road are considered to be constant i.e. temperature, weather, construction techniques, pavement design.

Table 3-6: SIGMA/W simplification

Model simplification	Tyre features	Comment
Rectangular roller	Shape ranges from round to rectangular depending on tyre type, age, make e.t.c	Model results would be higher than if round to rectangular
Width of roller is contact length of tyre	Width can be 200 mm to 400 mm	400 mm was used which is the maximum width in DeBeer (1996)
Flat contact	Tyres have ridges and patterned contact area	
Static	Dynamic	

2D traveling in east direction	3D	
Homogenous material	Not homogeneous	
Uniform pressure	Non-uniform pressure	
Single tyre with	Many different tyre widths	conservative
Horizontal stress is applied evenly	The tyre pulls the asphalt away from the road at the rear of the tyre when driving at a constant speed and this is amplified when changing speed.	

Clearly the SIGMA/W model is very simplified and provides only a guide on the changes in shear stress within the soil resulting from the application of different combinations of vertical and horizontal tyre stresses. This in turn further reinforces the point that the tyre-pavement interaction is very dynamic and difficult to model accurately.

Chapter 4

Results and Discussion

This chapter provides both the results and discussion of the fieldwork and parametric study. It begins by describing the traffic lane labelling scheme used for the identification of wheel paths followed by the rationale behind data manipulation and displays the manipulated points. Next, the results of the field work are presented. The fieldwork results begin by providing a subjective assessment of the ride quality of the roads following the design data (if available), vehicle counts, graphs of vertical deformation and discussion for each road. Finally is the SIGMA/W parametric study results from both the homogeneous material model and the idealised pavement model.

4.1 Interpretation

The method adopted for the labelling of lanes is shown in Figure 4-1 below.

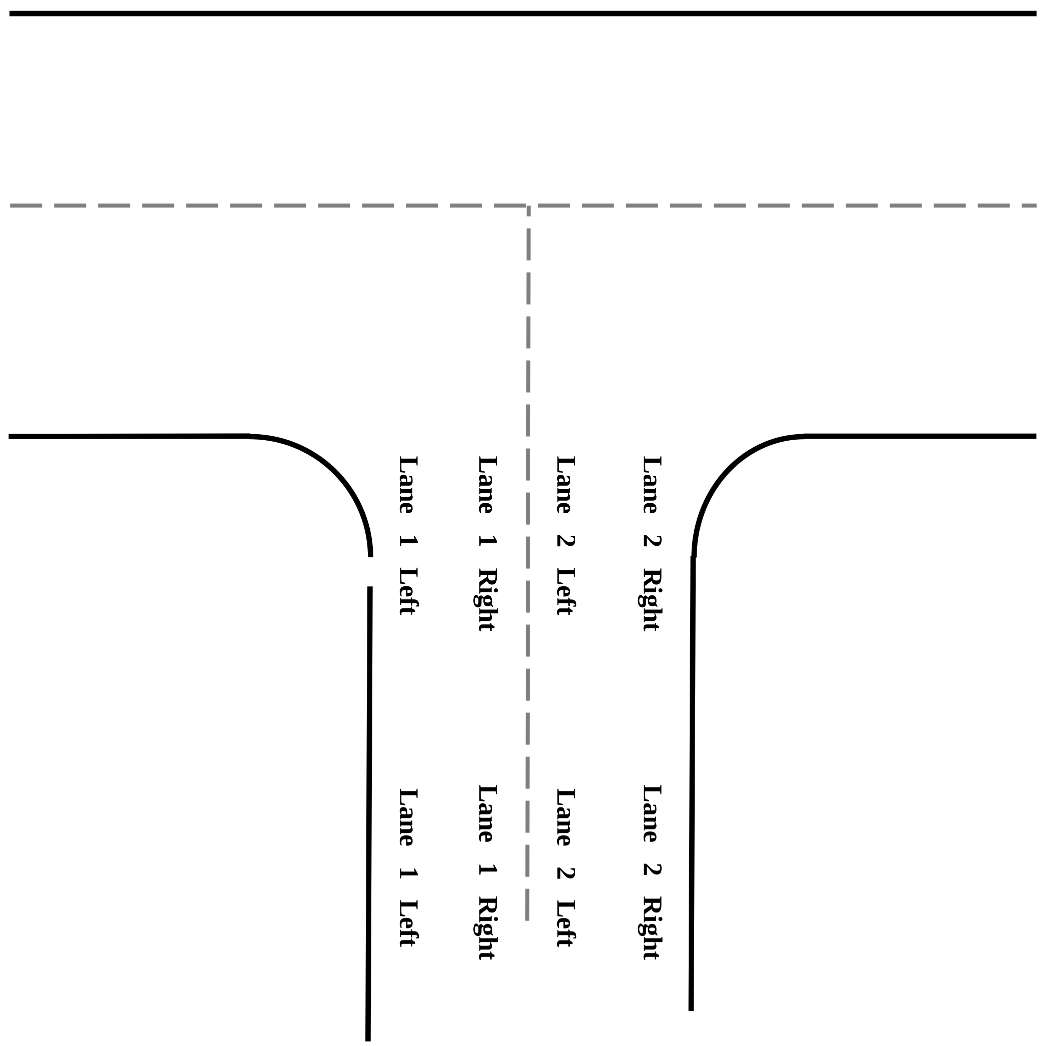

Figure 4-1: Wheel track labelling scheme

4.2 Interpretation Ride Quality Assessment

The first assessment of the investigation was assessing the riding comfort of the road. The estimated Mu and dynamics was also assessed at this stage. This aspect of the investigation is probably the most subjective part of the project, but does help to explain certain irregularities in the results. The method of assessment simply involved driving over the road and giving an average rating of the ride from a scale of 1 (very bumpy) to 5 (very smooth). These results are shown in Table 4-1 below.

Table 4-1: Ride-ability of roads

Road	Likely Mu	Dynamics	Ride Rating
Mica Street, Wacol	0.15	moderate	3
Acanthus Street, Darra	0.25	high	2
Beaufighter Avenue, Archerfield	0.10	low	2
Lavarack Avenue, Eagle Farm	0.20	high	3

The dynamics of the ride describes the level of bumpiness and in these areas. It can be seen in the following results that areas that experiences high dynamics experience increases in peak vertical deformation. Review of the movies shows vehicles bouncing in these areas.

4.2 Data Manipulation

This section describes the rational and method of data manipulation within areas of large vertical deformation without purposely influencing the results to support the hypothesis.

Several irregularities were observed in the vertical deformation data along the investigated lengths of roads. Upon inspection of the photographs and road alignment most of the irregularities could be explained. For example, the Figure 4-2 below shows a pothole at Wacol Lane 1-L at 80 m from the intersection. The vertical deformation at this location is approximately 650% to 800% greater when compared to measurements taken adjacent to the pothole. This defect was near a storm water drain and may have been caused by weakening of the pavement material from an increase in soil moisture.

Figure 4-2: Pothole at Wacol

Another example where relatively large vertical deformation can be clarified is at the Darra site Lane 2-L at 140 m from the intersection. The photo (Figure 4-4 below) shows wide longitudinal cracking extending deep within the asphalt at the location where the measurement was taken. If water ingresses into the underlying pavement and in turn weakening the pavement, then this could help explain the large deformation.

Figure 4-3: Longitudinal cracking at Darra

Results from Lane 1–R at Eagle Farm show some surprising large deformation between 55 m and 105 m. On inspection of the photos it can be seen that large longitudinal and crocodile cracking extending between these distances. While cracking at the Eagle Farm site was common, the abovementioned cracks seem wider and most appeared to extent through the whole depth of asphalt. If water was allowed to ingress into the underlying gravel then this could help to explain the excessive deformation. Lane 2-R also has significant cracking along the investigated length and could help to explain the large deformations at various distances along the road.

Where large irregular vertical deformation occurred and causes could be explained then the data point at that location was manipulated. Simply, the manipulated data points were calculated by averaging the neighbouring values. Table 4-2 below summarises each point of the field data that was manipulated.

Table 4-2: Manipulated data points

location	Original value	Manipulated Value	Comment
Darra Lane 2 L 140 m	17	5	Longitudinal cracking. Possible water ingress and ponding
Wacol Lane 1 L 4 m	50	12	Crocodile cracking and possible water ingress
Wacol Lane 1 L 12 m	50	12	Pothole
Wacol Lane 1 L 28 m	80	11	Pothole
Wacol Lane 1 R 1 m	15	2	Crocodile cracking and possible water ingress

Following are the results and discussion of the field work.

4.3 Mica Street, Wacol

Below are the vehicle counts, design and vertical deformation results for Mica Street, Wacol.

4.3.1 Vehicle count

Vehicle counts for Mica Street, Wacol is shown below for both lanes 1, 2 and 3 in Table 4-3 below.

Table 4-3: Vehicle count for Mica Street, Wacol

Site: Wacol Lane 1 and 2 Counted on 29 April 2011 at 8:40am - 9:40									
Class 1	Class 2	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10
110	3	9	12	6	2	0	2	7	4
Site: Wacol Lane 3 Taken on 29 April 2011 at 7:36am – 8:36									
Class 1	Class 2	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10
88	2	12	8	4	1	2	1	10	3

4.3.2 Design

No design information was provided for Mica Street, Wacol. However, to check what the design should comprise of the design Figure 8.4 - Design Chart for granular pavements with thin bituminous surfacing on Page 107 in Austroads Guide to Pavement technology Part 2: Structural Design for a subgrade CBR value of 5% modulus was used. The ESA values (predicted) were between 3.72x10^6 and 4.96x10^6. Austroads indicates that for an ESA of 4.6x10^6 the following design is suitable:

Less than 40mm thick asphalt;
450mm of unbound gravel;
Subgrade with a modulus of 50 MPa

4.3.3 Vertical Deformation

Figures 4-4 and 4-5 below show the vertical deformation verses distance from the intersection for lanes 1 and 2 at Mica Street, Wacol.

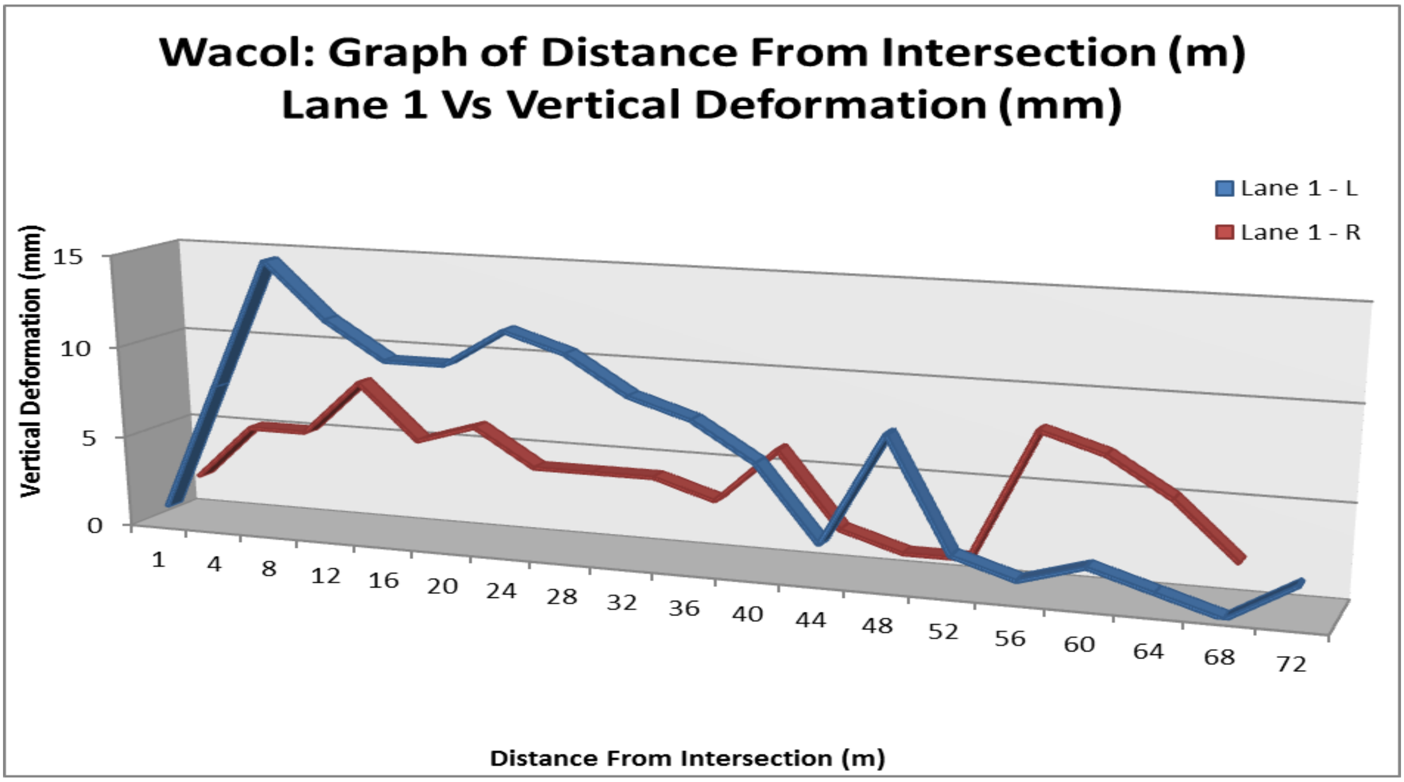

Figure 4-4: Wacol: Graph of Distance form Intersection (m) Lane 1 Vs Vertical Deformation (mm)

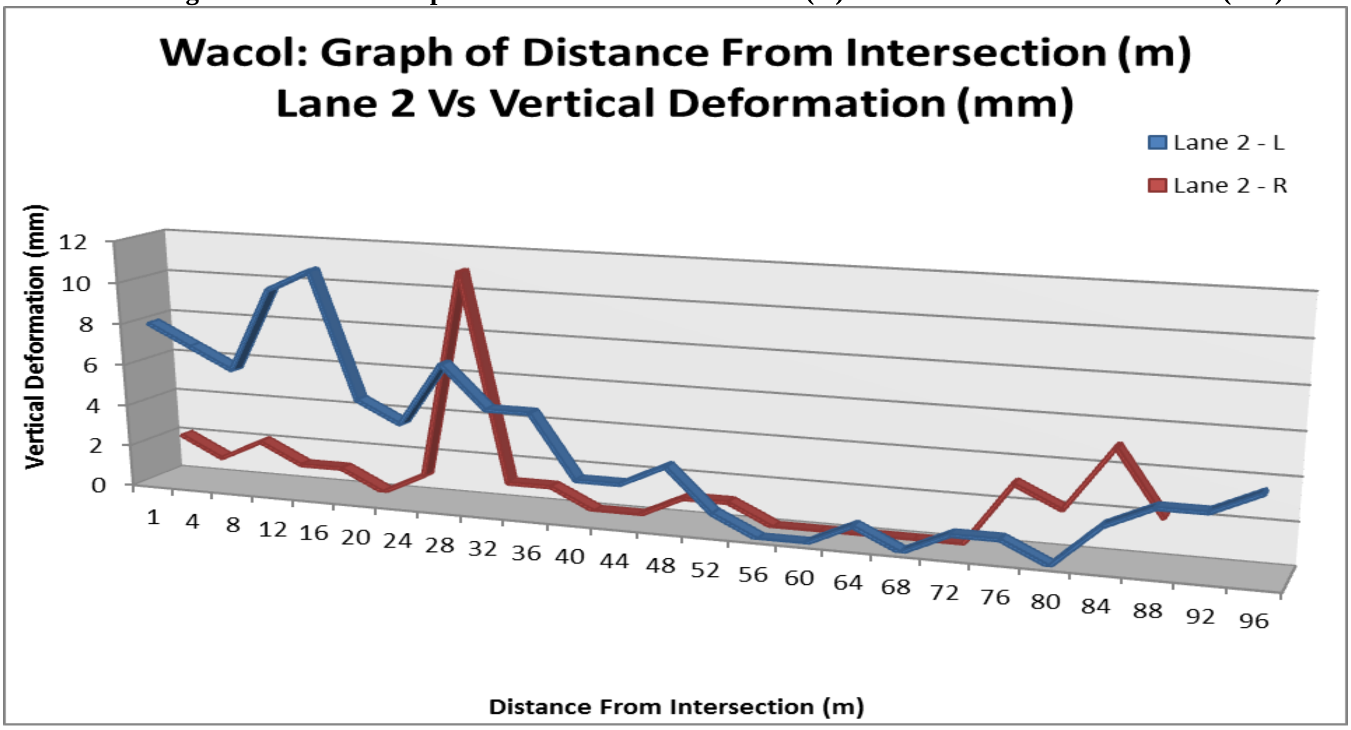

Figure 4-5 Wacol: Graph of Distance from Intersection (m) Lane 2 Vs Vertical Deformation (mm)

Lane 1-L and 2-L (outside wheel track) will be discussed together as both are deceleration lanes and display generally similar deformation patterns. Lane 2-L begins at 96 m at 3 mm vertical deformation and decreases to 0 mm at 80 m where it then increases slightly between 80 m and 72 m. Both Lane1-L and Lane 2-L vertical deformation then increase until reaching a maximum of between 10 mm and 14 mm between 4 m and 12 m before decreasing close to the intersection.

Lane 1-R and Lane 2-R (inside wheel track) display similar patterns, however values are less. The lower values may be a result of the wheel track being on the higher side of the lane resulting in a lower percentage of the load being distributed to the inside wheel track. The lateral slope of the road was not measured but can be assumed to be approximately 2 degrees to 3 degrees. While this may not seem like a large difference in loading between the inside and outside tyre, it may explain some of the reduction of vertical deformation between lanes.

It was observed on site, and can also be seen in the movies, that the deceleration patterns were slightly different then that identified in the literature review. One of the observations was that as vehicles approached the lights the velocity (and deceleration) was controlled in anticipation that the lights would change to green and a total stop would be avoided. If the lights remained red, then vehicles decelerated harder just before coming to a stop (within 25 m from the intersection). If on the other hand the light changes green, the vehicles started to accelerate before reaching the intersection. When this was the case it was observed that as the vehicles approached the intersection acceleration appeared to increase. This driving pattern may help to explain the results in both Lane 1-L and Lane 2-L

The graph of vertical deformation verses distance for the intersection for the outgoing lane (3) is shown below.

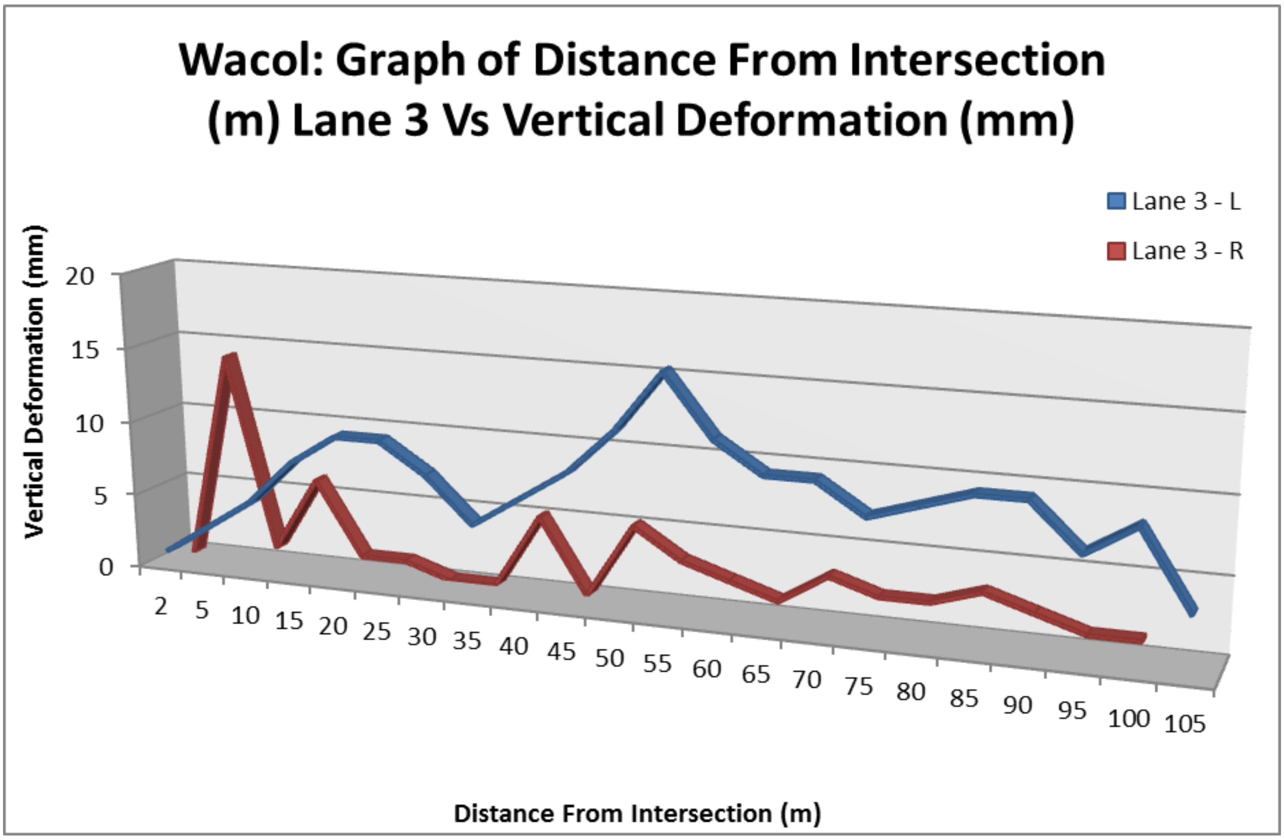

Figure 4-6: Wacol: Graph of Distance From Intersection (m) Lane 3 Vs Vertical Deformation (mm)

Both left and right wheel tracks in lane 3 display similar patterns. The results show a slight increase at 100 m in vertical deformation and remain relatively constant until around 65 m from the intersection where vertical deformation peaks between 40 m and 55 m. Deformation then begins to decrease until they start to increase and peak again between 5 m and 25 m before decreasing to zero between 0 m and 5 m.

Peak value in the acceleration lanes are again where vehicles are likely to change gear. When the vehicles entered the intersection trucks began to accelerate before changing gear at approximately 50 m, in these two areas peak values occur (within the most dynamic regions). In between the peak values acceleration was observed to be fairly constant and in these ranges deformation are also relatively consistent.

4.4 Acanthus Street, Darra

Below are the vehicle counts, design and vertical deformation results Acanthus Street, Darra.

4.4.1 Vehicle count

Vehicle counts Acanthus Street, Darra is shown below for both lanes 1 and 2 in Table 4-4 below.

Table 4-4: Vehicle count for Acanthus Street, Darra

Acanthus Street, Darra Lane 1 Counted on 5 May 2011 at 8:20am - 9:20am									
Cass 1	Class 2	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10
235	4	10	5	4	0	3	5	5	5
Acanthus Street, Darra Lane 2 Counted on 5 May 2011 at 9:25am - 10:25am									
Cass 1	Class 2	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10
56	3	5	2	5	1	3	1	2	0

4.4.2 Design

Design information for Acanthus Avenue was provided by Brisbane City Council (BCC). The pavement design comprised the following:

50mm BCC Type 3 AC (Multigrade binder) - Wearing course;
150mm BCC Type 4 AC (Multigrade binder) – Base;
195 mm sandy gravel with Silt - Sub-base;
CBR 5 precent;

Twenty year design; and

5.5 x 10^6 ESA (Equivalent Standard Axles).

In summary, the pavement comprised a 200mm asphalt layer over 150 mm of gravel then a 5% subgrade.

The Calculated ESA values for this road were between 5.0x10^6 and 5.8x10^6, and as can be seen the design ESA lies within this range.

To check if current design methods have been adopted at Acanthus Avenue, Darra the Design Chart 2 on Page 115 in Ausroads Guide to Pavement technology Part 2: Structural Design for a subgrade with a 50 MPa modulus was used. The figure indicates that for an ESA of 5.5x10^6 the following design is suitable:

200mm asphalt;

220mm of unbound gravel;

Subgrade with a modulus of 50 MPa

The design check indicates that current design practises for lightly loaded roads were adopted when designing Acanthus Avenue.

4.4.3 Vertical Deformation

Graph of distance from intersection (m) verses vertical deformation (mm) for the deceleration lane at the Darra site is shown below in Figure 4-7. As discussed previously the approach slope to the intersection was slightly down hill, and consequently vehicles were observed to have a greater deceleration at the site when compared to the other investigated sites. Also, when approaching the intersection vehicles were travelling close to or slightly higher than the speed limit. Moreover, acceleration was slower as vehicles travelled uphill.

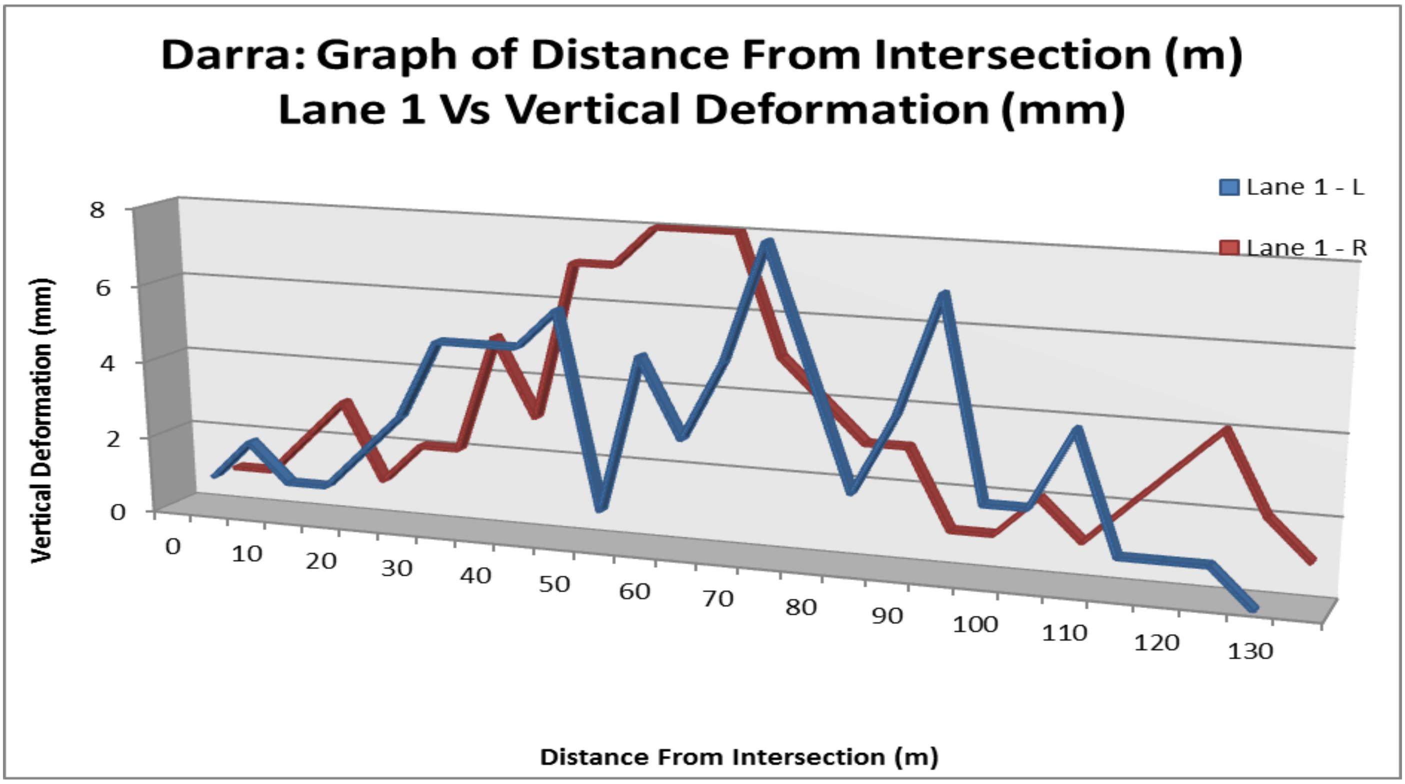

Figure 4-7: Darra Graph of Distance from Intersection (m) Lane 1 Vs Vertical Deformation (mm)

Lane 1-L (outside braking lane) results are moderately scattered however appear to display some association between areas of high surface shear and vertical deformation. Vertical deformation starts to increase between 130 m to 110 m from the intersection. Between 110 m and 55 m the results are dynamic and several peaks and troughs are apparent. This is where the lane experienced the highest surface shear and most dynamic loading conditions. However, within this range the average vertical deformation appears to have increased. Between 55 m and 0 m the vertical deformation decreases relatively constant with a decrease in deceleration.

In lane 1-R (inside deceleration lane) it can be seen that the vertical deformation generally starts to increases when the vehicles start to decelerate between 100 m to 130 m from the intersection. Vertical deformation then increases until reaching a maximum between 60 m and 80 m. This is where maximum deceleration was observed and also it is within the distances identified in the background study where maximum deceleration is likely occurs. Vertical deformation then begins to decrease as the vehicles deceleration reduces until reaching zero velocity at the intersection.

Graph of distance from intersection (m) verses vertical deformation (mm) for the acceleration lane at the Darra site is shown below in Figure 4-8;

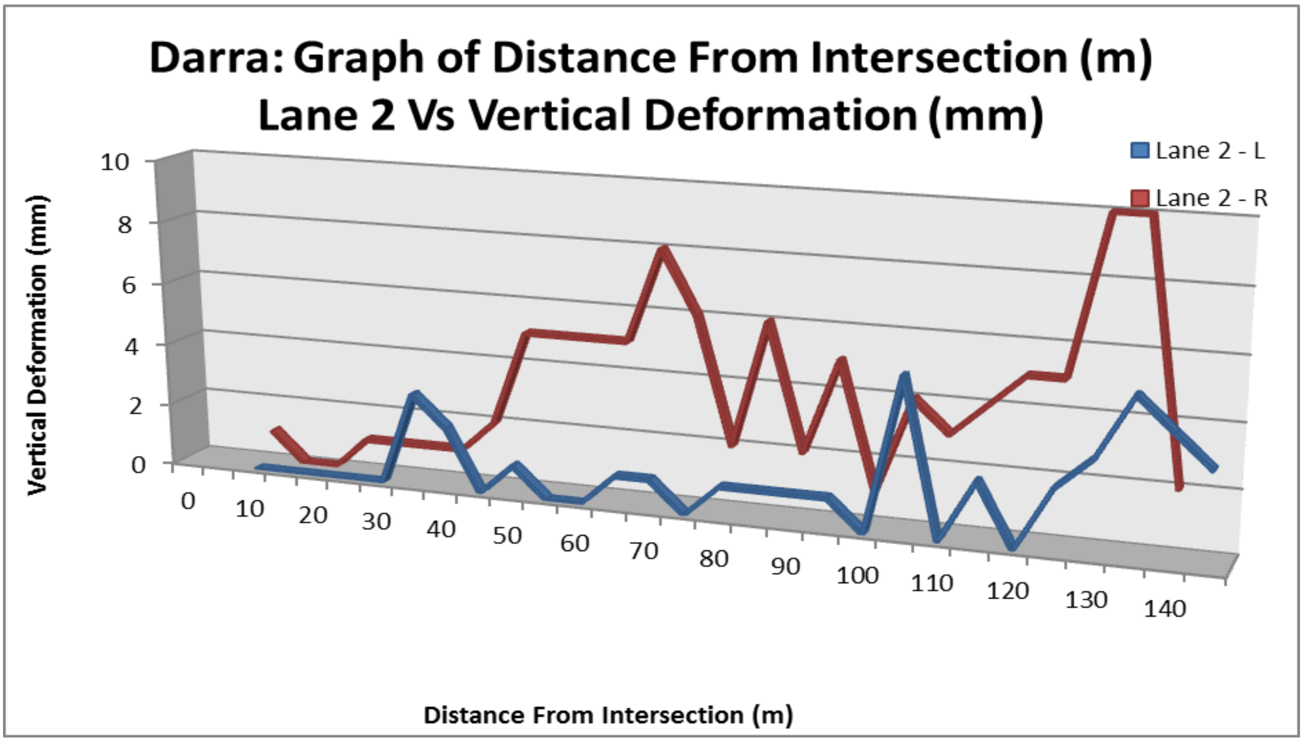

Figure 4-8: Darra: Graph of Distance from Intersection (m) Lane 2 Vs Vertical Deformation (mm)

When vehicles entered the road from Boundary Street, from the intersection, their speeds were approximated 15 km/hr and tended to be in the appropriate gear and needed to change to the next gear between 20 m and 35 m with subsequent change between 75 m and 100 m.

Lane 2-L (accelerating inside lane) results appear to be less defined. Vertical deformation starts increasing around 30 m from the intersection. At 35 m a peak in vertical deformation occurs where heavy vehicles are likely to change gears. This peak may be a result of the redistribution of load when changing gear as this was a relatively dynamic action. Between 45 m and 100 deformations are relatively constant before peaking between 100 m and 115 m, again where a change of gear was observed. Between 115 m and 140 m deformation begins increases.

Lane 2-R (accelerating outside lane) deformation increase between 10 m and 70 m. Between 70 m and 110 m the graph appears dynamic. A reason for this irregularity could be explained by a slight undulation of the road in this area. The movies show that vehicles begin to bounce between 65 m and 100 m. Between 110 m and 135 m deformation increases and then begins to drop between 135 m and 140 m.

4.5 Lavarack Avenue, Eagle Farm

Below are the vehicle counts, design and vertical deformation results for Mica Street, Wacol.

4.5.1 Vehicle count

Vehicle counts Lavarack Avenue, Eagle Farm is shown below for both lanes 1 and 2 in Table 4-5 below.

Table 4-5: Vehicle count for Lavarack Avenue, Eagle Farm

Lavarack Avenue, Eagle Farm Lane 1 Counted on 12 May 2011 at 9:35am - 10:30am									
Cass 1	Class 2	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10
77	0	25	9	11	2	4	1	1	3
Lavarack Avenue, Eagle Farm Lane 2 Counted on 12 May 11 at 10:35am - 11:30am									
Cass 1	Class 2	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10
48	0	10	3	2	1	2	1	2	0

4.5.2 Design

Design information was provided by Brisbane City Council (BCC). The pavement design at Lavarack Avenue comprised the following:

50mm AC
400-550mm Sandy Gravel - Base
CBR 5 per cent

The twenty year design traffic loading was 4.6 x 10^6 ESA (Equivalent Standard Axles).

In summary, the pavement comprised 50 mm asphalt layer over 400 mm to 500 mm gravel road base then a 5% subgrade.

The Calculated ESA values were between 3.72x10^6 and 4.96x10^6, and as can be seen the design ESA lies within this range.

To check if current design methods have been adopted at Acanthus Avenue, Darra Figure 8.4 - Design Chart for granular pavements with thin bituminous surfacing on Page 107 in Ausroads Guide to Pavement technology Part 2: Structural Design for a subgrade CBR value of 5% modulus was used. The figure indicates that for an ESA of 4.6x10^6 the following design is suitable:

Less than 40mm thick asphalt;
450mm of unbound gravel;
Subgrade with a modulus of 50 MPa

The design check indicates that current design practises for lightly loaded roads was adopted when designing Lavarack Avenue.

4.5.3 Vertical Deformation

Graph of distance from intersection (m) verses vertical deformation (mm) for the deceleration lane at the Eagle Farm site is shown below in Figure 4-9 below. As discussed previously the site was level with a slight corner between 110 m and 60 m from the intersection. Acceleration was the second largest out of all the roads and occurred close to the intersection.

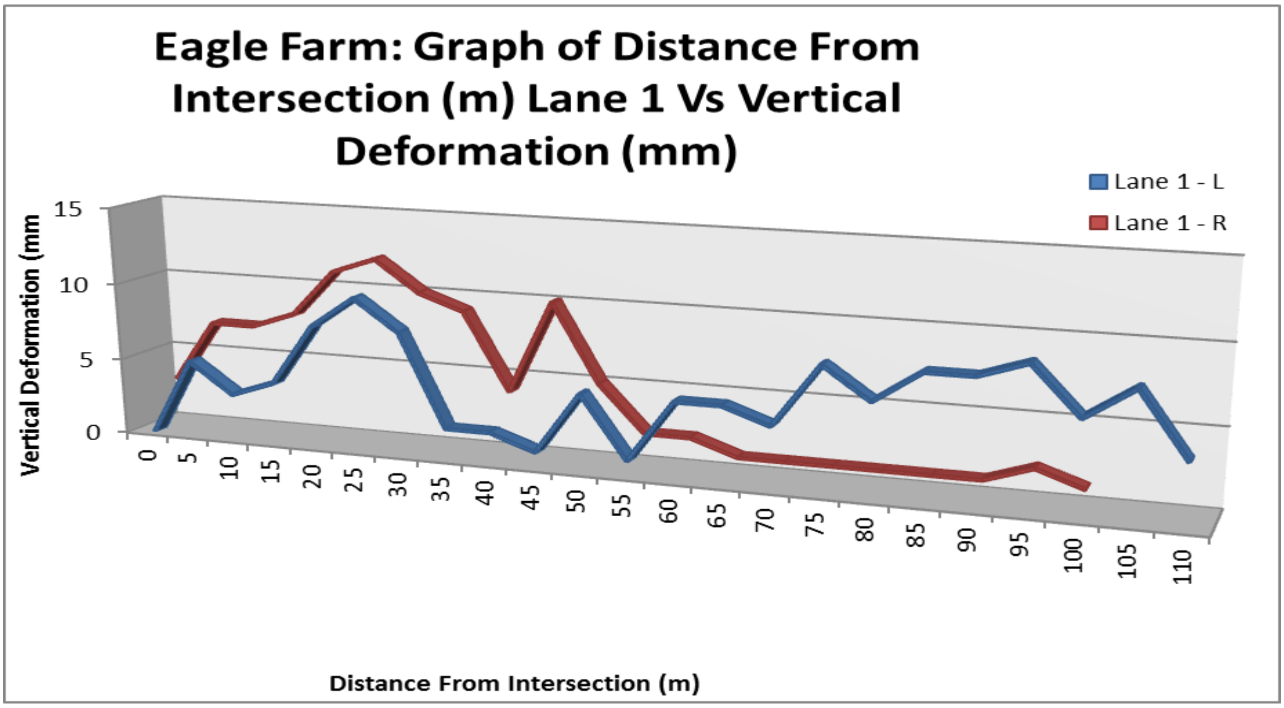

Figure 4-9: Eagle Farm: Graph of Distance From Intersection (m) Lane 1 Vs Vertical Deformation (mm)

Lane 1 (the deceleration lane) deformation matches very closely with the road conditions and braking patterns. As discussed (Section 3.4.4), the approach to the intersection has a right hand bend approximately between 110 m and 65 m from the intersection. As the vehicle rounds the bend more of the load is transferred to the Lane 1-L wheel track. From inspection of the graph it can be seen that the areas where loads are transferred to Lane 1-L wheel track vertical deformation is greater. This observation seems reasonable as the shakedown theory suggests i.e. the larger the repetitive load the greater the deformation.

Once vehicles round the bend it can be observed, on the movies, that braking then commences. By inspection of the graph between 0 m and 55m the vertical deformation in both lanes is very similar and reflects closely the polynomial shear shape identified in Section 2.3. It should be noted, however, that the curve appears closer to the intersection than identified in the literature review and when compared to the other sites. As discussed in Section 3.4.4 and on inspection of the movies vehicles braked hardest, and generated the most shears, closer to the intersection due to the anticipating of driving through the intersection. If required to stop, the vehicles braked hardest between 10 m and 35 m from the intersection.

Therefore, the graph of deformation verses distance for Lane 1 at Lavarack Avenue, Eagle Farm shows another good example of increasing vertical deformation in areas of increasing shear.

Graph of distance from intersection (m) verses vertical deformation (mm) for the acceleration lane at the Eagle Farm site is shown below in Figure 4-10.

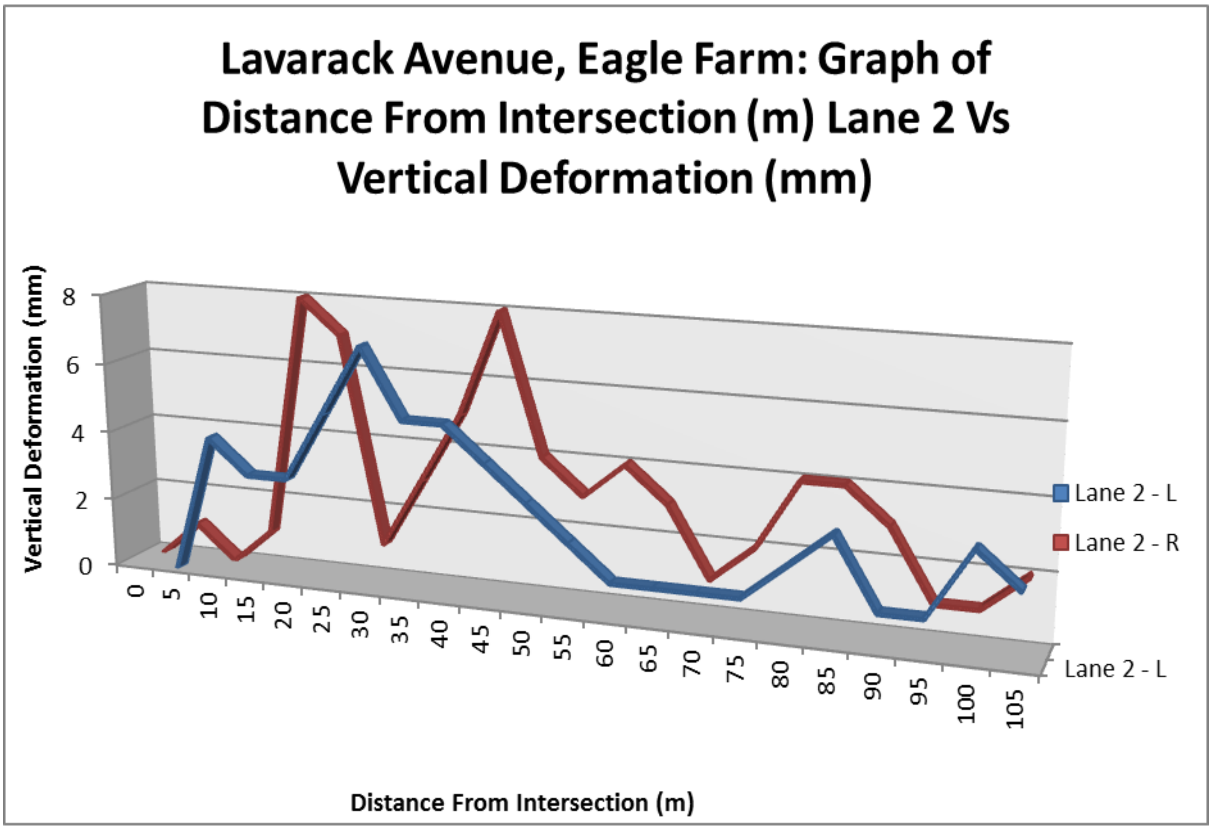

Figure 4-10: Eagle Farm: Graph of Distance From Intersection (m) Lane 2 Vs Vertical Deformation (mm)

Lane 2, the acceleration lane, displays dynamic patterns where vehicles change gears. These areas can be identified in the movies and in the graph above between approximately 20 m and 35 m, and between 70 m and 90 m. At Eagle Farm in the accelerating lane identifying patterns of increasing vertical displacement with distance is less obvious. In fact, by just looking at the graph it would be difficult to draw any such conclusion.

A cause for this can be seen in the movies as vehicles accelerate on Lane 2. These show that when vehicles enter Lane 2 they area already traveling at close to a comfortable speed and consequently required little acceleration within the distance investigated. One of the other factors dictating the speed of vehicles when accelerating is the approaching corner. This results in vehicles accelerating from approximately 40kM/hr up to 60 km/hr over a distance of more than 100 m. Consequently, the surface shears generated at the tyre/pavement interface over the investigated area would be considered to be in the lower range of expected Mu. Thus little or no additional vertical deformation would be expected.

Another factor that may contribute to the lack of apparent association between shear and deformation may be that the number of vehicles, particularly heavy vehicle, that traffic the site. This number is low when compared to the other sites and the low traffic flow may contribute to this lack of apparent association.

From inspection, the graph displays no obvious increase in vertical deformation in areas of high surface shear. However, with the low traffic flow and low Mu the lack of additional vertical deformation may actually support the project finding. For example, low shear should result in low additional vertical deformation.

4.6 Beaufighter Avenue, Archerfield

Below are the vehicle counts, design and vertical deformation results for Beaufighter Avenue, Archerfield.

4.6.1 Vehicle count

Vehicle counts Beaufighter Avenue, Archerfield is shown below for both lanes 1 and 2 in Table 4-6 below.

Beaufighter Avenue, Archerfield Lane 1 Counted on 5 May 2011 at 8:20am - 9:20am									
Cass 1	Class 2	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10
23	0	16	10	4	0	0	2	11	9
Beaufighter Avenue, Archerfield Lane 2 Counted on 5 May11 Time: 8:20am - 9:20am									
Cass 1	Class 2	Class 3	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10
18	0	7	5	3	1	0	1	3	4

4.6.2 Design

Design information for Beaufighter Avenue could not be obtained as the road was a private road. However from the traffic count the pavement would be expected to comprise the following (as a minimum) determined from Figure 8.4 - Design Chart for granular pavements with thin bituminous surfacing on Page 107 in Ausroads Guide to Pavement technology Part 2: Structural Design;

Less than 40mm thick asphalt;

420mm of unbound gravel;

Subgrade with a modulus of 50 MPa

The Calculated ESA values were between 2.6x10^6 and 3.5x10^6

4.6.3 Vertical Deformation

Graph of distance from intersection (m) verses vertical deformation (mm) for the deceleration lane at the Archerfield site is shown below in Figure 4-11 below. As discussed previously the site was level and most vehicles trafficking the site came from a service station 60 m from the intersection. Mu values at the site were lowest out of all the selected sites.

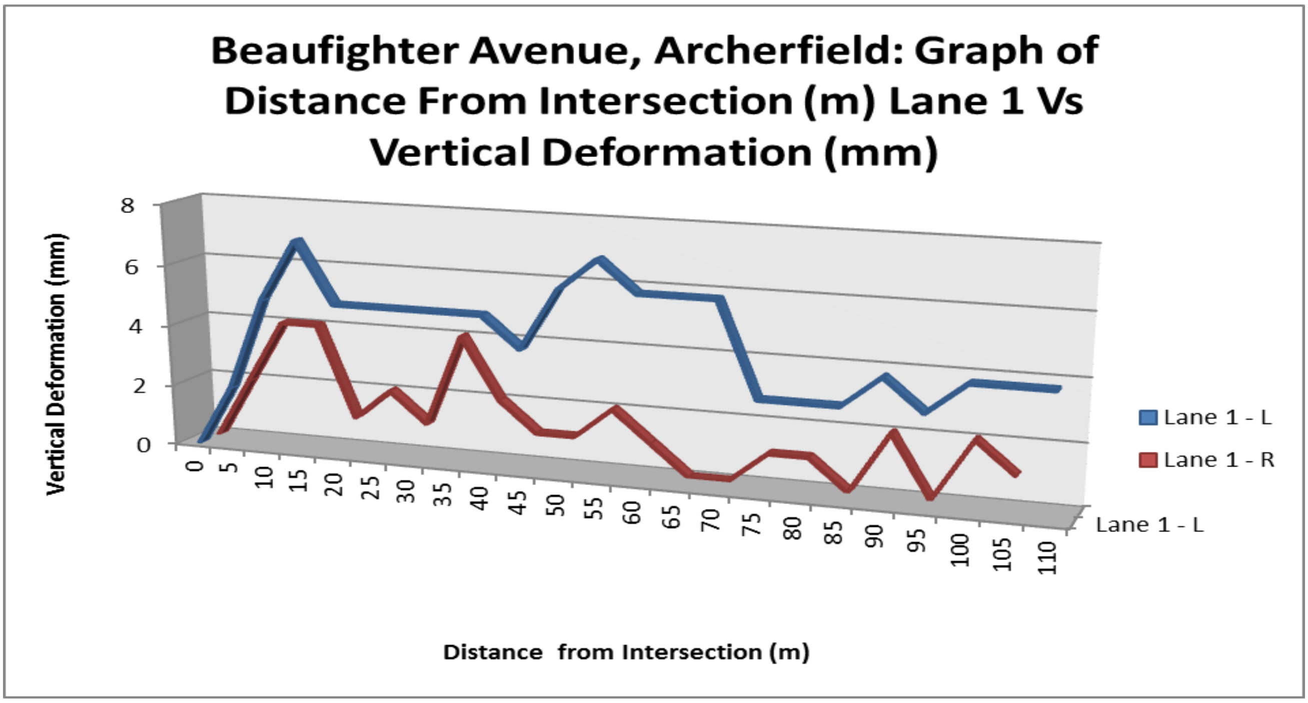

Figure 4-11: Eagle Farm: Graph of Distance From Intersection (m) Lane 1 Vs Vertical Deformation (mm)

Both Lane1-L and Lane 1-R display similar vertical deformation patterns however, the deformation in Lane 1-L is relatively larger. This may be a result of the increase in wheel load on the outer wheel due to the slope of the road. Around 105 m deformation is relative moderate and increase in both wheel lanes at around 70 m. At this distance it can be observed in the movies that vehicles begin deceleration. As the majority of vehicles accelerated out from the service station, vehicles were generally either accelerating or braking along the investigated area.

The observed increase in deformation between 70 m and 15 m is therefore located where the pavement experiences the highest surface shear, and in the case of Beaufighter Avenue Mu appears relatively constant along this length. This is due to the low change in speed needed as the vehicles approach the intersection.

Graph of distance from intersection (m) verses vertical deformation (mm) for the acceleration lane at the Archerfield site is shown below in Figure 4-12.

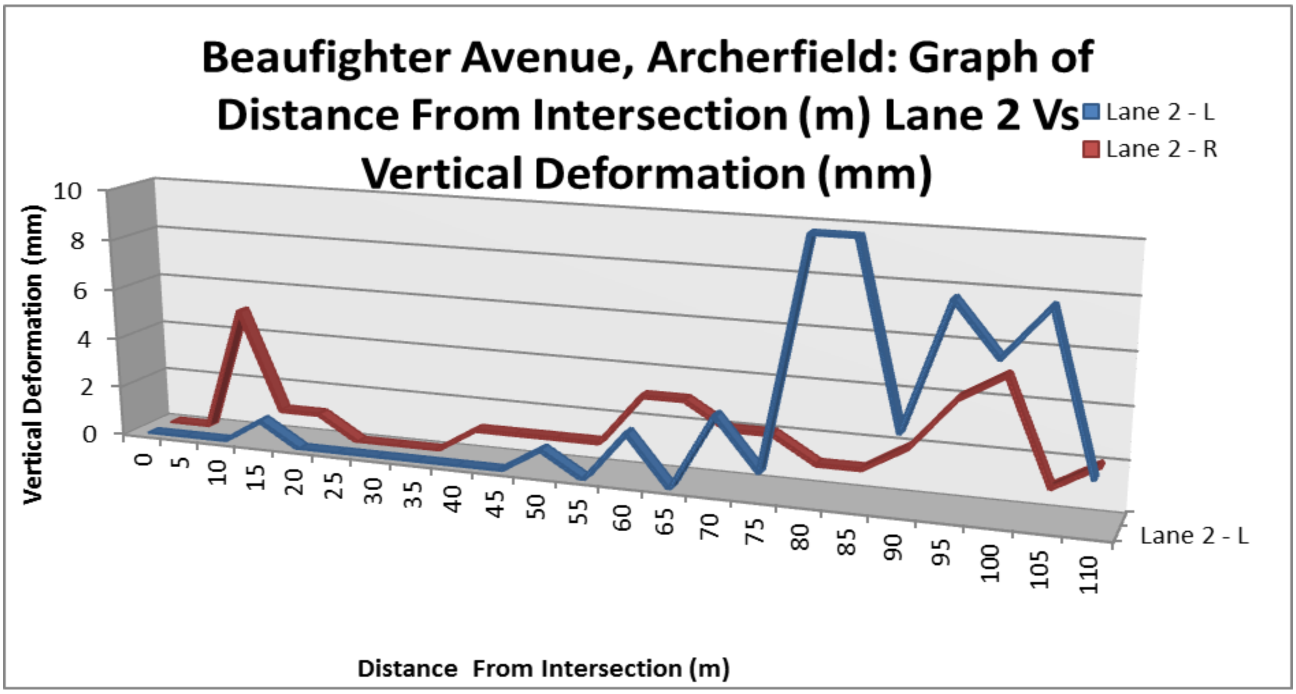

Figure 4-12: Eagle Farm: Graph of Distance from Intersection (m) Lane 2 Vs Vertical Deformation (mm)

Results from Lane 2 –L show some large deformation between 105 m and 80 m. On inspection the photos show this area seems to have a wet surface. The days preceding the investigation some moderate rainfall fell in the area. Other sections of road appeared dry. The wetter surface may indicate an increase of subsurface moisture. The cause of the large deformation at this location is speculative but may come from poor subsurface drainage and/or water ingress through the asphalt into the pavement gravel.

With the exception of the peaks around 10 and 60 m the vertical deformation between 75 m and 0 m is on average less than 3 mm. A reason for lack of vertical deformation could be due to the low traffic volume and low Mu.

4.7 Parametric Study Results

After the verification of the model, both the homogenous and idealised modes were analysed for a stationary vehicle (no horizontal stress). The value of maximum shear stress in the homogenous model was approximately 22% of the vertical stress while the maximum shear stress in the idealised model was between 42% and 48% of the vertical stress. In other words the value of maximum shear is approximately twice as big (for a stationary vehicle) in the idealised model when compared with the homogenous model.

One of the other things to note when no horizontal stress is applied, is that the position of maximum shear is approximately 200 mm below the surface for the homogeneous material and within the top 50 mm in the idealised model. The top 400 mm in the idealised model (50 mm asphalt and 350 mm gravel) is a lot stiffer than the homogenous material model, and this could account for the shear stresses being higher near the surface.

A comparison between the percentage increases in shear stress with changes in Mu was also made. Figure 4-13 below shows this relationship for both models. A value

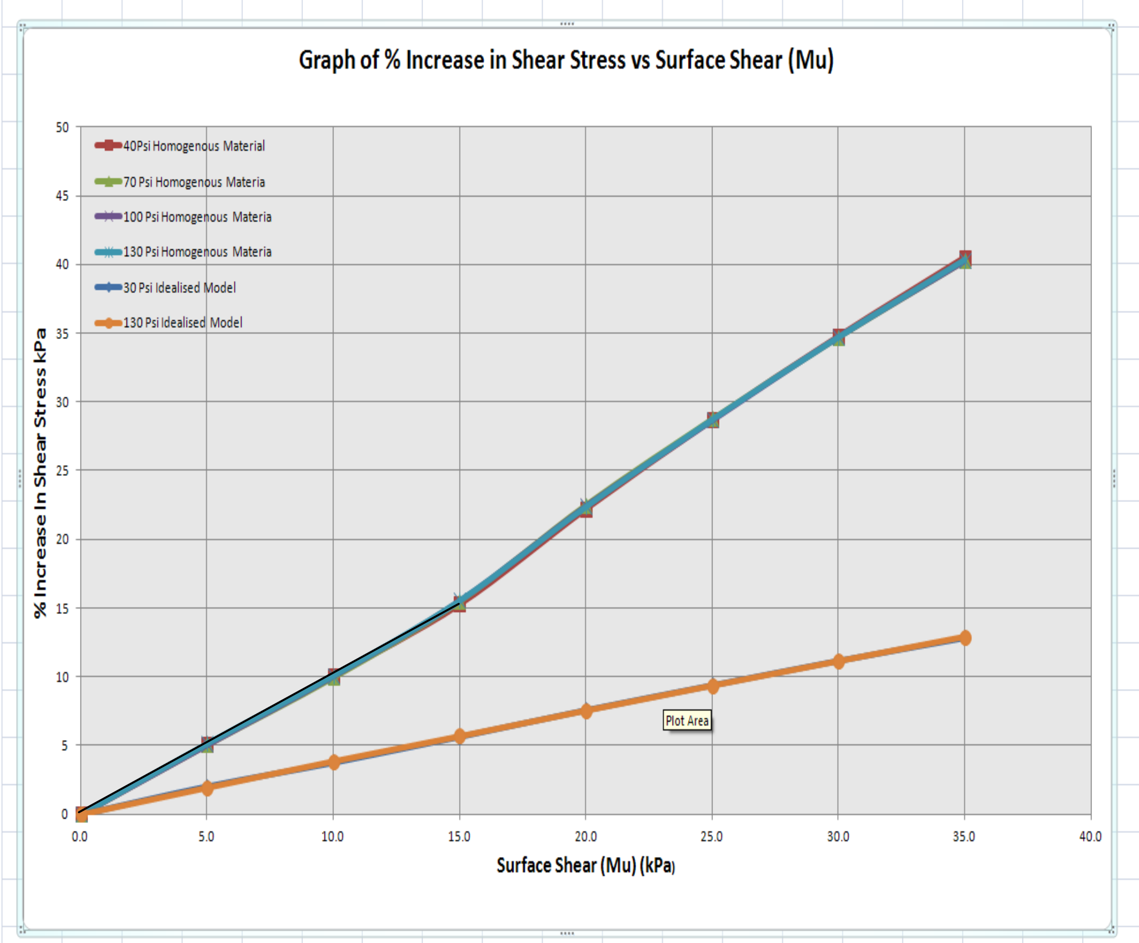

Figure 4-13: Graph of % Increase in Shear Stress Verses Surface Stress

As discussed in the limitation section, which after the model was created and the results formulated it was realised that the mesh size near the surface was too large to determine the value of maximum surface shear with a high degree of accuracy. In the above graph the slightly wavy shape in the homogenous material results from 0% to 15% is more likely to be linear with the wavy shape resulting from the large mesh size.

If we first look at the results from the homogenous material between 0 % and 15% it can be seen that a percentage increases in surface stress results in the same percentage increase in shear stress. For example, if a tyre pressure of 275 kPa has an increase of 5 % surface shear from 14 kPa to 28 kPa, the result is a 5 % increase in shear stress from 64 kPa (0.22x275+0.05(0.22x275) to 67 kPa (0.22x275+0.1(0.22x275). After an increase of 15 % surface shear (Mu) the increase in shear stress is around 6.7 % where the % change in shear stress then decreases linearly to around 5.5% for every 5% increase in surface shear until the maximum surface shear value of 35%.

The below shear stress contours in Figure 4-14 and 4-15 below may help to explain this behaviour. As the surface shear (homogenous model) increases from 0 % to 15 % the position of maximum shear stress gets closed to the surface. For example, without a surface shear applied to the surface the maximum shear force (xy) is located approximately 200 mm below the surface. When the maximum surface shear is applied the position of maximum shear force is located very close to the surface. When the position of maximum shear force is below the surface any increase in surface shear results in an equal percentage increase in the maximum shear stress. When a surface shear of greater than 15% is applied to the surface the position of maximum shear force is near the surface and results in increasing shear stress until reaching (Mu of) 20%. Surface shears greater than 20% appear to begin to stabilise then decrease.

However, if we look at the results of the idealised model with a percentage increase of 2%, and decreasing with increasing Mu, the increase and location of the maximum shear stress appears to be dependent on the material properties (ie layers, thickness, and stiffness). As discussed previously, the location of the maximum shear stress is located near the surface when no horizontal stress is applied.

In terms of what happens in the homogenous material it could be concluded that when vehicles impose a surface shear, then any percentage (of the horizontal stress or Mu) increase will result in an increase of shear stress of between 5% and 7%. Moreover, the position of maximum shear stress changes from a depth of approximately 0.5 of the tyre width (TW) to the surface when Mu is 0.2 or greater.

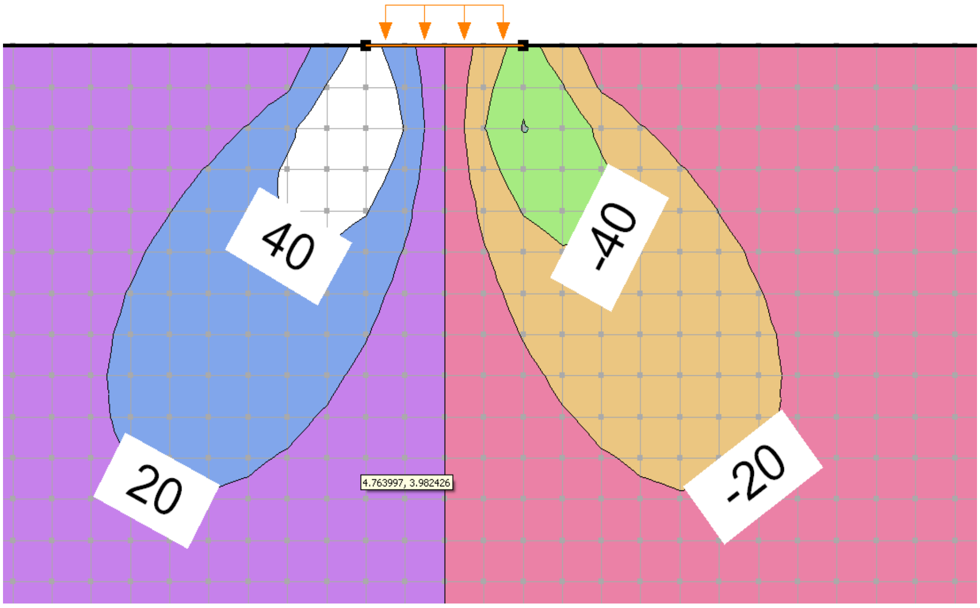

Figure 4-14: 276 kPa Vertical Stress and 0 kPa Horizontal Stress

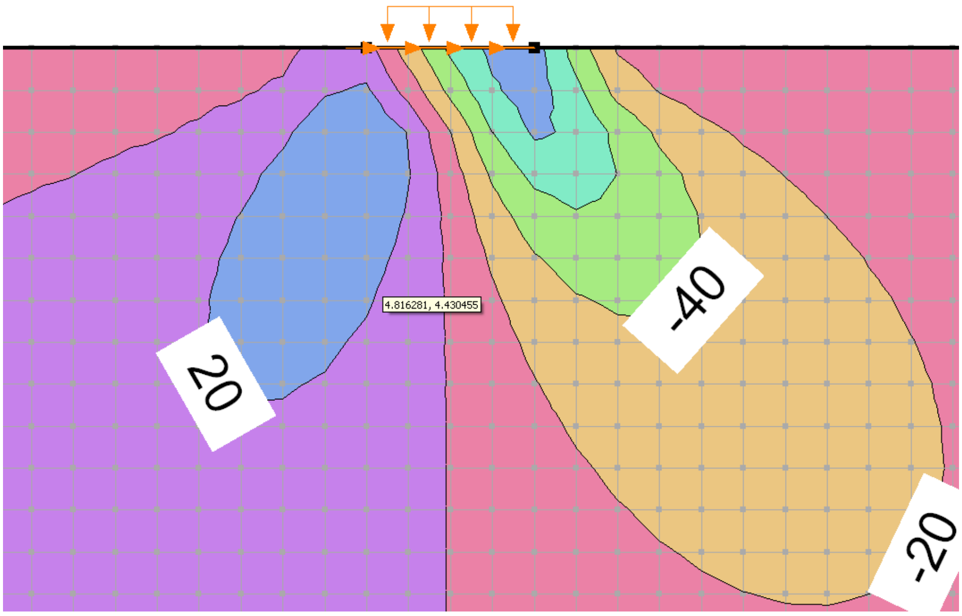

Figure 4-15: 276 kPa Vertical Stress and 97 kPa Horizontal Stress

Looking at results of the idealised model (Figure 4-13) it can be seen that a 5% increase in surface shear result in an increase of 2% of the maximum shear stress. As the surface stress is increased the percentage increase in maximum shear stress gradually decreases from 2% at o kPi surface shear to 1.7% at 35% surface shear. One of the differences between the homogenous model and the idealised model was the position of the maximum shear stress between Mu of 0.0 and 0.35. In the homogenous material at Mu=0 the position of maximum shear is around 0.5TW and rises to the surface at around Mu of 0.15. In the idealised model, the position of maximum shear is near the surface at all Mu values, including Mu=0.

This is could be due to the profile with the stiffest material located at the surface and becoming less stiff with depth. Asphalt, at the surface, has an E value of 3500 kPa and provides a protective layer over the weaker base-course and sub-grade material. The pavement contours show that the stiffer material at the surface caused the position of maximum shear stress to be located at a higher depth within the pavement. Beneath the asphalt is the base-course material and sub-base with an E value of 500kPa and 350kPa respectively.

Another interesting observation is that when values are compared between both models, the values of maximum shear in the idealised model are between 120% (Mu=0) and 80% (Mu=0.35) greater than the homogenous model.

Practically, it could be concluded that a 5% increases in surface shear results in a 2% increase in the maximum shear stress within a typical pavement. This also means that, bigger vertical stresses result in bigger shear stresses and in turn causes greater deformation.

Chapter 5

Conclusion

The stresses and strains developed at the tyre-pavement interface, as moving vehicles traffic on roads, are complex and dynamic and highly dependent upon numerous factors. The interaction is dependent on pavement material and construction techniques, vehicle type and size, tyre and pavement characteristic, and speed and acceleration of the vehicle. Consequently, pavement design is based on a number of simplifications that allow engineers to design pavements based on the DESA and material properties.

This project studied the reaction of pavements in areas that experience high surface shear or in other words, looked at the pavement response in areas that experience differing degrees of acceleration and deceleration imposed by heavy vehicles. Additionally the project assesses the appropriateness of current design methods. To achieve this objective, four sites were examined for assessment and parametric studies carried out to determine the theoretical pavement response to imposed tyre stresses.

The findings confirm that vehicles accelerating and decelerating impose additional stresses within the pavement. However, the increase is approximately 2% above the shear stress imposed when the vehicle is traveling at a constant speed and consequently only result in slight additional vertical deformation. In a typical pavement, with higher modulus material at the surface, the maximum shear stresses were found to be close to the surface. It was also found that where acceleration/deceleration is low (low Mu) little additional vertical deformation is likely to occur and similarly, in areas where acceleration/deceleration is high (high Mu) vertical deformation is likely to be increased. Also, deceleration rates are generally greater than acceleration.

Another feature in areas where vehicles accelerate/decelerate is the dynamic response when changing gear. When the gear is disengaged/engaged there is a short period where the load is redistributed back and forth until dampened by the suspension. This causes a dynamic increase in vertical stress and consequently results in additional (peak) vertical deformation where vehicles are likely to change gears.

Vertical deformations in areas that experience high surface shear were however within the limits set by Austroads. Consequently, in terms of pavement permanent strain all investigated roads performed satisfactorily. Asphalt deterioration was also considered however, little difference was observed between the areas that experience high surface shears and the areas that experienced low surface shears. There was however one exception to pavement suitability and this was in areas where vehicles turn sharply (corners). These areas generally show signs of severe pavement deterioration i.e. potholes.

Where design data was obtained, it was concluded that the pavements were designed in accordance with Austroads specification and that the current practices are suitable for areas that experience surface shears, imposed by the accelerating/deceleration of vehicles.

Based on the findings of this project, no design improvements are recommended. Although vertical deformation were well under tolerable limits no improvements should be considered without further research and consultation with Austroads.

Chapter 6

Further Study

Further understanding of the tyre-pavement interaction could be of practical use in the design and construction of pavements in areas of high surface shear. In turn, improved design and construction has the potential to decrease costs while at the same time maintaining performance.

The three areas studied in this project (namely the field study, literature review and the parametric study) can all be built upon in further studies. It is considered that increasing the understanding in each of these areas could in provide a foundation into better design, construction and pavement performance. The following subsections detail some of areas that could be studied further.

5.1 Additional Field Studies

The benefit of undertaking field studies is that they provide uncontrolled data that can be used to assess every stage of the pavement life cycle from the investigation and design phase to the long term performance of pavements. Gathering this sort of information could help to understand the complexity and variability of the pavement response.

In terms of field studies, it is clear that additional sites investigated could be carried out in order to build upon the current knowledge base. These additional investigations could be carried out within areas that experience different surface reactions and different ages of the pavement. Different surface reaction could include sites that experience high surface shear, light traffic, lower or higher travel speeds (different Mu rations) and known pavement configurations.

To improve considerably on this project a better pavement measuring technique can be used. Ideally, purpose built sensors can be attached to vehicles and take measurement across the whole width and length of the road in a single pass and without hindering traffic flow with no increase in personal safety risk. Devices have also been developed (seismic measuring devices) that can provide information on the sub-surface strata and this may be used to assess permanent strain in each layer. This project assessed surface vertical deformation at the most trafficked wheel track. By collecting information over the whole road (surface and sub-surface) valuable comparisons could be made

Another useful field study would be to look at the total pavement life cycle, which would include every phase. It would begin with the investigation phase and how soil data was investigated and interpreted followed by the construction phase where this information is put into action. The focus at this stage (action) would be on construction techniques and the different between the assumptions in the design stage and the site conditions and construction techniques. This would be followed by a similar study but over a longer period of time. Deformation information could be collected at the commissioning and continue to be monitored for a period of time.

5.2 Parametric Studies

As discussed in Section 3.1 the parametric study is a static, elastic 2D analysis. Clearly, undertaking parametric analysis/modelling that is closer to real life scenarios would provide more usefully information. Undertaking a parametric study that is dynamic, plastic/elastic and 3D may provide a better understanding of the tyre-pavement interaction. The model could also simulate the negative stresses imposed at the rear of the tyre and have a ribbed tyre contact area.

5.3 Literature Review

Further study into the appropriateness of industry design practices could be undertaken. While the availability of technical resources on pavement design is vast, it would be valuable to collect information into current industry road design practices used by design engineers. It is expected that design procedures would not deviate far from published resources and Australian Standards, however finding out about design practices and rules of thumb may be useful for the evaluation of pavement performance and construction.

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Appendix A – Project Specification

University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

ENG4111/4112 Research project

PROJECT SPECIFICATION

FOR: **Ryan Jay Kemp**

TOPIC: Investigates the effect of surface shear on pavements near road intersection.

SUPERVISORS: Jim Shiau

ENROLMENT: ENG 4111 S1, 2011

ENG 41112, 2001

Project aim: This project seeks to study the effect of surface shear on the performance of road pavements. The appropriateness of the current design methods will be investigated using collected road data and theoretical analysis.

Program: (Issue A, 19 February 2011)

1) Research current Australian and international pavement design methods focusing on intersection pavement design and the consideration of shear force on pavement design.
2) Collect data, pictures and videos to investigate current pavement performance near intersections, stop/give way signs…etc. Say minimum 5 intersections and their respective analyses.
3) FE analysis (Sigma/W) – elastic/static analysis considering the effect of large vehicle surface shears.
4) Carry out parametric studies.
5) Determine if current design methods have been used at the investigated intersections, the appropriateness the design and the consideration of shear force in the design method.
6) Present findings based on the theoretical approach.
7) Make a number of suggestions with respect to the study in (1), (2), (3) and (4)

Agreed _________________________ (Student)_____________________(Supervisor)

Date: / / 2011 Date / / 2011

Appendix B - Archerfield Lane 1-L Photos

5m

10m

15m

20m

25m

30m

35m

40m

45m

50m

55m

60m

65m

70m

75m


80m


85m


90m


95m


100m


105m


110m

Appendix C - Archerfield Lane 1-R Photos

5m

10m

15m

20m

25m

30m

35m

40m

45m

50m

55m

60m

65m

70m

75m

80m

85m

90m

95m

100

105

Appendix D - Archerfield Lane 2-L Photos


5m


10m


15m


20m


25m


30m


35m


40m


45m


50m


55m


60m


65m


70m


75m

80m

85m

90m

95m

100m

105m

110m

Appendix E - Archerfield Lane 2-R Photos

5m

10m

15m

20m

25m

30m

35m

40m

45m

50m

55m

60m

65m

70m

75m


80m


85m


95m


100m


105m


110m


115m

Appendix F - Darra Lane 1-L Photos


5m


10m


15m


20m


25m


30m


35m


40m


45m


50m


55m


60m


65m


70m


75m

80m

85m

90m

95m

100m

105m

110m

115

120m

125m

130m

Appendix G - Darra Lane 1-R Photos


5m


10m


15m


20m


25m


30m


35m


40m


45m


35m


40m


45m


50m


55m


60m

80m

85m

90m

95m

100m

105m

110m

115m

120m

125m

130m

135m

140m

Appendix H - Darra Lane 2-L Photos

10m

15m

20m

25m

30m

35m

40m

45m

50m

55m

60m

65m

70m

75m

80m

85m

95m

100m

105m

110m

115m

120m

125m

130m

135m

140m

Appendix I - Darra Lane 2-R Photos

10m

15m

20m

25m

30m

35m

40m

45m

50m

55m

60m

65m

70m

75m

80m

85m

90m

95m

100m

105m

110m

115m

120m

125m

130m

135m

140m

145m

Appendix J – Eagle Farm Lane 1-L Photos

0m

5m

10m

15m

20m

25m

30m

35m

40m

45m

50m

55m

60m

65m

70m

75m

80m

85m

90m

95m

100m

105m

110m

Appendix K – Eagle Farm Lane 1-R Photos

0m

5m

10m

15m

20m

25m

30m

35m

40m

45m

50m

55m

60m

65m

70m

75m

80m

85m

90m

95m

100

Appendix L – Eagle Farm Lane 2-L Photos

5m

10m

15m

20m

25m

30m

35m

40m

45m

50m

55m

60m

65m

70m

75m

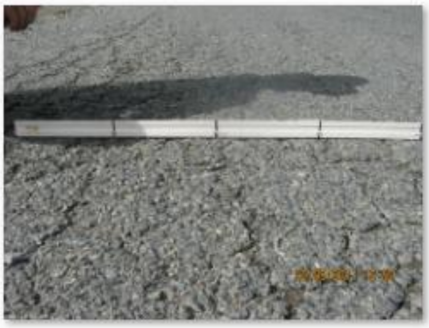
80m

85m

90m

95m

100m



105m

Appendix M – Eagle Farm Lane 2-R Photos

5m

10m

15m

20m

25m

30m

35m

40m

45m

50m

55m

60m

65m

70m

75m

80m

85m

90m

95m

100m

105m

Appendix N – Wacol Lane 1-L Photos


1m


4m


8m


12m


16m


20m


24m


28m


32m


36m


40m


44m


48m


52m


56m

60

64m

68m

72

Appendix O – Wacol Lane 1-R Photos


1m

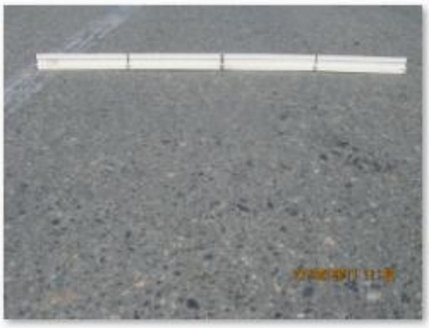

4m


8m


12m


16m


20m


24m


28m


32m


36m


40m


44m


48m


52


56

60

64

68

72

Appendix P – Wacol Lane 2-L Photos


1m


4m


8m


12m


16m


20m


24m


28m


32m


36m


40m


45m


50m


55m


60m

65m

70m

75m

80m

85m

90m

95m

100m

105m

110m

Appendix Q – Wacol Lane 2-R Photos

1m

4m

8m

12m

16m

20m

24m

28m

32m

36m

40m

45m

50m

55m

60m

65m

70m

75m

80m

85m

90m

95m

100m

Appendix R – Wacol Lane 2-R Photos


2m


5m


10m


15m


20m


25m


30m


35m


40m


45m


50m


55m


60m


65m


70m

75m

80m

85m

90m

95m

100m

105m

Appendix S – Wacol Lane 2-R Photos

2m

5m

10m

15m

20m

25m

30m

35m

40m

45m

50m

55m

60m

65m

70m

75m

80m

85m

90m

95m

100m

Appendix T – Project Plan

Timeline Project Eng4111/4112 2011, Ryan Kemp Student Number: 0050039489

Week	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
Week Starting at	28-Feb-11	07-Mar-11	14-Mar-11	21-Mar-11	28-Mar-11	04-Apr-11	11-Apr-11	18-Apr-11	25-Apr-11	02-May-11	09-May-11	16-May-11	23-May-11	30-May-11	06-Jun-11	13-Jun-11	20-Jun-11	27-Jun-11	04-Jul-11	11-Jul-11	18-Jul-11	25-Jul-11	01-Aug-11	08-Aug-11	15-Aug-11	22-Aug-11	29-Aug-11	05-Sep-11	12-Sep-11	19-Sep-11	26-Sep-11	03-Oct-11	10-Oct-11	17-Oct-11	24-Oct-11	31-Oct-11	07-Nov-11	14-Nov-11	21-Nov-11	28-Nov-11
Task																																								
Weekly report	X	X	X	X	X	X	X	X	X	X	X	X	X																											
Project Specification			X	Submitted 19 March due 22 March																																				
Saftey Issues												X SWMS completed																												
resource Requirements						X																																		
Collect Field Data						Movies, Photographs and pavement information 5 sites																																		
Collect research Data																																								
Anaysis on SigmaW																																								
Progect Appreciation																																								
Analise Results																																								
Submit extended Abstract																																								
ENG4903 Professional Practice 2																																								
Dissertation																																								

X completed task
timeline for task
assessments and work required to hand in

Appendix U - SWMS

PAVEMENT INVESTIGATION FIELD WORK	SWMS 001

Project	Research Project ENG4111/4112
Topic	Investigates the effect of surface shear on pavements near road intersections

Equipment	Vehicle ☒	Camera, movie camera plus tripod ☒	Measuring equipment ☒
	Field sheets ☒		

Personal Protective Equipment ☒ to be worn all the time while on site		
☒ Safety steel cap footwear must be worn at all times in work areas.	☐ A hard hat must be worn.	☒ High visibility clothing must be worn. No loose clothing.
☐ Hearing protection must be worn when using this machine.	☐ Gloves must be worn (unless it is unsafe to do so - demonstrated by a risk assessment).	☐ Long and loose hair must be contained.
☒ Safety/sun glasses must be worn.	☐ A dust mask or ☐ respirator (depending on the hazard) must be worn when hazardous substances are present.	☒ Sunscreen and wide brim must be worn to protect against ultraviolet radiation.
☒Long sleeve shirt and pants or overalls required.		

OHS Risk Assessment Matrix	***PROBABILITY*** *HOW LIKELY IS IT TO HAPPEN?*			
CONSEQUENCE HOW BAD COULD IT BE?	A. Highly likely: could occur at any time	B. Likely: could occur sometime	C. Unlikely: could occur, but very rarely	D. Very unlikely: could occur, but probably never will
1. Kill or cause permanent disability or ill health	**1**	**2**	**4**	**7**
2. Long term illness or serious injury	**3**	**5**	**8**	**11**
3. Medical attention and several days off work	**6**	**9**	**12**	**14**
4. First aid needed	**10**	**13**	**15**	**16**

HIERARCHY OF CONTROLS	
ELIMINATION	Removing the hazard, or practice, altogether.
SUBSTITUTION	Using a safer alternative, eg less hazardous alternative.
ENCLOSE OR ISOLATE	Use barriers around plant, enclosing noisy process, removing the hazard from the area
ENGINEERING	Guarding, isolation/cutout devices
ADMINISTRATIVE	Safe work procedures, instruction, training, signage.
PPE	Worn to safe guard against hazards.

RISK RATING	
1-3	**Critical Risk –** Immediate action is required. The activity must not proceed and an alternative safer method of work is required or additional controls must be implemented to reduce the likelihood and/or consequence.
4-6	**High Risk –** Important to act on this very soon. The activity must not proceed, review the risk controls based on the hierarchy of controls and add additional controls to reduce the risk level. A documented system of work including standard operating procedures, training, monitoring and supervision is required. Continued exposure would only be considered in exceptional circumstances, the decision to do so should be made at Principal Level after due consideration of cost versus benefit. Any decision to continue the exposure to the risk must be subject to continuous review.
7-10	**Medium Risk –** Risk control measures are required. Exposure to the risk may continue provided it has been appropriately assessed, has been mitigated to As Low As Reasonably Practicable (ALARP), and is subject to periodic review to ensure the risk does not increase.
11-16	**Low Risk –** Action taken when possible/manage by procedures. Exposure to the risk is acceptable, but is subject to periodic review to ensure the risk does not increase.

N0.	***Step/Activity***	***Hazard***	***Risk Rating***	***Control Measure (or Safe Work Procedure reference)***	***New Risk Rating***	***Action by***	***Comment (OK or concern)***
1	Establishment to site	Incidents/accidents caused while driving	12	Safe driving techniques	12	RK	OK
2	Set up movie camera and filming vehicle travel	Being struck by an oncoming vehicle.	7	Set up camera near existing sign and at least 1.0 m away from curb. Keep vigilance. Do not carry out activity if fatigued.	11	RK	OK
3	Map defects	Being struck by an oncoming vehicle.	7	Observe and map defects from side of road. Keep vigilance. Do not carry out activity if fatigued.	11	RK	OK
4	Photograph pavement	Being struck by an oncoming vehicle.	4	Only undertake activity on weekends and times of low traffic flow. Do not carry out activity if fatigued. Always use a competent observer. Attempt to always be in the line of site of the vehicle. Always face oncoming traffic when on the road.	11	RK and observer	OK
5	Working on site	Dehydration	12	Keep fluids up. PPE (wide brim hat)	15	RK and Observer	OK

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I, the undersigned, confirm that this SWMS has been explained and its contents are clearly understood.

NAME	SIGNATURE	DATE	STUDENT NUMBER (if applicable)