# 2. VEHICLE-PAVEMENT INTERACTION CONCEPTS

## 2.1 Introduction

Chapter 2 consists of a literature review providing the currently available information for the five main themes addressed in this thesis. These five themes are: vehicle-pavement interaction systems, pavement roughness, vehicle components, pavement components, and vehicle and pavement fingerprinting. The focus on vehicle-pavement interaction systems and pavement components are more detailed than the other three themes as the bulk of the thesis focuses on these two themes. The other three themes are included as vital information from them is needed in the further analyses.

This thesis focuses on the primary transient response of flexible South African pavements to dynamic vehicle loads. It explicitly excludes transfer functions, economic models, pavement performance, rigid pavement response and bridge response.

## 2.2 Vehicle-Pavement Interaction Systems

### 2.2.1 Introduction

Systems methodology comprises the efficient planning, design and implementation of new systems, and the structuring of the state of knowledge or operational modelling of existing systems. A problem-solving process should provide for systematic incorporation of all the factors of interest, and should be a logical simulation of the progression of activities involved in efficiently solving the problem. The system should be clearly recognised and identified, to ensure clarity on inputs, objectives and constraints. A system consists of interacting components that are affected by external factors (Haas et al, 1994).

Vehicle-pavement interaction is the system in which the vehicle and the pavement exert mutual forces on each other. It should describe the vehicle and its components, the pavement and its components, and the way in which all of these components influence each other. This section focuses on the currently used vehicle-pavement interaction systems, and their dominant features and limitations.

### 2.2.2 Types of systems

Three viewpoints towards vehicle-pavement interaction covering the spectrum of available systems can be identified. These originate from the pavement management, vehicle engineering and combined pavement and vehicle engineering fields.

<u>Pavement management viewpoint</u>

This viewpoint has matured through years of development and is best described by Haas et al (1994). It was developed in response to the need to formulate the overall pavement problem in broad conceptual and theoretical terms that would enable the solution of a variety of pavement problems (Figure 2.1).

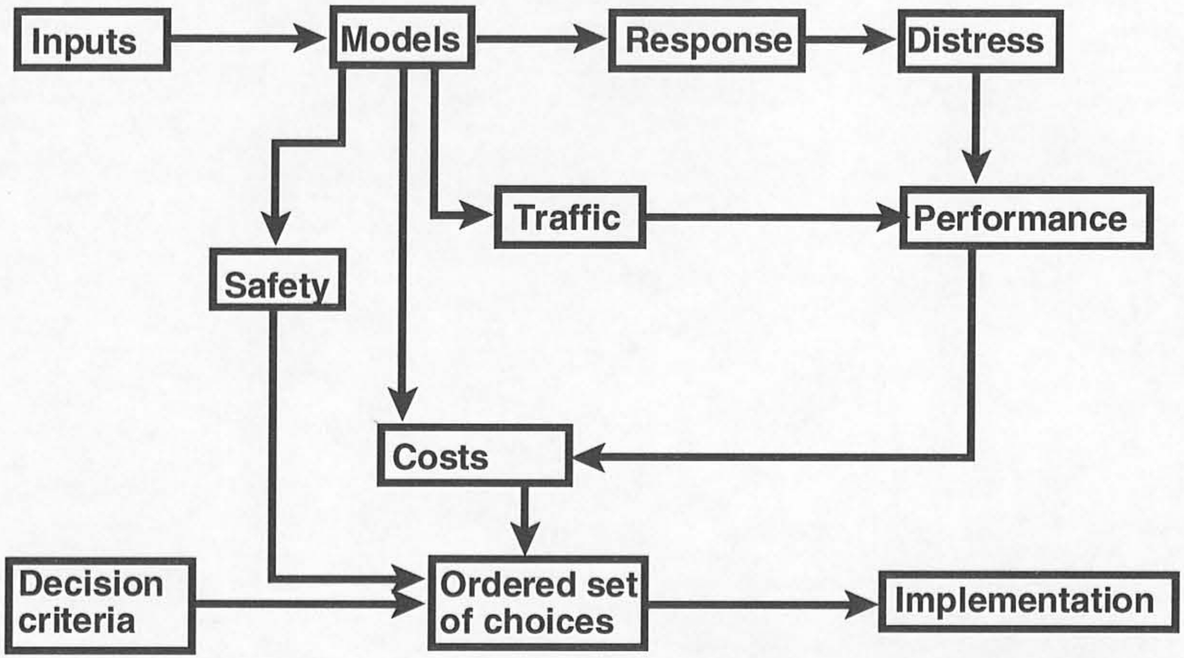



**Figure 2.1:** Pavement Management Systems (PMS) viewpoint to vehicle-pavement interaction (from Haas et al, 1994).

The system consists of eleven components. Four are essential to the vehicle-pavement interaction system, and the remaining seven complete the pavement management system. The four essential vehicle-pavement interaction components are the Inputs and Traffic (pavement and load variables), Models (defining vehicle and pavement reactions) and Response (pavement behaviour). The remaining seven components of Distress, Performance, Safety, Costs, Decision Criteria, Ordered set of choices and Implementation are not directly affected by the interaction between pavement and vehicle variables, but depend on the outcome of the response calculations. This system requires extensive input data to the various behavioural models. The primary response variables are evaluated using limiting and decision criteria, and after enough iterations to provide the required acceptability criteria, a final decision on the pavement design is made.

The dominant features of this viewpoint are its complete view of the overall pavement problem, including all possible input variables, a complete set of decision criteria and a modular approach towards behavioural models.

The main limitations of this viewpoint are the focus on pavement issues and the potentially complicated input data requirements for the behavioural and performance models.

Pavement engineering and the specific factors incorporated in either static or transient pavement analyses forms part of the general pavement management viewpoint. Currently the reaction of the vehicles to the pavement and the transient response of the pavement to tyre loads are generally not evaluated.

Collop and Cebon (1995a, 1995b) developed the Whole-Life Pavement Performance Model (WLPPM) and Ullidtz (1987; 1997) the Mathematical Model of Pavement Performance (MMOP). Both these systems are similar to the PMS viewpoint with slightly more explicit quantification of the interaction between vehicles and pavements. It does however focus on pavement performance issues, and will thus not be discussed in detail. De Pont et al (1998) indicated that the WLPPM provides the opportunity to experiment with different behavioural models within the existing pavement performance analysis framework.

Vehicle engineering viewpoint

This viewpoint is best described by Fancher and Gillespie (1997), Gillespie (1992) and Orr (1988). Vehicle engineers attempt to produce a product with unique advantages compared to other products on the road, while highway designers tend to encourage policies that promote roads with uniform characteristics for the level of service intended. Almost all trucks are different in terms of dimensions and axle loads. Highway laws and design standards have considerable influence on truck design, determining allowable limits on characteristics such as dimensions, loads and tyre tread width.

The weights and dimensions associated with a vehicle combination are the primary parameters by which acceptable vehicles are defined in road use laws. Vehicle-pavement interaction largely depends on the truck characteristics and operational conditions. It is vital to encourage evolution of both the pavement system and the truck transportation system toward ensuring better compatibility, particularly in matters of designing pavements to accommodate heavy vehicles, designing heavy vehicles to operate on those pavements and adopting policies that will lead to improved safety and efficiency of heavy vehicle transportation (Fancher and Gillespie, 1997).

Gillespie (1992) and Orr (1988) view the vehicle-pavement interaction system from a ride dynamic viewpoint, causing pavement roughness to be the only pavement parameter of importance, as this is one of the primary ride excitation sources. One of the main objectives of the vehicle engineer is to provide the driver, occupants and goods in a vehicle with as smooth a ride as possible. Therefore, the vibrations and dynamic response of the vehicle, caused by pavement roughness, is primarily a vehicle design and analysis parameter.

The dominant features of the vehicle engineers' viewpoint systems are the emphasis on the need for interaction between vehicle and pavement engineers, and the effect of pavement regulations on vehicle design. The limitation of these systems is the ignorance of pavement response issues.

<u>Combined pavement and vehicle engineering viewpoint</u>

In the DIVINE (Dynamic Interaction of Vehicles and Infrastructure Experiment) project the vehicle-infrastructure system was considered in an integrated way (Figure 2.2). The interaction between vehicles and pavements, and between vehicles and bridges has been separated, as different variables and mechanisms are at work. A full physical description of all the relationships involved is challenging, due to the complexity of the dynamic systems involved and the fact that some behaviours are spatially-related, some are time-related and some are cumulative with regard to repeated passes of the vehicle (Divine, 1997).

Pavement roughness initiates the vertical dynamics of the moving vehicle and the resulting dynamic tyre loads cause pavement responses that contribute to pavement distress. The pavement responses depend on the combination of dynamic tyre load, speed and local pavement strength, the depth at which the pavement response is measured, and the influence of the tyre contact patch in distributing the dynamic tyre load to the pavement structure. Over time and by repeated loading of heavy vehicles, the profile changes due to vertical surface deformations affecting the dynamic tyre loads. The pavement structural strength changes over time due to the accumulated effects of pavement responses as well as environmental influences. The net effect on pavement responses and distress will depend on the spatial relationship between dynamic tyre loads, pavement profile changes and pavement structural strength variations (Divine, 1997).

The dominant feature of this system is that it was developed together by a group of vehicle and pavement engineers and claim to incorporate all the relevant components. The main limitations of this system are caused by its completeness, as it requires relatively complicated input data and computational effort.

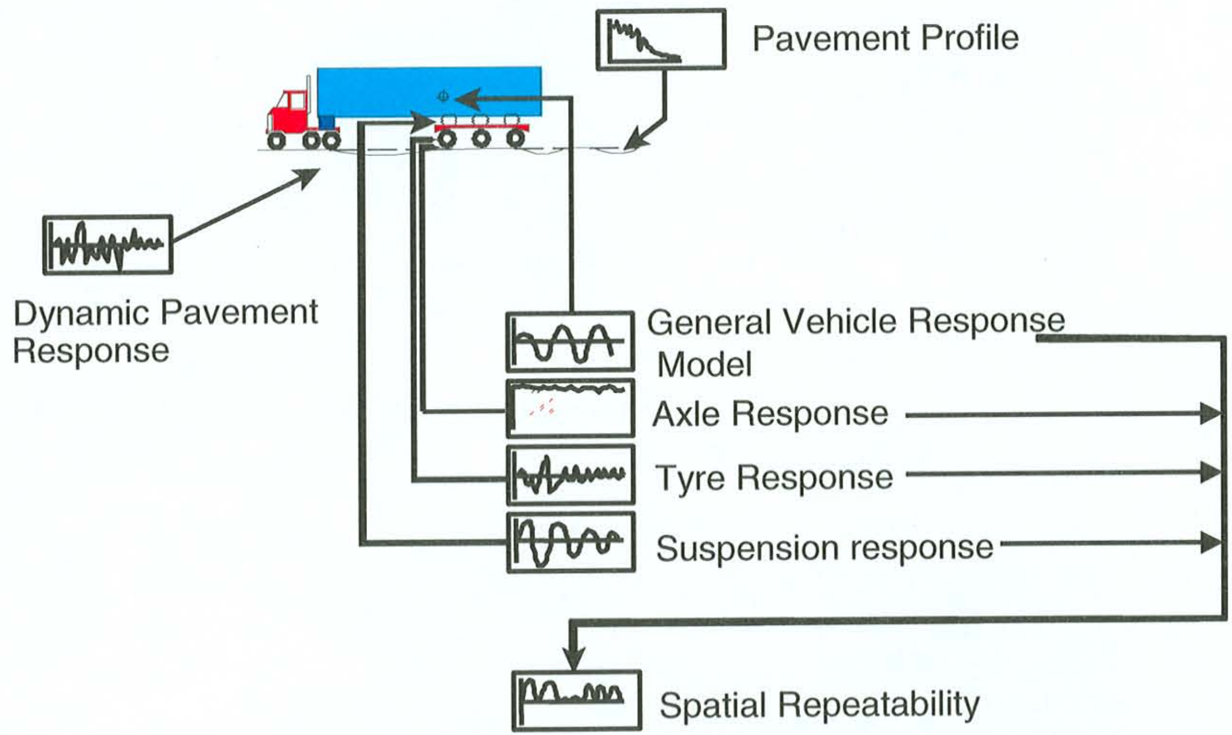



**Figure 2.2:** **Divine approach to vehicle-pavement interaction (Divine, 1997).**

### 2.2.3 Conclusions on vehicle-pavement interaction systems

The dominant features and limitations of the vehicle-pavement interaction systems originating from three viewpoints were discussed. Traditionally pavement and vehicle engineers both view the vehicle-pavement interaction system with bias towards their own knowledge and experience, and only through combined efforts such as DIVINE, and in areas such as the pavement management field, are attempts made to incorporate all the relevant components and interactions into the system.

The following general limitations were identified in most of the systems. A clear distinction does not exist between pavement behaviour and performance issues. It is further possible, due to the modular approach taken in most systems, to use behavioural models at various technology and/or complexity levels in the same analysis. The ultimate effect (beneficial or not) of such combinations is not clear (i.e. least denominator effect). These issues are addressed in Chapter 4.

## 2.3 Pavement Roughness

### 2.3.1 Introduction

The main cause of vehicle induced dynamic loading is the irregularities of the pavement surface (pavement roughness) (Gillespie, 1992). These irregularities cause an irregular input to the vehicle though the tyre-suspension combination. The response of the vehicle to these inputs constitutes the dynamic nature of vehicle loading. In this section pavement roughness definitions, models and data are investigated, specifically regarding the effect on heavy vehicle response.

### 2.3.2 Definitions and Concepts

Pavement roughness is defined as the variation in surface elevation that induces vibrations in traversing vehicles (Sayers et al, 1986a), or as " the deviations of a surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic pavement loads, and drainage, for example, longitudinal profile, transverse profile and cross slope" (ASTM, 1996).

Frequency ranges for various surface characteristics are specified by the PIARC Technical Committee on Surface Characteristics. The roughness frequency range is that range which induces relative motion in road vehicle suspension systems over a reasonable range of operating speeds (McClean and Ramsay, 1996). The frequency range with wavelengths between 0,5 and 50 m is considered best to indicate pavement roughness.

Pavement roughness is one of the prime indicators of the deterioration of a pavement (Sayers et al, 1986b; TRH4, 1996). Roughness indices are used to have a simple value indicating the roughness level and trends in roughness level over time of a specific pavement. These indices are calculated from either the response of a roadmeter to the pavement roughness inputs, or using mathematical equations and measured pavement profiles.

Several roughness indices exist. They do not all measure roughness in the same way, and are not necessarily sensitive to the same types of roughnesses or applicable to the same conditions. The World Bank sponsored a major study of pavement roughness (the International Road Roughness Experiment (IRRE)) during which various methods for obtaining pavement roughness data, analysis of these data and presentation into standard formats were investigated (Sayers et al, 1986a; 1986b). The concept of the International Roughness Index (IRI) was consequently developed.

The IRI roughness scale best satisfied the criteria of being time-stable, transportable, relevant and readily measurable (Sayers et al, 1986a). It is a standardised roughness measurement related to the various response-type road roughness measurement systems (RTRRMS) with units of metre per kilometre (m/km) or millimetre per metre (mm/m). It is widely accepted as the index of choice for reporting pavement roughness. The IRI roughness scale is shown in Table 2.1 (Sayers et al, 1986b).

The true value of the IRI is obtained by obtaining a suitable accurate measurement of the profile of a wheeltrack, processing it through an algorithm that simulates the way a reference vehicle would respond to the roughness inputs, and accumulating the suspension travel. It is calculated at a standard speed of 80 km/h, as pavement roughness is dependent on vehicle speed (Sayers, 1995; Gillespie, 1992). IRI indicates the extent to which the surface of the pavement has deformed with respect to the specific wavelengths that affect the response of a specific vehicle travelling over the road (Mann et al, 1997).

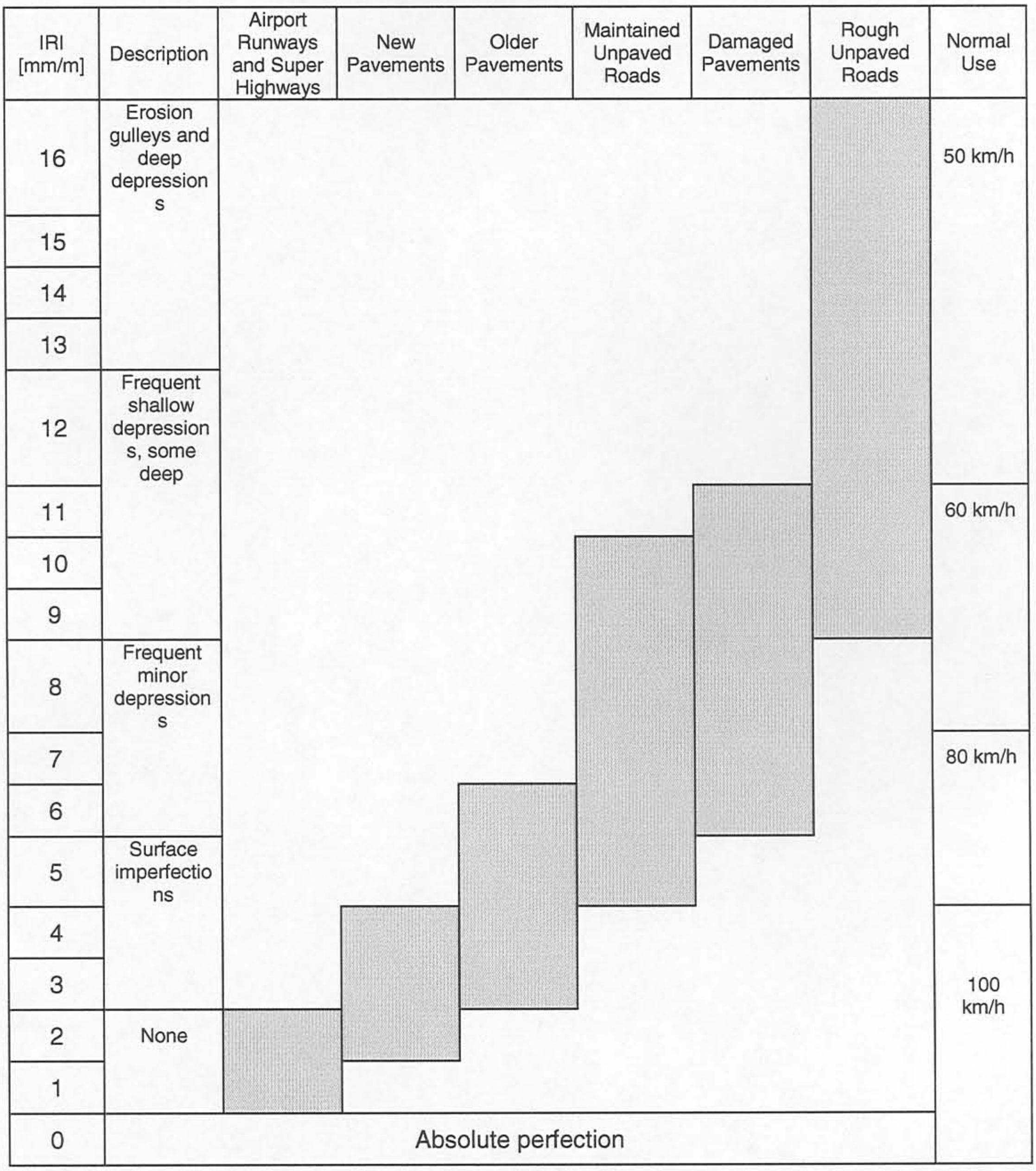
The Half-car Roughness Index (HRI) is based on the same equations and assumptions as the IRI, but the average of the two wheeltracks are used in the calculation of the statistic. HRI is always less than or equal to the IRI calculation. IRI indicate vehicle response at the tyres, while HRI indicate vehicle response at the centre of the vehicle (Kannemeyer, 1997).

The IRI of two pavements may be similar although they have different roughnesses. This is possible if one pavement has more pronounced longer wavelengths and the other more pronounced shorter wavelengths, and both these bands fall into the IRI wavelength band (Mann et al, 1997).

IRI is particularly sensitive to shorter wavelengths associated with axle resonance, and longer wavelengths linked with body bounce. These wavelengths cause dynamic load variations that reduce the road holding ability of tyres and contribute to road damage caused by commercial vehicles (Mann et al, 1997). The IRI scale ignores wavelengths outside the 1.3 m to 30 m wavelength band since these do not contribute to the roughness experienced by road-using vehicles at speeds near 80 km/h (approximately 17 to 1,35 Hz) (Sayers et al, 1986a). The IRI is most sensitive to slope sinusoids with wavelengths of 15,4 and 2,3 m, with a gain of 1,5 and 1,65 respectively. The gain decreases to 0,5 for wavelengths of 30,3 and 1,3 m (Mann et al, 1997).

The concept of the Ride Number (RN) was developed to be a relevant (good correlation with other roughness indices), portable (ability of different profiling systems to obtain comparable values for similar profiles) and simple algorithm (Sayers and Karamihas, 1996).

**Table 2.1: The IRI roughness scale (Sayers et al, 1986b).**




The IRI is not related to all vehicle response variables. It is most appropriate when a roughness measure is desired that relates to overall vehicle operating cost, overall riding quality and overall surface condition (Sayers and Karamihas, 1996). It is intended to reflect the pavement roughness attributes that affect the ride quality of passenger vehicles, and was not intended to describe the pavement roughness characteristics affecting heavy trucks, as is needed in this study. IRI does not show sensitivity to excitation frequencies as observed under heavy vehicle traffic (IRI sensitivity at 1,5 and 11 Hz, while heavy vehicle sensitivity at 3,5 and 12 Hz) (Papagiannakis and Gujarathi, 1995). Because of these different wavelengths affecting different vehicles, IRI is a poorer measure of ride quality for truck drivers than for car occupants. Trucks may be more sensitive to longer wavelengths inducing pitch and roll response modes (Mann et al, 1997; McLean and Ramsay, 1996).

Papagiannakis and Gujarathi (1995) described a pavement roughness statistic called Truck Response to Roughness Index (TRRI). The TRRI was developed to allow the roughness experienced by heavy vehicles to be expressed, allowing for the difference in dominant frequencies experienced by trucks and cars. TRRI is a frequency-domain calculation of the Root-Mean-Square acceleration of the frame of a reference truck. In Table 2.2 the IRI and TRRI values for two typical pavements are shown. It indicates that the difference between the two indices is not a constant, as the relative differences between the various parameters in the model cause the output to be dependent on the specific wavelengths present in the pavement profile analysed.

**Table 2.2: IRI and TRRI values for two typical pavements (Kemp, 1997).**

| Pavement description | IRI [mm/m] | TRRI [m/s$^2$/m] | Percentage difference [%] |
|---|---|---|---|
| Very good | 1,67 | 2,01 | 20,8 |
| Good | 2,68 | 3,31 | 23,8 |

A vehicle travelling on a pavement has two response modes. These are the body bounce at frequencies typically around 1 to 4 Hz, and the axle hop at frequencies around 10 to 18 Hz (Huhtala, 1995). Vehicle response to the pavement profile can be modelled in the frequency domain as a response function. The vehicle response characteristics amplify profile frequencies around the natural frequencies of the response modes, and attenuate profile frequencies well removed from those of the response modes. Mathematically, the vehicle frequency response function acts as a multiplier to the input road profile Power Spectral Density (PSD) to give the PSD of the vehicle response. For frequency characterisation of road profiles and frequency domain analysis of vehicle responses to the profile, the road profile can be characterised as a PSD. The PSD shows the variance in road profile elevation (or slope) as related to spatial frequency (McClean and Ramsay, 1996).

### 2.3.3 Pavement roughness input data

For this thesis, the actual pavement profile is of more importance than the roughness index, as the vehicle response models need actual pavement profiles as input. Various methods exist for measuring pavement roughness, depending on the objective and level of technology needed. The three main methods are panel ratings, response type road roughness measurements (RTRRMs), and profiling (Kannemeyer, 1997). Profiling is the best measurement method for use in vehicle-pavement interaction work, as this provides data to be used as input into vehicle-response models, and the spectral content of the pavement roughness can be calculated and used objectively in evaluation of the pavement.

In South Africa the High Speed Profilometer (HSP) is mainly used for pavement profile measurements. Rod-and-level equipment is generally more tedious than the HSP. The HSP measures the pavement profile at 80 km/h and a spatial frequency of 245 mm. These data are collected for two wheeltracks 1,6 m apart, with an accuracy of 0,26 mm. Currently there are approximately 7 770 km of historic data available, with more data being collected constantly (Kemp, 1997). Representative input data selected from these profiles are used in this thesis.

### 2.3.4 Conclusions

Pavement profile and roughness data are of vital importance to the analysis of vehicle-pavement interaction, as it provides the stimulus for dynamic pavement loading. Various pavement roughness indices are available, but the IRI is mostly accepted as a reference standard. The IRI is not as effective for analysing pavement roughness experienced by heavy vehicles, because the dominant frequencies experienced by cars and heavy vehicles differ. A different pavement roughness statistic (TRRI) is preferred to describe pavement roughness experienced by heavy vehicles.

Based on the information provided in this section, both the IRI and the TRRI are selected as indicators of pavement roughness. The IRI is used for comparison with existing databases, while the TRRI is used to indicate the roughness levels as experienced by heavy vehicles. The spectral content of pavement roughness is used where it can contribute to the understanding of specific vehicle responses. The input data for the vehicle response models are provided in terms of actual pavement profiles as measured with the HSP.

## 2.4 Vehicle Components

### 2.4.1 Introduction

This section focuses on vehicle components and models describing the response of vehicles and their components to external inputs. In the context of this thesis, the source of these external inputs is pavement roughness. Firstly, vehicle components and their critical parameters are identified. Next, vehicle models and response evaluation techniques are evaluated. Finally, some dynamic loading issues are discussed. Specific vehicle component characteristics typical for South African conditions (and used in this thesis) are summarised in Section 2.6.

### 2.4.2 Vehicle components

Vehicle components can be classified as sprung and unsprung masses. The sprung mass includes all parts supported by the suspension (including portions of the suspension members) while the unsprung mass includes all parts not carried by the suspension system, but supported directly by the tyre, and considered to move with it (Gillespie, 1992). The main vehicle components for discussion are the tyres, suspension, dimensions, configuration and load.

Tyres are important to pavements as they are the only contact point between the vehicle and the pavement and the point at which the actual distribution of contact stresses and load are conveyed to the pavement. Very little attention was traditionally given to tyres and their behaviour by pavement engineers.

The three basic functions of a tyre are to:

a. Support the vertical load while cushioning the vehicle against pavement shocks;
b. Develop longitudinal forces for acceleration and braking, and
c. Develop lateral forces for cornering (Gillespie, 1992).

Various combinations of tyre sizes and types are used on heavy vehicles. The main tyre types used are cross- and radial ply tyres. Truck tyres differ in width from 285 mm to 425 mm and diameter between 537 mm and 1 309 mm (SABS 1550, 1994). Normally tyres with widths of less than 315 mm are used as dual sets (except on steering axles) while those with widths of more than 315 mm can be used as single tyres (super singles).

Tyre characteristics depend on the construction of the tyre and the materials used. Tyre construction (i.e. radial or cross ply) affects the footprint and directional stability of the tyre, while different material compounds affect the way in which the tyre deforms and deteriorates (Gillespie, 1992). The main operational characteristic of tyres is the tyre inflation pressure. Specific tyre inflation pressures are recommended for use for specific tyres depending on the tyre construction, tyre load and tyre size (SABS 1550, 1994).

The contact stress distribution and magnitude between the tyre and the pavement surface is affected by parameters such as the tyre type, tyre inflation pressure, tyre load, tyre condition, speed and pavement condition (De Beer et al, 1997a). Different contact stress distributions are important for detailed tyre-pavement interaction studies. In this thesis only the maximum vertical contact stresses between the tyre and pavement are used as input parameter to the pavement structure.

Tyre models are used to describe the behaviour of tyres under operational conditions. The objectives of tyre models are to calculate the lateral forces on a tyre given certain operational characteristics (i.e. the vertical axle load, slip parameters, velocity and frictional pavement properties) (Preston, 1996). For the purposes of this thesis typical maximum vertical contact stresses ($CS_{max}$) as modelled using the relationships developed by De Beer et al (1997a) from Vertical Road-Surface Pressure Transducer Array (VRSPTA) data are used. The VRSPTA is used to measure tyre-pavement contact stresses in three dimensions. De Beer's approach appears the most suitable for further use in this thesis, because of its simplicity and the ability to provide vertical contact stress data without the need for complex input data. Equation 2-1 is used to calculate the maximum vertical contact stress peaks.

$$CS_{max} = 80{,}449 + 0{,}902TiP + 16{,}121\ Load$$

where :

$CS_{max}$ – Maximum Contact Stress [kPa]

$TiP$ – Tyre Inflation Pressure [kPa]

$Load$ – Wheel Load [kN]

$R^2 = 92\%$

Std error = 138 kPa

**Equation 2-1: Maximum vertical contact stress peaks**

There are mainly two types of suspension used on heavy vehicles in South Africa. These are air suspension and steel suspension (Van Niekerk, 1992; Campbell, 1997). The operational characteristics of these two types of suspensions differ, resulting in different effects on pavement loads. The type of springs and dampers, together with the dynamic load sharing between tandem axles are of most significance to the dynamic interactions of the truck with the pavement (Gillespie et al, 1993).

The main functions of a suspension system are to connect the wheels to the vehicle frame, position the tyres on the road, ensure road contact during travelling, transmit drive, braking and turning forces and spring and shock absorber loads and isolate the vehicle from road induced vibrations (Limpert, 1982).

The main characteristics of suspension systems affecting their behaviour are the composite vertical and roll stiffnesses, and the damping (Pretorius, 1990). The composite vertical stiffness consists of the combined stiffnesses of the individual springs. The composite roll stiffness is a function of the individual spring rates, lateral spring spacing and any auxiliary roll stiffness, and indicates the resistance of the suspension to vehicle roll. Suspension damping is a combination of the Coulomb damping (originating from interleaf friction) and viscous damping (originating from shock absorbers). Spring properties are characterised by the force-deflection characteristics of the suspension system (Gillespie et al, 1993).

In the Divine (1997) experiment it was concluded that indications are that a vehicle equipped with air suspension may cause less damage to a pavement than the same vehicle equipped with steel suspension. The results are, however, not conclusive and dependent on factors such as the type of pavement, distress, and vehicle. A sprung mass frequency of 1.5 Hz, viscous damping of 20 per cent and Coulomb damping of less than 50 per cent of the viscous damping are desirable for road friendly suspensions. These conditions may be difficult to achieve with steel suspensions.

The general dimensions of a vehicle include the overall length of the combination of vehicles, the width of the vehicle, distances between the axles and distances between the tyres on an axle. Restrictions are normally placed on the dimensions of vehicles through the road regulations (Fancher and Gillespie, 1997; Pretorius, 1990). The main effect of dimensions on vehicle-pavement interaction lies in the intervals between load applications from the various axles, and the percentage of load spread to each of the axles. These intervals are influenced by the speed at which the vehicle travels. It influences the load application frequency and therefore also the pavement response characteristics (Pretorius, 1990; Gillespie et al, 1993).

The configuration of a vehicle indicates the combination of components such as a truck/tractor and trailer. Although many combinations of components and vehicles are possible, industry makes use of a few optimum configurations. These are normally the configurations providing the most economical payload options within the framework of the allowed axle load regulations (Fancher and Gillespie, 1997). One of the systems used in South Africa for indicating vehicle configuration incorporates the number of axles on the vehicle and in each axle group (Nordengen et al, 1995). No indication is given of the tyre types and sizes, or physical dimensions of the vehicle.

The main effect of different vehicle and axle configurations on pavements lies in the effect that the specific combination of axles, and dimensions between axles, have on the response of the pavement.

The vehicle load is the primary vehicle component traditionally considered by pavement engineers. This is also one of the main vehicle parameters regulated through the maximum allowable axle loads. Studies have shown that fatigue of flexible pavement is highly dependent on individual axle loads while gross weight governs rutting for thicker (50 to 150 mm) asphalt surfacings (Gillespie et al, 1993). Load positioning on the vehicle and between axles and sides of the vehicle will affect the centre of gravity of the vehicle, which again is an important factor in the dynamic response of the vehicle.

### 2.4.3 Vehicle models

Todd and Kulakowski (1989) indicated analytical, experimental and computer simulations as three research methods that can be used for examining truck dynamics. Analytical methods are not practical due to the complexity of the mathematical models involved, while experimental methods are costly and limited by safety requirements. The most successful approach is to conduct a limited number of field tests to provide actual truck performance data to validate computer simulation programs. These computer simulation programs are used to extrapolate the experimental results over the range of test conditions where experimentation would be too dangerous or expensive.

Various vehicle models can be used for vehicle response to pavement roughness calculations. The simpler models consist of either quarter-vehicle or half-vehicle simplifications of the vehicle and can often be used with good effect (TFP; Macadam et al, 1980; Todd and Kulakowski, 1989). These models require less input data and computational effort, but their results may not be as accurate as the more complex models. The components are usually defined as part of the whole system, and only their characteristics can be changed and not their physical models.

The more complicated vehicle models allow for interchangeable component models and user-defined operating conditions, but are more expensive to develop, validate and use than the simpler models (DADS, 1997). It does, however, make good research sense to use these advanced models as reference models and use the most accurate simpler models to do production analyses. This approach is adopted in this thesis and thus a more advanced model (the reference model) and a simpler model (the production model) are identified.

The advanced vehicle model used is closely linked to the specific software used. In these advanced models the vehicle components can be modelled individually and these models combined to form a mathematical vehicle model. With the pavement profile as input to this mathematical model, the various response modes of the vehicle (i.e. roll, pitch and yaw) and the tyre loads at specific positions and time intervals, can be calculated (DADS, 1997). A typical vehicle model using the more advanced approach is shown in Figure 2.3.

Various programs exist for performing advanced vehicle-response calculations. Some of these are DADS (DADS, 1997), MADYMO (Jansen et al, 1996) and AUTOSIM (Sayers, 1991a; 1991b). All these programs have unique features and component models. As the focus of this thesis is on pavement response, a program was selected by evaluating the various programs on their published performance, locating a partner who owns one of the programs found suitable for the analysis, and co-operation with the partner on the detailed vehicle response analyses. Such a partner was found in Reumech Ermetek, who currently owns a copy of DADS, has extensive experience in the use of the program, and has validated the response of the program with real conditions and found it satisfactory (Nell, 1998).

DADS (DADS, 1997) is a computer simulation tool that is used to predict the behaviour of single or multibody mechanical systems. The mechanical model is analysed and position, velocity, acceleration and reaction forces are predicted for all parts of the model. Analyses can be performed in static, kinematic or dynamic modes. An advanced tyre module based on the Magic Tyre Formula is available in DADS. The standard tyre force elements calculate the normal, longitudinal and lateral tyre forces in the tyre-ground interface plane.

Most of the simpler vehicle response models use a simplification of the vehicle as a sprung and unsprung mass (or masses) connected via various springs and dashpots (TFP; Todd and Kulakowski, 1989). A typical half-vehicle simulation model is shown in Figure 2.4.

The Tyre Force Prediction Program (TFP) and PHASE4 (Macadam et al, 1980) programs are typical programs for the analysis of simple vehicle response simulations. PHASE4 is a time-domain mathematical simulation of a truck-tractor, a semi-trailer and up to two full trailers. TFP (TFP) is an analytical model of a truck or truck-tractor and semi-trailer combination used to predict the forces that occur between the tyres of the vehicle and the road. The program is similar to PHASE4, except that vehicle response is simulated at constant speed, and no turning or braking manoeuvres or roll effects can be simulated. It is only a two-dimensional vehicle model. Both programs have been used for many studies available in the literature on vehicle-pavement interaction (Van Niekerk, 1992).

Van Niekerk (1992) evaluated both TFP and PHASE4 for use in simulations of dynamic vehicle loading. Both were found to provide satisfactory results. TFP (which requires simpler input data) results were subsequently compared with measured dynamic load data and it gave a reasonable indication of the overall trend of the dynamic axle masses.

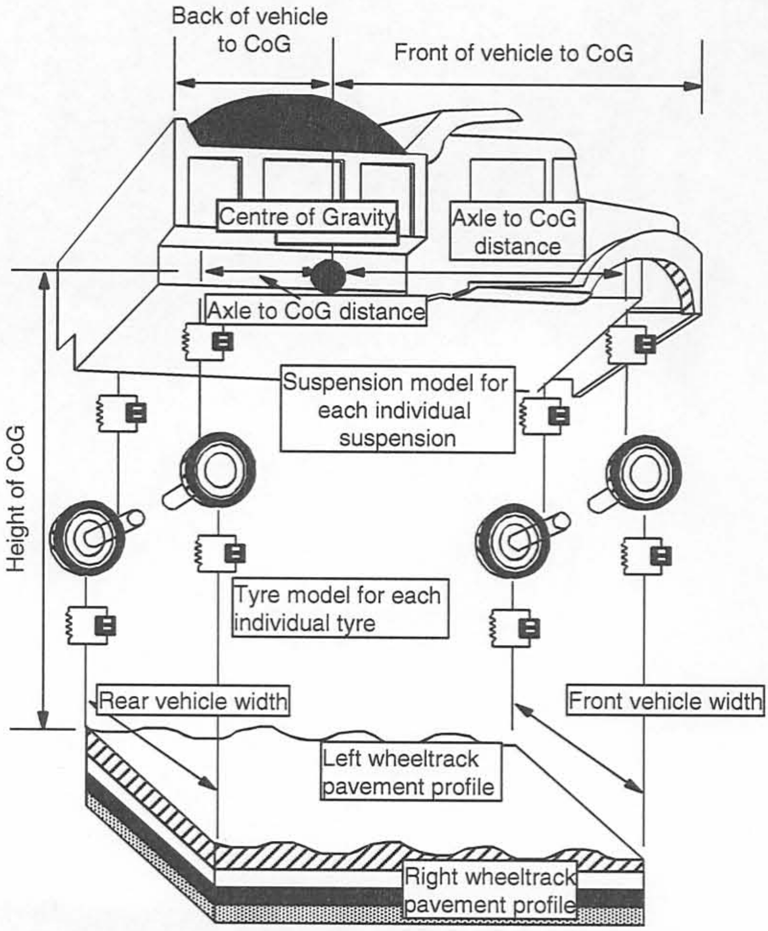



**Figure 2.3:** **Typical complex vehicle model indicating characteristics defined for all components (Sousa et al, 1988).**

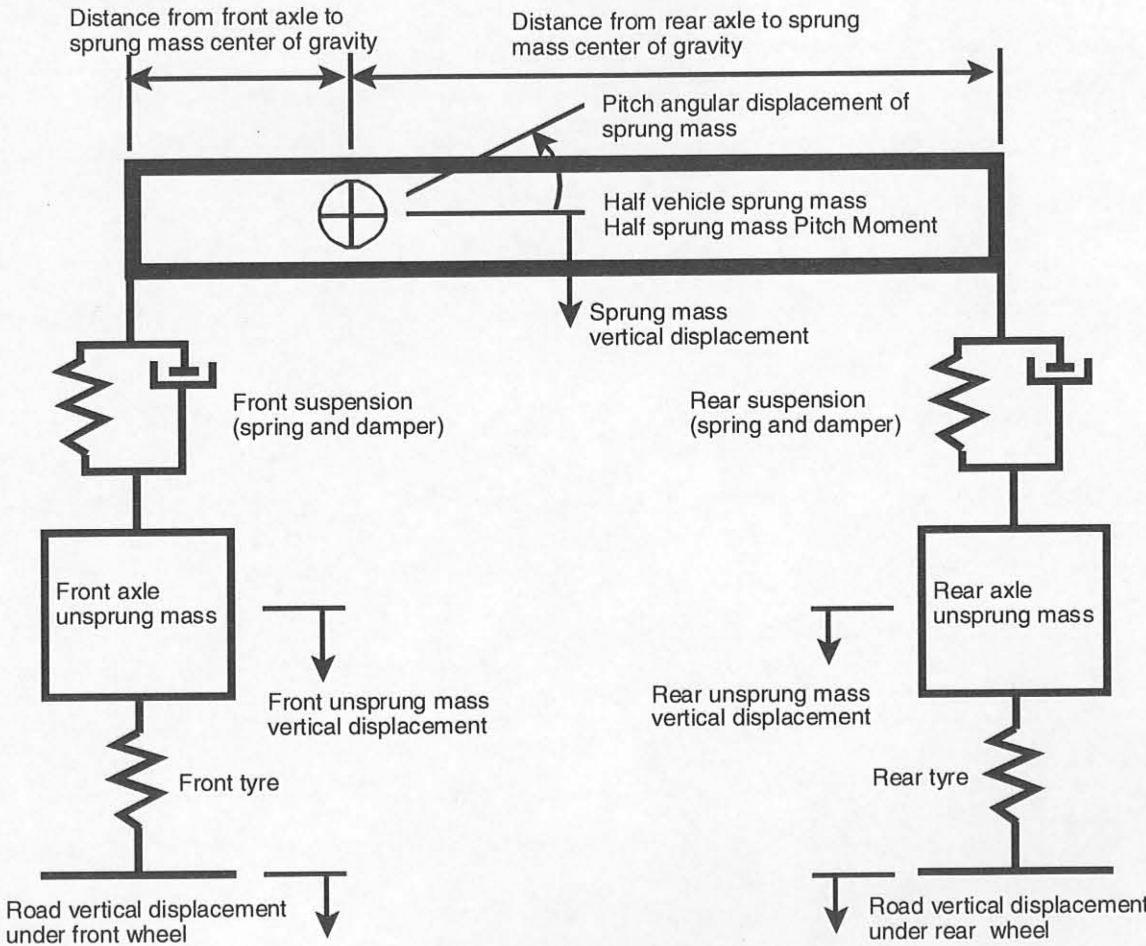



**Figure 2.4: Typical simple vehicle simulation model (Todd and Kulakowski, 1989).**

Kenis and Hammouda (1996) indicated through a sensitivity study of three truck dynamic simulation programs that vehicle roll plays an insignificant role in dynamic tyre force calculations, and that a pitch-bounce model is sufficient for most applications.

TFP was selected for use in this thesis as it provides all the information required for the simpler vehicle response program, and is computationally friendlier and requires less input data than PHASE4. DADS was selected as the reference vehicle response program.

### 2.4.4 Dynamic tyre loading

Pavement loading has been shown by various authors to be a dynamic (time-dependent) phenomenon (Divine, 1997). A pavement experiences a truck as a moving, time-varying set of contact stresses applied at the pavement surface. These stresses are determined by the static load carried by each tyre, the dynamic variation in load at each tyre (as affected by the suspension and tyre characteristics), the nature of the pressure distribution arising from the total load applied to the surface under the tyre, and in-plane forces applied to the surface in the form of shear stresses (Gillespie et al, 1993). The dynamic load component has been shown to be between 5 and 50 percent of the static load component, depending on factors such as the vehicle (and vehicle components) dynamic response, vehicle operating conditions and pavement roughness level.

The dynamic load effect of an axle is typically expressed as the Dynamic Load Coefficient (DLC) (Equation 2-2) or the Dynamic Stress Coefficient (DSC) (Equation 2-3). DLC indicates the Coefficient of Variance (CoV) of the applied variable load while DSC indicates the CoV of the stress measured at a specific position in the pavement structure (Sweatman, 1983; Kenis et al, 1997). Typical DLC values range between 0,01 and 0,40, while typical DSC values range between 0,03 and 0,22 (Kenis et al, 1997; Sweatman, 1983; Divine, 1997).

$$DLC = \frac{\text{std Load}}{\text{avg Load}}$$

**Equation 2-2: Dynamic Load Coefficient.**

$$DSC = \frac{\text{std } \varepsilon}{\text{avg } \varepsilon}$$

where

std Load – standard deviation of wheel force distributi on
std ε – standard deviation of strain history
avg Load – average wheel force
avg ε – average of strain history

**Equation 2-3: Dynamic Stress Coefficient.**

Dynamic load profiles for all heavy vehicles are characterised by two distinct frequencies. Body bounce (1.5 to 4 Hz) generally dominates the dynamic loading, and is mainly caused by the response of the sprung mass of the vehicle to the pavement roughness. Axle hop (8 to 15 Hz) becomes more significant at higher vehicle speeds and higher pavement roughnesses, and is mainly caused by the reaction of the unsprung mass to pavement roughnesses. The main cause for the dominating effect of the body bounce component may lie in the load ratio of approximately 10:1 between the sprung mass and the unsprung mass (Gillespie, 1992).

Two types of variability exist in pavement loading. These are the longitudinal loading variability and the cyclic variable load on a discrete point of the pavement (Divine, 1997; O'Reilly and Brown, 1991). While body bounce and axle hop frequencies characterise the longitudinal load profile, the cyclic variable load frequency depends on the speed and tyre patch length. For practical reasons the shortest discrete loading point can be defined as one tyre patch length. This causes typical load frequencies of between 18 Hz and 92 Hz (for a 300 mm tyre patch length and speeds of 20 km/h and 100 km/h).

Schematically the difference between the dynamic loading as experienced by the vehicle (longitudinal load variable) and the cyclic variable loading as experienced by the pavement on a discrete location can be described as in Figure 2.5. The y-axis indicates the actual load value, and on the x-axis time as experienced by the vehicle and the pavement is shown. The load history as experienced by the vehicle is continuous at body bounce and axle hop frequencies. The transient pavement load history is cyclic with frequency dependent on the vehicle speed and distances between axles and vehicles. Chatti et al (1995) indicated that use of a stationary pulse to model dynamic loading on a discrete point is insufficient to model the dynamic load effect. Mamlouk (1987) indicated that axle loads cause a series of half sine wave stresses in a pavement.

It is the opinion of the author that the dynamic loading effect could be analysed as two distinct but related processes. The primary load mode is the cyclic (transient) load on the discrete pavement position caused by the specific load generated at the specific point. The secondary load mode is the loads caused on the remainder of the pavement by the vehicle as it travels before and after the discrete point under investigation, and which is transferred to the discrete point under investigation via waves in the pavement. The relative importance of these two load modes depends on factors such as the variability of the load, the response characteristics of the pavement (i.e. damping), and the specific materials encountered. This opinion receives further attention later in this thesis. One of the alternatives proposed to simplify the analysis procedure is to neglect the effect of the secondary load mode.

Le Blanc and Woodrooffe (1995) defined spatial repeatability as the tendency for tyre load signals to have the peaks and lows to generally recur at the same locations on the pavement. This may either be caused by similar or different vehicles. Correlation coefficients of between 0.2 and 0.5 indicate moderate correlation, with higher coefficients indicating good spatial repeatability. Spatial repeatability specifically affects the cyclic damage variation on discrete locations, as it causes accumulation of increased damage on specific points.

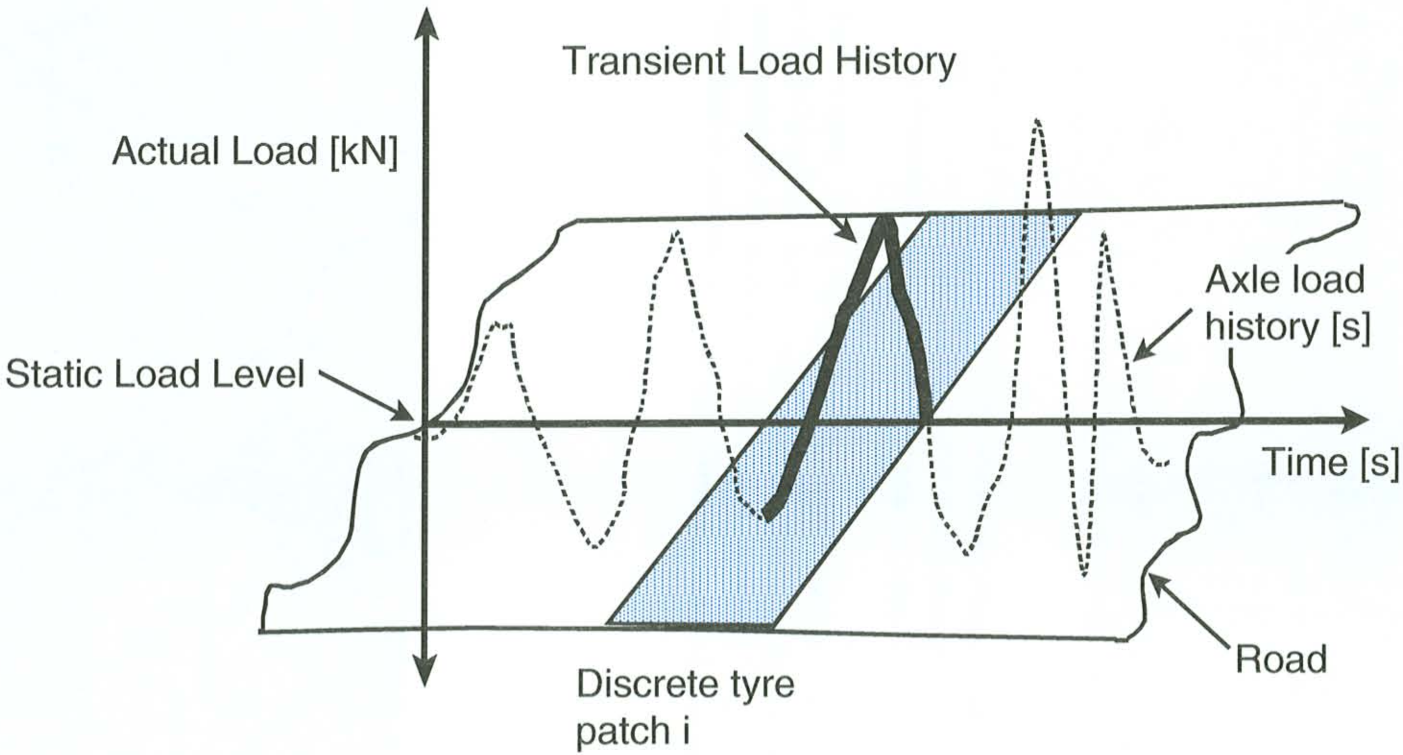



**Figure 2.5:** **Longitudinal axle load history and transient load history.**

### 2.4.5 Conclusions

The vehicle components of importance to vehicle-pavement interaction, specifically for linear constant-speed movement, were investigated in this section. The objective was to provide the necessary understanding, tools and data to enable calculation of vertical dynamic pavement-roughness-induced vehicle loads on pavements.

The main vehicle components identified are the tyres, suspension, dimensions, configuration and load of the vehicle. Standard parameters of each of these components were identified.

It was shown that vehicle models and evaluation techniques could be either detailed and complicated, or simple. DADS was selected as the reference vehicle model and evaluation technique for bench marking the simpler TFP program, selected for production analyses.

Dynamic pavement loading consists of two phases. The primary phase is the load effect on a discrete position of the pavement while the second phase is the load effect caused by the time-history of the vehicle away from the discrete point under investigation.

## 2.5 Pavement Components

### 2.5.1 Introduction

Pavement response constitutes the main part of this thesis. This is specifically with regard to the transient response of a pavement structure to a dynamic load input. The static response of a pavement structure and the static load input to a pavement are regarded as special cases of the general dynamic load and transient response model of the pavement structure.

Structural models for analysis of pavement response to traffic loads range in complexity from simple empirical models to sophisticated models that attempt to realistically describe the behaviour of the materials. The selection of the type of model depends on the ability of the designer to quantify the required material inputs and interpret the results of the models (Haas et al, 1994).

Desai and Gallagher (1984) stated that a valid solution to a problem in soil mechanics (of which pavement engineering can be seen as a specific branch) must satisfy the following three basic equations:

a. Equations of equilibrium (of stresses and forces) (Timoshenko and Goodier, 1951);
b. Equations of compatibility (of strains and displacements), and
c. Constitutive equations (or the relations between stresses and strains in the material).

This section starts with a discussion of the material models (constitutive equations) for the typical South African pavement construction materials used in this thesis, followed by models and techniques for the description of transient pavement structure response. Finally, the dynamic effects of vehicles on transient pavement structure response are discussed.

### 2.5.2 Materials

The main materials used in flexible pavement structures in South Africa are discussed in TRH14 (1985). Each of these materials reacts uniquely to external loads and environmental changes. In this section the typical constitutive equations and properties for seven of the materials identified for further use in this thesis in Section 2.6 are discussed. The eighth material, a double seal, does not posses any structural properties and is thus ignored for the analyses. The seven materials can be grouped into four groups. These are:

a. Thin (< 50 mm) asphalt (AC);
b. Granular materials (G1, G4, G6);
c. Cemented materials(C3, C4), and
d. Soils (SG1).

The load-response characteristics of materials for application in mechanistic models define their constitutive equations. Mechanistic material evaluation requires a compromise between complexity of testing and analytical procedures and the ability to perform tests cost-effectively under simulated field-conditions (Haas et al, 1994). Various generic material response models exist. Yoder and Witczak (1975) have shown three properties of materials to be sufficient to describe material behaviour needed for pavement analysis:

a. The stress-strain function (linear or non-linear);
b. The time-dependency of strain for given constant stress (viscosity), and
c. The strain recovery after stress removal (elasticity or plasticity).

These three properties are schematically shown in Figure 2.6. Various combinations of these three properties are possible to describe material response.

Asphalt materials typically show visco-elastic or elasto-visco-plastic behaviour (Motrescu and Visser, 1995). The mechanical models of visco-elastic behaviour include the common Maxwell, Kelvin, Burgers and Generalised models. Jooste (1997) effectively used the Burgers model for the analysis of visco-elastic behaviour of asphalt materials, to model the strain behaviour of asphalt under both laboratory and pavement loading conditions. The typical Burgers model is shown in Equation 2-4.

$$\varepsilon(t)=\frac{\sigma}{E_1}+\frac{\sigma t}{\eta_1}+\frac{\sigma}{E_2}\left[1-e^{(-tE2/\eta 2)}\right]$$

where

$\varepsilon$ – strain (time dependent)
$\sigma$ – applied stress [MPa]
E – elastic modulus of spring [MPa]
$\eta$ – coulomb damping factor [MPa.s]
t – time [s]

**Equation 2-4: Burgers model for visco-elastic behaviour in asphalt (Jooste, 1997).**

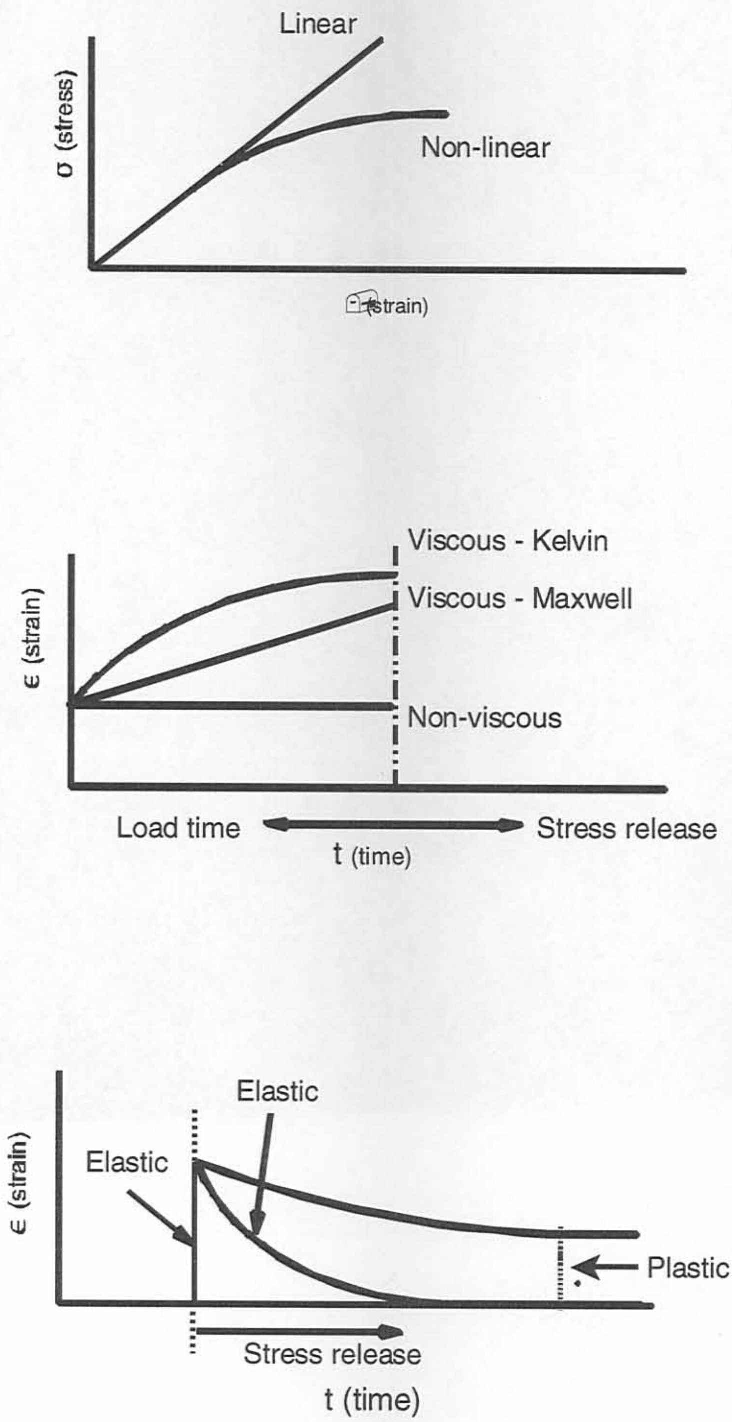



**Figure 2.6:** **Material characteristics (after Yoder and Witczak, 1974).**

$$E^* = \frac{\sigma_0}{\varepsilon_0}\left[\cos\ \delta + i \sin\ \delta\right] = E_1 + iE_2$$

where

$E^*$ – complex modulus [MPa]

$\sigma$ – applied stress [MPa]

$\varepsilon$ – strain

$\delta$ – phase angle

$E_1$ – in – phase component of complex modulus [MPa]

$E_2$ – out – of – phase component of complex modulus [MPa]

**Equation 2-5: Complex modulus (Jooste, 1997).**

The material parameters used in the Burgers model are the spring stiffnesses and the coulomb damping factors. These parameters can be established using a dynamic or complex modulus experiment where an oscillatory stress is applied to the specimen. The strain response for an oscillatory stress of $\sigma_0$ sin$\omega$t will be $\varepsilon_0$ sin($\omega$t + $\delta$). $\delta$ is the phase angle by which the strain response lags the stress response. The complex modulus (E*) can be calculated using Equation 2-5 and the mechanical loss using Equation 2-6. Equation 2-7 provides means for calculating the spring stiffnesses and coulomb damping factors for use in the Burgers model.

$$\tan\ \delta = \frac{E_2}{E_1}$$

**Equation 2-6: Mechanical loss (Jooste, 1997).**

$$E^* = \frac{\left(\left[p\ q_1\omega^2 - q_2\omega^2\left(1 - p_2\omega^2\right)\right] + i\left[p_1 q_2\omega^2 + q_1\left(1 - p_2\omega^2\right)\right]\right)}{\left(p_1^2\omega^2 + \left(1 - p_2\omega^2\right)^2\right)}$$

where

$$p_1 = \frac{\eta_1}{E_1} + \frac{\eta_1}{E_2} + \frac{\eta_2}{E_2}$$

$$p_2 = \frac{(\eta_1\eta_2)}{(E_1E_2)}$$

$$q_1 = \eta_1$$

$$q_2 = \frac{(\eta_1\eta_2)}{E_2}$$

E, $\eta$ from Equation 2 – 4

**Equation 2-7: Spring stiffness and coulomb damping factors for Burgers model (Jooste, 1997).**

Granular materials typically behave in a non-linear elastic mode (Sweere, 1990). Sweere (1990) identified a modified Boyce's G-K model for predicting the resilient stress-strain behaviour of granular materials, where the stresses and strains are separated into volumetric and shear components. The model is shown in Equations 2.8 and 2.9. Motrescu and Visser (1995) identified non-linear elastic models for describing the permanent deformation behaviour of granular materials. They identified the approach followed by Brown and Pappin to provide an adequate model of granular material plastic deformation (Equation 2.10). This is the only model allowing the influence of residual stresses to be modelled. It was shown to produce results that were in good agreement with all aspects of granular material behaviour, provided that a residual stress component (7 to 14 kPa) induced by compaction is postulated.

$$\varepsilon_v = \frac{1}{K_1 \, p^n \left(1 - \beta q^2 / p^2\right)}$$

**Equation 2-8: Volumetric strain (Sweere, 1990).**

$$\varepsilon_s = \frac{1}{3\, G_1 \, p^m \;\; q/p}$$

where

$K_1$ – bulk modulus [MPa]

$G_1$ – shear modulus [MPa]

n, m – stress – dependency factors (1 – linear elastic, 0 – minimum)

$\beta$ – damping coefficien t

$$p - \frac{1}{3(\sigma_1 + \sigma_2 + \sigma_3)} \text{ [MPa]}$$

$$q - \frac{1}{\sqrt{2\left\{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2\right\}^{0.5}}} \text{ [MPa]}$$

**Equation 2-9: Shear strain (Sweere, 1990).**

$$M_r = k_1 \, \theta^{k_2} \, \sigma_d \, k_3$$

where

$M_r$ – resilient modulus [MPa]

k1, k2, k3 – material constants

$\theta$ – sum of principal stresses [MPa]

$\sigma_d$ – deviatoric stress [MPa]

**Equation 2-10: Resilient modulus (Motrescu and Visser, 1995).**

The material parameters needed for evaluation of the granular material constitutive equations have traditionally been measured using tri-axial tests. The five material parameters for Equations 2-8 and 2-9 can be determined using a regression analysis from resiliency tests, while the principal stresses can be calculated from the input and output data of the tri-axial tests.

$$E_R = k_1 \, \theta^{k_2}$$

where

$E_R$ – resilient modulus [MPa]

$k_1, k_2$ – material parameters

$\theta$ – sum of the principle stresses [MPa]

**Equation 2-11: Typical k-Θ model (Uzan, 1985).**

Lightly cemented materials typically behave in a linear elastic mode, especially when used as subbase layers and during the initial phase of their lives. Slight non-linear (stress-stiffening) behaviour may develop towards the effective granular phase (De Beer, 1998). For the purposes of this thesis a typical k-θ model (Equation 2-11) is used for describing resilient behaviour (Uzan, 1985). Tri-axial test can again be used to determine the various material parameters needed. The material parameters for the two phases have to be evaluated separately.

Subgrade soils normally behave in a non-linear elastic mode (Brown, 1996). Motrescu and Visser (1995) identified several non-linear elastic models. They indicated that both the Second stress invariant model and the Cornell constitutive model can be used for subgrade material modelling. As typical values for the material constants could not be obtained from the literature, the Brown and Pappin model (Equation 2-10) is again used in this thesis. The parameters needed for evaluation of Equation 2-10 for subgrade materials are shown in Table 2.3 with typical ranges.

A summary of the material properties needed to evaluate the various equations, together with the typical method for evaluation and published values of these parameters are shown in Table 2.3. Current methods for evaluating these parameters cost-effectively are evaluated and discussed in Chapter 6 of this thesis.

The constitutive equations selected for the various materials indicate that only the asphalt is sensitive to load frequency (time-dependent behaviour). All of the other materials are stress level dependent.

Mamlouk (1987) emphasised that the accuracy of any transient pavement analysis technique ultimately depends on the accuracy by which the materials are characterised. Desai and Gallagher (1984) further indicated that dynamic analysis is intimately related to the capability of measuring the necessary soil properties. These two vital aspects are further addressed in later chapters of this thesis.

**Table 2.3: Material properties needed for each material type in analysis.**

| Material | Model (Equation) | Parameter | Method | Typical published values |
|---|---|---|---|---|
| Asphalt (AC) | Burgers (Equation 2.5) | σ [kPa] | Dynamic modulus | 690 kPa[a] |
| | | $\eta_{1,2}$ [MPa.s] | | 5000, 1000[a] |
| | | $E_{1,2}$ [MPa] | | 2000, 1500[a] |
| Granular (G1, G2, G6) | Modified G-K (Equations 2.8 and 2.9) | $k_1$ [MPa] | Tri-axial test | 57 MPa |
| | | $G_1$ [MPa] | | 404 MPa |
| | | β | | 0.17[b] |
| | | n | | 1.00[b] |
| | | m | | 0.33[b] |
| | Brown-Pappin (Equation 2.10) | $\sigma_3$ [kPa] | Tri-axial test | 0,1 to 4,4[c] |
| | | $\sigma_d$ [kPa] | | 0,7 to 9,3[c] |
| | | $k_1$ | | 128 to 1282[c] |
| | | $k_2$ | | 0,32 to 1,49[c] |
| | | $k_3$ | | -0,08 to -1,53[c] |
| Cemented (C3, C4) | Uzan (Equation 2.11) | $\Sigma_1$, $\sigma_2$, $\sigma_3$ | Tri-axial test | 200, 50, 50[d] |
| | | $k_1$ | | E = 200 to 600[d] |
| | | $k_2$ | | 0[d] |
| Subgrade (SG1) | Brown-Pappin (Equation 2.10) | $\sigma_3$ [kPa] | Tri-axial test | 0,0 to 0,9 |
| | | $\sigma_d$ [kPa] | | 0,3 to 1,5 |
| | | $k_1$ | | 46 to 340[c] |
| | | $k_2$ | | 0,09 to 0,51[c] |
| | | $k_3$ | | -0,32 to -1,38[c] |

a. Jooste (1997). Typical thick (120 mm) asphalt on crushed stone base and gravel subbase.
b. Sweere (1990). Crushed concrete.
c. Rohde (1990). Various granular base courses and sandy subgrade materials.
d. De Beer (1998). Various base and subbase applications of lightly cemented materials.

### 2.5.3 Transient Response Analysis of Pavement Structures

In this thesis pavement response analysis techniques are used as toolboxes for obtaining the transient response of pavements under varying conditions. The mathematical background of these techniques is only covered as background information. The limitations of the identified techniques are thus not specifically investigated, although such limitations that may influence the outcome of analyses are discussed.

The process of mechanistic pavement analysis effectively consists of four steps:

a. Definition of the pavement problem in mathematical terms (conversion from physical reality to mathematical model);
b. Calculation of the stresses and strains in the pavement structure using an analysis technique and material constitutive laws (conversion of stresses to strains and displacements in pavement);
c. Calculation of the expected lives of the various components using transfer functions (conversion of stresses, strains and displacements to damage parameters), and
d. Calculation of the economic life of the pavement (conversion of damage parameters to economic terms) (Haas et al, 1994).

The focus of this thesis is on steps a and b of this process.

Three different approaches can be followed in pavement analysis. These are the static, quasi-transient and transient approaches (Chatti, 1992). The static approach has been used traditionally (and continues to be used). Either elastic layer theory or finite element approaches are used for analysing flexible pavements. No inertia or damping effects are quantified in the pavement.

The quasi-transient approach is based on the concept of positioning the load at subsequent positions along the pavement for each new time step, and assuming the load to be static at each position. Using static analysis the loading position causing the most severe effect is determined via influence lines which provide the variation in the static response at a fixed point due to a unit load traversing the pavement. The quasi-static approach has been justified by the fact that traffic velocities are less than 10 per cent of the critical velocity (propagation velocity of a transverse displacement wave through the pavement) of typical pavements. Sousa et al (1988) indicated that inclusion of dynamic effects might be validated if the stress field caused by a group of tyres are considered, but that a quasi-transient approach should be valid under certain combinations of load types, vehicle velocities, pavement stiffness and pavement roughness.

The transient approach is based on analysis of a layered structure of solids. Transient models vary in complexity depending on the structure and load characteristics. Zafir et al (1994) stated that there are two important factors that should be considered in any transient pavement analysis. These are the inertia associated with the moving load and the dependency of the material properties on the applied stress and the loading frequency. The fluctuation nature of the applied load should also be considered if actual conditions are simulated. The loading frequency effect is considered through the use of appropriate material constitutive equations.

Two basic approaches for incorporating transient pavement structure response to tyre loading were identified in the literature. Either of these two approaches can be used to develop the differential equations needed for solution of the problem. Each approach has some unique features and limitations, which are discussed briefly in this section.

The first approach is that using Newton's second law of motion (Lourens, 1992; Cook, 1995; Jooste and Lourens, 1998). The transient response of the pavement structure is described using an Ordinary Differential Equation (ODE) with parameters characterising the stiffness, damping and mass of the pavement materials (Equation 2-12). These parameters are used to solve for the displacements in the pavement structure due to a specified load function. The displacements are used together with the constitutive equations for the various pavement materials to calculate the stresses and strains in the pavement structure.

$$F(t) = K\,u(t) + C\,u'(t) + M\,u''(t)$$

where

$F(t)$ – loading function

$K$ – stiffness matrix

$C$ – damping matrix

$M$ – mass matrix

$u(t)$ – displacemen t matrix

$u'(t)$ – velocity matrix

$u''(t)$ – accelerati on matrix

**Equation 2-12: Ordinary Differential Equation for transient response of pavement structure.**

The essential material parameters are the stiffnesses, mass properties and damping coefficients of the layer materials. Some authors (Lourens, 1992) use the Young's modulus for quantification of the stiffness matrix. It may be argued that the resilient modulus (Monismith, 1992) or effective elastic modulus (De Beer et al, 1997b) may also be appropriate parameters. The resilient modulus refers to the relationship between the applied stress and recoverable strain measured in a repeated load test (Monismith, 1992). The effective elastic modulus refers to the elastic modulus obtained from backcalculation of in situ Multidepth Deflectometer (MDD) measured elastic deflections (De Beer et al, 1997b).

The mass properties typically depend on the unit mass density of the material in the layer. The damping coefficients can be determined using typically tests such as the resonant column (Jordaan, 1996; Allen and Deen, 1980). Jordaan (1996) indicated that the test is expensive and fairly sophisticated, and has seldom been used in South Africa. Hardin and Drnevich (1972) have also shown that load-deflection tests yielding a shear-stress shear-strain curve can be used to calculate the damping coefficient from the area of the load-unload hysteresis curve. Typical values for the stiffness, mass and damping parameters for typical South African pavement materials are shown in Table 2.4.

The second approach (Mamlouk, 1987; Sebaaly et al, 1985; Davies and Mamlouk, 1985; Roesset and Stokoe, 1990) is based on the governing equations for elastodynamics (wave equations). These equations are used to develop the Helmholtz Partial Differential Equation (PDE) (the governing equation for steady-state (harmonic) elastodynamics) (Equation 2-13).

**Table 2.4: Typical material properties for use in Equation 2.10 (Lourens, 1992).**

| Material type | Young's modulus [MPa] | Poisson's ratio | Mass [kg/m$^3$] | Damping ratio D [%] |
|---|---|---|---|---|
| Asphalt | 1 500 | 0,4 | 2 400 | 5 |
| Crushed stone | 350 | 0,35 | 2 300 | 5 |
| Stabilised subbase | 2 000 | 0,2 | 2 000 | 5 |
| Selected layer | 200 | 0,4 | 2 000 | 5 |
| Natural soil | 100 | 0,4 | 2 000 | 5 |

$$G\Delta\Delta + (\lambda + G)\nabla\nabla \bullet u = -\rho\omega^2 u$$

where

$\lambda, G$ – Lame's constants

$\rho$ – mass density

$\omega$ – circular frequency of excitation

$u_i$ – $i^{th}$ cartesian component of displacement vector

$\Delta$ – Laplacian operator

$\nabla$ – gradient

**Equation 2-13: Helmholtz partial differential equation (Mamlouk, 1987; Sebaaly et al, 1985; Davies and Mamlouk, 1985; Roesset and Stokoe, 1990).**

Five material parameters are used. These are Young's modulus, Poisson ratio, material damping, mass density and layer thickness (Sebaaly et al, 1985). Material linearity and isotropy and no-slip between layers are assumed. Inertia of a pavement is an explicit part of the elastodynamic analysis and thus no further allowance is necessary. Material damping is accommodated by using the complex modulus ($E^*$) (Equation 2-14). The various material parameters can again be evaluated using the tri-axial test.

$$E^* = E(1 + 2i\beta)$$

where

$E^*$ – complex modulus [MPa]

$E$ – Young's modulus [MPa]

$\beta$ – damping ratio [%]

**Equation 2-14: Complex modulus (Sebaaly et al, 1985).**

The differential equations developed using the two approaches discussed, can be solved using typically one of two approaches. These are either an analytical technique or a numerical technique (Haas et al, 1994).

Various numerical techniques (i.e. finite elements, finite differences, boundary elements) can be applied to the analysis of pavement structures. The numerical technique used in this thesis for the analysis of transient pavement response is the finite element method. The use of finite element methods provide a good estimate of stresses and strains in pavement structures, but slight differences may occur in calculated deflections due to differences in boundary conditions. Finite element methods for the analysis of pavements permit modelling of more complex material characteristics and pavement geometries than is possible with the analytical solution methods (Haas et al, 1994).

The ordinary differential equation (ODE) developed using Newton's second law of motion is used (Equation 2-12). The required forces to balance the pavement's response to input displacements are calculated, followed by calculation of the governing stresses in the pavement body. The strains and deflections throughout the pavement body are calculated using material models (constitutive laws). This procedure is iterative in nature for non-linear and stress-dependant materials. The complexity of the numerical approach used varies. It ranges from simple axi-symmetrical models to full three-dimensional pavement structures. The more advanced models require more computational abilities but also provide more accurate outputs. The assumptions used in the finite element analysis vary depending on the specific technique used. These may include simplifications such as axi-symmetrical load conditions, uniform load distributions and linear elastic material responses.

Hardy and Cebon (1992a; 1992b; 1993; 1994) described the convolution theory (Equation 2-15; general form) as a technique for the analysis of transient pavement response. A solution of the response of the pavement structure to a unit impulse load, as well as the load function, is needed. The integral over the product of these two functions provide the transient response of the pavement structure at a specific time and position. Assumptions with regard to the stiffness of the materials involved have to be made. The assumptions inherent to the specific solution of the response of the pavement structure to a unit impulse load, are transferred to the solution. Hardy and Cebon (1992a; 1992b; 1993; 1994) indicated that the only way to include speed and frequency effects rigorously in road-response calculations is to use a transient road-response model. Their model describes the response of an isotropic dynamically linear pavement to fluctuating loads moving over its surface (Equation 2-16; time domain). They simplified the convolution theory to yield a quasi-transient calculation because they have shown that the effects of loading frequency on pavement strains are relatively minor compared with the effect of speed (for the pavement and loads analysed). In this influence function approach, the effect of load frequency is neglected, while the correct allowance is maintained for vehicle speed and dynamic tyre load magnitudes.

$$y(t) = \int_{-\infty}^{\infty} h(t - \tau)\, f(\tau)\, d\tau$$

where

$y(t)$ – response at time $t$

$f(\tau)$ – input force at time $\tau$

$h(t)$ – response at time $t$ to a unit impulse at time $t = 0$

**Equation 2-15: Convolution theory in general form (Hardy and Cebon (1992a; 1992b; 1993; 1994)).**

$$y(x, t) = \sum_{m=1}^{N_f} \int_{-\infty}^{t} h\left[x - (d_m + v\tau),\, t - \tau\right] f_m(\tau)\, d\tau$$

where

$y(x, t)$ – response at position $x$ in the wheelpath at time $t$ [m]

$N_f$ – number of inputs (or tyres)

$t$ – time [s]

$x$ – position [m]

$d_m$ – longitudin al position of tyre (force) $m$ at time zero [m]

$v$ – vehicle speed [m/s]

$\tau$ – time [s]

$f_m$ – force [kN]

$h(x, t)$ – response at position $x$ and time $t$ to a unit impulse at the origin at time zero [m]

**Equation 2-16: Convolution theory in time domain (Hardy and Cebon (1992a; 1992b; 1993; 1994)).**

A typical response function (h(t)) used by Hardy and Cebon (1993) is that for an Euler beam supported by a damped elastic foundation with mass, elastic and damping parameters chosen to simulate the response measured during the AASHO road tests. The response of this type of system to moving harmonic loads, where the loads are simulated by a linear quarter-car model running over a 10 mm step to excite the vehicle was evaluated and found to agree with expected results.

Markow et al (1988) described a quasi-transient method for incorporating dynamic load effects on pavement response calculations. They argued that the pavement response to a load P at a distance x is assumed to be equivalent to the response to a lesser load P' placed directly at the position of interest. The response at a specific position ($x_0$) due to an arbitrary load F(x) is therefore the product of the influence function (I) and the force function (Equation 2-17). This technique appears very similar to the convolution approach.

$$R(x_0) = I(x_0 - x)F(x)$$

**Equation 2-17: Response of pavement due to force F using quasi-transient method (Markow, 1988).**

It forms part of this thesis to evaluate the different techniques described for their appropriateness in analysing pavement structures dynamically. Although the focus is more on materials and pavement parameters, the use of different mathematical tools and/or the development of new ones where necessary and appropriate are not excluded from this thesis.

### 2.5.4 Parameter effects

Some of the more well-researched and proven effects of the various vehicle, pavement and material parameters are presented in this section. The focus is specifically on those issues where a static and dynamic load application and pavement response will differ. The objective is to highlight possible interaction issues and identify the parameters of importance. Only the results of such research are highlighted, and detail information with regard to the test conditions and background can be found in the references indicated. Some of the findings may be more focussed on the behaviour of the thicker asphaltic layers.

Very little work has been done on the response of typical South African pavement structures with thinner surfacing layers and thicker granular and cemented layers to dynamic loads and transient response. De Beer (1991) did some work on the elastic deflections caused by heavy vehicles when running at speed, and Lourens (1992) performed some measurements and analysis on transient pavement response to dynamic load conditions. Recently, Jooste and Lourens (1998) investigated the effect of dynamic analysis versus static analysis on asphalt pavements.

Lourens (1992) and Chatti et al (1995) found that the vertical stresses in the asphalt decrease with increased load speed. The influence sphere of the load moved shallower, and the total surface deflection increased. The Radius of Curvature (RoC) also increased. The speed effect is more pronounced on surface deflection than on tensile strain in the asphalt layer. The increased dynamic load levels, strains and fatigue may be offset by the shorter load duration. On rougher pavements the additional strains may dominate (increasing fatigue levels) while on smoother pavements the load duration may dominate, leading to decreased fatigue. This is very dependent on the models and actual response of the pavement materials (Gillespie et al, 1993). Hardy and Cebon (1994) found that material frequency dependence might be part of the cause for different pavement reactions at different speeds.

Gillespie et al (1993) found a linear relationship between pavement rut and gross vehicle load magnitude and duration. The layer thicknesses were the only parameter comparable in magnitude of influence on damage to axle load. Increases in load speed caused increased load magnitudes but decreased load durations (increased load frequency). Suspension type did not influence rut development. Single tyres were shown to cause increased rut depths but similar rut volumes (in terms of volume of material displaced) than for dual tyres operating at the same load. Tyre inflation pressure affects the contact area and load duration. Increased tyre inflation pressures cause higher strains but at shorter load durations in the pavement (Chatti et al, 1995). Radial tyres tend to concentrate on the wheelpaths, causing potentially increased rut.

Single axle suspension type has a limited effect on fatigue levels. Tandem axle dynamics, however show a definite influence of suspension type with between 25 and 50 per cent differences in dynamic loads developed due to suspension types. Increased pavement roughness levels may increase fatigue between 50 and 400 per cent (Gillespie et al, 1993).

Kenis et al (1997) found the mean dynamic load independent of the speed, and close to the static load.

De Beer (1991) showed the existence of a phase lag between the point of maximum load and the point of maximum strain. This phase lag was dependent on the load speed. Mamlouk (1987) indicated that this load-deflection time lag is mainly due to inertia effects in the pavement.

The range of compressive to tensile strain of the strain cycle is important when evaluating the fatigue behaviour of the materials in the pavement. Fatigue damage is highly influenced by surfacing thickness with thinner surfacings being affected more than thicker surfacings (Gillespie et al, 1993).

Jooste and Lourens (1998) indicated that static response models could overestimate tensile strains in the asphalt by more than 50 per cent. For the pavements investigated, the effect of transient pavement analysis was more important than the effect of non-uniform tyre inflation pressures, random variations in asphalt stiffness and layer thickness, and visco-elastic effects. Lourens (1992) showed that the stresses and deflections in the pavement structure differ substantially for static and dynamic loads. The existence of a remnant stress in the pavement after the passing of the load was indicated. The magnitude of this stress is dependent on the load speed.

Hardin and Drnevich (1972) found the strain amplitude, effective mean principal stress and void ratio to be of prime importance in the value of the shear modulus (G) and the damping coefficient of the material. They showed that increased strain amplitude cause decreased shear modulus and increased damping ratios. Increasing effective mean principal stress cause increasing shear modulus and damping coefficient, and increasing void ratios cause decreasing shear moduli and damping coefficient. Vuceti et al (1998) indicated that the shape of the cyclic loading cycle affects the measured damping coefficient and should be clearly defined when reporting results.

Heukelom and Klomp (1967) indicated that increased temperatures might cause asphalt stiffness to decrease. De Beer et al (1997b) indicated that increased moisture condition might cause a decrease in unbound granular stiffness. Decreased load durations may cause increases in asphalt stiffness. Stress level mainly influence unbound granular stiffness. Very low densities may cause decreases in stiffness for most materials.

### 2.5.5 Conclusions

The objective of this section was to provide the current state of information regarding pavement components and their importance in dynamic vehicle-pavement interaction, to enable further development and application work to be performed.

Constitutive equations were identified for the four main material types identified as appearing in typical South African pavements.

Two methods for developing the differential equations for analysis of transient response of pavement structures to dynamic loads were identified. Two different solving methods for the problems were discussed.

Current literature indicates that primary pavement response (stresses, strains and deflections) is strongly influenced by vehicle speed. Material parameters are also influenced by factors such as load duration, stress levels and environmental conditions. A thorough definition of operating conditions thus needs to be defined for any analysis of transient pavement response.

## 2.6 South African Vehicle and Pavement Fingerprinting

### 2.6.1 Introduction

This section focuses on identification of typical vehicle and pavement components currently used in South Africa (fingerprinting). These data are needed as input to the analyses in Chapters 5 and 6. The correct input data are vital as the cause for many differences in pavement engineering and the management thereof between South Africa and the USA and Europe, is the different pavement materials, structures and vehicle populations. If relevant and valid conclusions and guidelines for South African conditions are to be developed it is vital to ensure use of typical local information.

The data in this section originate from various sources. The type of data needed is not currently freely available in South Africa. Typical sources are the National Traffic Information System (NATIS), the South African National Roads Agency Ltd (SANRAL), provincial pavement management system (PMS) databases, tyre, suspension and vehicle manufacturers, fleet operators and various other publications citing statistics regarding vehicles and roads.

Some difficulties in collecting this type of information should be appreciated. The component manufacturers and fleet operators are generally not willing to part with information they regard as strategic to their business. Trends and assumptions thus have to be used as input for these parameters. It is expensive in terms of money, manpower and time to sample vehicles to collect the information, and such samples are easily biased due to the location, season or conditions under which the sample is taken.

Pavement structure information is scattered and not always in a user-friendly format. Due to the change from four to nine provinces in 1994, the databases are not all up to date and due to funding problems, it is doubted whether these databases will be running effectively in the near future. Some of the road agencies and departments do still provide a valid service in this regard, and the focus of this fingerprinting effort was on their information. Some bias may thus exist in the information.

The information provided should be viewed as the best available under conditions of secrecy and logistical problems. It is one of the recommendations of this thesis that a thorough fingerprinting of South African heavy vehicles, components and pavements be performed on a regular basis to enable pavement engineers and managers to operate more effectively.

### 2.6.2 Vehicles and components

The vehicle components of importance in this thesis are the tyres, suspension, load and vehicle configuration and dimensions.

Radial tyres are currently increasing in popularity in South Africa, with estimates ranging between 50 and 70 per cent of the heavy vehicle market (Barnard, 1997; Campbell, 1997) and surveys indicating up to 95 per cent (SATMC, 1997) of heavy vehicle tyres on the road to be radial. The most popular heavy vehicle tyre size in South Africa is the 12R22.5 tyre size (between 50 and 59 per cent) followed by the 315/80R22.5 (between 19 and 27 per cent) (SATMC, 1997; Steyn and Fisher, 1997). Although previous studies (Van Niekerk, 1992) identified the use of super single tyres as a possible increasing trend in South Africa, recent information indicates their use to be limited to between 0.8 and 2 per cent of the market (SATMC, 1997; Steyn and Fisher, 1997).

Tyre inflation pressures for heavy vehicles in South Africa range between 150 and 1 000 kPa (Steyn and Fisher, 1997). Data indicate that typical tyre inflation pressures on steering axles for cross-border vehicles is 729 kPa (Standard Deviation of 125 kPa) while the average tyre inflation pressure on non-steer axles are 670 kPa (Standard Deviation of 87 kPa) (Steyn and Fisher, 1997).

Steel suspension is mainly used in South Africa. The current air suspension usage is estimated to be between 5 and 20 per cent of all heavy vehicles, and to be limited mostly to special types of vehicles such as those conveying fragile goods (Campbell, 1997).

The important suspension parameters for evaluating dynamic performance of vehicles include the composite vertical stiffness, composite roll stiffness and damping (Pretorius, 1990). Typical values were obtained from Pretorius (1990) and are shown in the summary in Table 2.5.

The system used for vehicle classification is based on the number of axles of a heavy vehicle. Previous studies (Pretorius, 1990) have used a combination of the number of axles and drive axles on truck-tractors and rigid vehicles, and a combination of truck-tractor axles and trailer axles to classify truck-tractors and trailers with separately. However, as these vehicles are mostly used in combination the author decided to make use of the system where the total number of axles in each of the axle groups on a vehicle or combination are used to classify the vehicle or combination (Nordengen et al, 1995). The thirteen most typical vehicles and/or combinations identified for South African conditions are shown in Table 2.6, together with their respective unladen masses, number of axles, number of axle groups and population as determined for internal use and cross border use.

An attempt was made to establish the percentage of each of the various classes of vehicles on South African roads. The current National Traffic Information System (NATIS) is not up to date and therefore data from the National Association of Automobile Manufacturers in South Africa (NAAMSA) were used to establish the internal heavy vehicle spectrum (NAAMSA, 1998). Data from the annual cross-border surveys undertaken by the CSIR were used to indicate the vehicle population crossing South African borders (Nordengen et al, 1995). As these data are compiled for slightly different parameters than the classification used, the thirteen classes have been simplified to 6 classes for the internal traffic and 3 classes for the cross border traffic.

For the purposes of pavement response the number of axles, distance between individual axles, distance between axle groups and individual axle loads of vehicles and vehicle combinations are important. The data used in this thesis are based on a survey conducted at the Beitbridge borderpost (Steyn and Fisher, 1997). The data compare favourably with published data on specific vehicles. These data are shown in Table 2.6 for three of the most typical combinations used in South Africa. These are the 11, 123 and 1222 classes. These three classes are used for the further analyses in this thesis, as they constitute (according to the available data) three of the most popular vehicle classes, and they further represent a rigid vehicle, a combination of a truck tractor and one semi-trailer (2 separate bodies) and a combination of a truck tractor and two semi-trailers (3 separate bodies).

The maximum allowable legal axle loads and dimensions of heavy vehicles in South Africa, are regulated in the national road traffic act (Wessels, 1996). These data for the three typical vehicle types identified for further use in this thesis are shown in Table 2.5.

Average speeds for heavy vehicles (GVM > 7 000 kg, average number of axles 4.2) on national roads in South Africa were 79.9 km/h (standard deviation 10.2 km/h) (Bosman et al, 1995). This is similar to the legal speed limit of 80 km/h for trucks on the 40 roads included in the calculation.

The vehicle response software (Section 2.4.3) requires the Pitch Moments of Inertia (MoI) of the truck, trailer and payload as input. Typical calculated values are shown in Table 2.6.

### 2.6.3 Pavements and components

The physical pavement components for which information is required are material type and properties, layer thickness, layer combinations and pavement roughness. Selected typical trends and statistics for South African pavements and pavement structures are given in this section. The data should be seen as the most probable breakdown of surfacing and base layer type for South African conditions given the limitations indicated in Section 2.6.1. It is based on information from the national and six of the provincial road authorities.

**Table 2.5: Matrix of vehicle components to be used in this thesis.**

| Parameter | Unit[1] | 11 | | 122 | | | 1222 | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Tyre type | | 12R22.5 Radial | | | | | | | | |
| Tyre inflation pressure | kPa | 730 | 670 | 730 | 670 | | 730 | 670 | | |
| Tyre spring rate | lb/in/tyre | 4 800 | 4 800 | 4 800 | 4 800 | 4 800 | 4 800 | 4 800 | 4 800 | 4 800 |
| Suspension type | | Steel | | | | | | | | |
| Spring rate | lb/in/side | 1 000 | 1 500 | 800 | 5 000 | 6 000 | 800 | 5 000 | 6 000 | 6 000 |
| Viscous damping rate | in/second/ side | 15 | 0 | 15 | 0 | 0 | 15 | 0 | 0 | 0 |
| Coulomb damping rate | | 500 | 1 000 | 500 | 1 000 | 1 000 | 500 | 1 000 | 1 000 | 1 000 |
| Unsprung weight | lb/axle | 1 200 | 2 300 | 1 200 | 2 300 | 1 760 | 1 200 | 2 300 | 1 760 | 1 760 |
| Pitch MOI[2] truck | 1 000 in.lb.sec^2 | 102 | | 147 | | | | | | |
| Pitch MOI Trailer | | | | | | 661 | | | | |
| Pitch MOI payload | | 390 | | 2 310 | | | | | | |
| Permissible Vehicle combination mass | tonne | 16,5 | | 43,5 | | | 56,0 | | | |
| Unladen vehicle combination mass | | 6,5 | | 15,0 | | | 20,0 | | | |
| Achievable payload | | 10,0 | | 28,5 | | | 36,0 | | | |
| Axle load | | 7,7 | 9,0 | 7,7 | 18,0 | 18,0 | 7,7 | 18,0 | 18,0 | 18,0 |
| Length | m | | | | | | 22 | | | |
| Width | | 2,5 | | 2,5 | | | 2,5 | | | |
| Distance between axles on tandem | | 1,4 (0,03) | | | | | | | | |
| Distance between first and second axle groups | | 4,8 (0,47) | | 3,1 (0,56) | | | 3,1 (0,56) | | | |
| Distance between second and third axle groups | | | | | | 6,3 (1,1) | | | 6,3 (1,1) | |
| Distance between third and fourth axle groups | | | | | | | | | | 6,4 (1,5) |

All distances are between the centreline of the last tyre of the first axle group and the centreline of the first tyre of the following axle group.
Values in brackets indicate standard deviation.

[1] A combination of imperial and metric units are shown as sourced from the various references.
[2] Moment of Inertia

**Table 2.6: Heavy vehicle classes and population in South Africa (based on NAAMSA, 1998; Nordengen et al, 1995).**

| Class | Type* | Unladen vehicle / combination mass [tonne] | Number of axles | Number of axle groups | Internal use [%] (NAAMSA, 1998) | Cross border use [%] (Nordengen et al,1995) |
|---|---|---|---|---|---|---|
| 11 | R | < 7,5 | 2 | 2 | 37,9 | 23,2 |
| 12 | R | 7,5-10,0 | 3 | 2 | 5,5 | |
| 111 | TT+ST | | 3 | 3 | | |
| 112 | TT+ST | 10,0-12,5 | 4 | 3 | 11,0 | 17,5 |
| 22 | R+T | | 4 | 2 | | |
| 1111 | R+T | | 4 | 4 | | |
| 122 | TT+ST | 12,5-15,0 | 5 | 3 | 20,9 | |
| 113 | TT+ST | | 5 | 3 | | |
| 123 | TT+ST | 15,0-17,5 | 6 | 3 | 4,1 | 59,3 |
| 1222 | IL | > 17,5 | 7 | 4 | 20,6 | |
| 1222 | TT+T | | 7 | 4 | | |
| 1232 | IL | | 8 | 4 | | |
| 1222 | IL+T | | 7 | 4 | | |

* Indicates the individual vehicle of which the combination consists. R - Rigid, TT - Truck Tractor, ST - Semi-Trailer, IL - Interlink, T - Trailer

These data indicate that approximately 39 per cent of the provincial roads that appear on the pavement management databases are paved. However, if unpaved roads currently not appearing on the provincial databases are included, this figure should decrease to approximately 16 per cent provincial paved roads for the whole country (Steyn, 1997). All the national roads are paved. The percentage breakdown of current surfacing types and base layer material types are given in Table 2.7.

Typical pavement structures are proposed in the catalogue of TRH4 (1996). For the purposes of this thesis three typical pavement structures were selected for analysis. The first structure (resembling typical national roads) consists of an asphalt surfacing with a G1/G2 base layer, the second (resembling a typical provincial road) consists of a seal and a cemented base layer and the third (resembling a typical rural road) consists of a seal and a granular base layer. The structures are shown in Table 2.8. The traffic class for the national road is ES100 (30 to 100 million 80 kN axles / lane) road category A for dry regions, while that for the provincial roads are ES3 (1 to 3 million 80 kN axles / lane) road category B, and ES0,3 (0,1 to 0,3 million 80 kN axles / lane) road category C. These three structures are seen as representative of typical South African pavements.

**Table 2.7: Percentage breakdown of surfacing and base layer material types for South African National and Provincial roads (Steyn, 1997; Kannemeyer, 1997; Provincial PMS data, 1997).**

| Pavement / layer type | | National roads | Provincial roads | All roads |
|---|---|---|---|---|
| Surfacing type | Seal | 46,3 | 82,5 | 75,2 |
| | Asphalt | 44,6 | 2,4 | 11,8 |
| | Slurry | 0,0 | 3,6 | 1,9 |
| | Other | 9,1 | 11,5 | 11,1 |
| Base layer material | G1/G2 Crushed stone | 56,0 | 17,8 | 25,4 |
| | Cemented | 13,0 | 34,7 | 30,4 |
| | Gravel | 9,0 | 37,8 | 32,0 |
| | Other | 22,0 | 9,7 | 12,2 |

**Table 2.8: Typical pavement structures analysed in this thesis (from TRH4).**

| Layer | National road structure | Provincial road structure | Rural road structure |
|---|---|---|---|
| Surfacing | 50 mm Asphalt | Double seal | Double seal |
| Base | 150 mm G1 | 125 mm C3 | 125 mm G4 |
| Subbase | 300 mm C3 | 152 mm C4 | 125 mm G6 |
| Subgrade | 500 mm SG1 | 500 mm SG1 | 500 mm SG1 |

The typical flexible materials found in South Africa are shown in TRH14 (1985). These materials can be classified into natural unbound, natural bound, lightly stabilised, strongly stabilised and asphaltic materials. Each of these types of materials has various subclasses, depending on the origin of the material, the way in which it was treated, the physical properties of the material and the way in which it is normally used.

The focus in this thesis is on the eight types of materials used in the three typical pavement structures shown in Table 2.8. Double seals are typically not assumed to contribute to the structural capacity of pavements, and no constitutive equations exist for them. For the remaining seven materials, constitutive equations and typical material parameters are discussed in Section 2.5.3.

The typical road roughnesses for South African national roads have been shown to be mainly (87,4 per cent) less than and equal to 2,5 IRI. The average is 2,0 IRI with a standard deviation of 0,6 IRI (Kannemeyer, 1998). Based on these values three typical pavements with roughnesses of 1,4, 2,0 and 2,6 are selected for analysis.

### 2.6.4 Summary

Based on the information collected and provided in this section, the matrix of information provided in Tables 2.6 and 2.8 summarises the input data for the analyses to be performed in this thesis. These matrices are based on the parameters that occur most frequently in the various components. From the phenomenological approach taken in this study, it is believed that focus on these major parameters should provide a deeper understanding of the governing principles in dynamic load and pavement response.

## 2.7 Conclusions

The objective of this chapter is to provide relevant background information needed for further studies in the subjects addressed in this thesis. In this regard, the following conclusions are drawn for the five main themes addressed:

**Vehicle-pavement interaction systems.**
Various systems are available for describing vehicle-pavement interaction. The dominant features and limitations of the systems were shown. Major limitations that need attention are the bias towards either vehicles or pavements, the uncertainty regarding the use of behavioural models or performance models, and the level of complexity at which the system should be analysed. A robust system should be developed allowing all the major factors to be evaluated on an equal footing.

**Pavement roughness.**
Pavement roughness is the main cause for dynamic loads. The IRI and TRRI were selected as indicators of roughness levels for this thesis. Actual HSP data will be used as indicators of pavement roughness in this thesis.

**Vehicle components.**
The main vehicle components identified as important in vehicle-pavement interaction are the tyres, suspension, vehicle dimensions, configuration and load. DADS and TFP will be used as vehicle simulation models for this thesis. Dynamic load appears to consist of two distinct phases that needs consideration in vehicle-pavement interaction.

**Pavement components.**
Material constitutive models were identified for the main materials evaluated in this thesis. The different methods available for describing transient pavement response were described. Vehicle speed, stress level and environmental conditions were identified as the major factors in influencing transient pavement response. A good understanding of material properties is needed if a transient pavement response analysis is attempted. More work needs to be done on evaluating the available material constitutive models and parameter values. The actual factors of importance and their relative contributions to pavement response for South African conditions should be evaluated. A standard process needs to be developed for transient analysis of pavement response.

**Fingerprinting.**.

Three typical vehicle combinations and three typical pavement structures were identified as typical of South African conditions, and further analysis in this thesis will focus on these. Difficulties were experienced in obtaining valid information on both vehicle and pavement components. A thorough fingerprinting of the important pavement and vehicle issues should be performed on a regular basis to ensure availability of valid data to industry.

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